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<td>American Society of Civil Engineers</td>
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<tr>
<td>ASME</td>
<td>American Society of Mechanical Engineers</td>
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<td>American Society of Testing and Materials</td>
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PREFACE

The Mid-Continent Transportation Symposium 2000 was the third biennial transportation research conference cosponsored by the Center for Transportation Research and Education (CTRE) at Iowa State University and the Iowa Department of Transportation. The conference and proceedings were developed under a collaborative arrangement between the two organizations formalized in a Memorandum of Agreement in 1992 and expanded in 1996 to include the University of Iowa and the University of Northern Iowa. This year the symposium was held in conjunction with the spring conference of the Missouri Valley Section of the Institute for Transportation Engineers (MOVITE).

The day-and-a-half conference provided an opportunity for Iowans and other Midwesterners to focus on regional and national transportation research and technology transfer issues through presentations of the caliber generally found at national events. The conference allowed transportation professionals from the region to attend a content-rich event without having to travel outside the region.

This document is the set of papers presented at the conference. Twenty-six categories of papers were presented in five concurrent sessions. Papers were included from all areas of interest, ranging from transportation infrastructure design to intelligent transportation systems to transportation policy and asset management systems. Together the 79 presentations at the symposium showcase the variety of research that is shaping the next era of transportation.

Stephen J. Andrle, Director
Center for Transportation Research and Education
Iowa State University
Ames, Iowa
Development and Implementation of a Process for Reconciling Subarea Macro and Micro Scale Modeling Applications

William Troe

Microscale modeling applications for corridor and subarea traffic operations analysis have become an instrumental element in the traffic engineer’s toolbox. These modeling applications, while revolutionary in providing the analyst with the capabilities of dynamic evaluation of alternate concepts, require a significant level of data and include a broad range of assumptions. Key inputs to the models are traffic volumes. The source of these volumes are many times macroscale travel demand forecasting models. These macroscale models may have initially been developed for use in the long-range transportation planning process for the region or have been adapted from the regional model for more detailed application in a specific corridor or subarea. A key focus of the microscale applications over the macroscale operations models has been in reformulating the methods applied in distributing traffic over the analysis period. The applications to date have been successful in obtaining a significant level of support in the analysis advancements, however, the traffic inputs are many times simply products of travel demand model applications developed for macroscale purposes. In reality, the traffic forecasting models need to be adapted prior to expecting the products to be reasonable for use in microscale applications. The purpose of this paper is to provide documentation of a number of alternates evaluated in attempting to establish a methodology for developing peak-hour travel data from an initial regional model for application in a microscale subarea analysis. Within the paper, the strong and weak points of each of the alternate applications evaluated as part of a subarea study are documented. A primary purpose of the subarea study in Lincoln, Nebraska was to develop a subarea analysis methodology for application throughout the metro area. It has been the hope of the authors that the methods in this paper could be used as a beginning point, or to add to the current set of knowledge, for establishing a convenient, reasonable link that would allow regional travel model outputs to be a source for microscale corridor applications.

INTRODUCTION TO THE SUBAREA TRAFFIC ANALYSIS PROCESS

The intent of developing a subarea traffic analysis process was to extend use of the regional travel analysis tools developed for long range transportation planning by refining the data outputs such that they could be used as inputs to the microscale analysis programs used in subregional/corridor analyses. The process developed through working with local planning and engineering is documented in Figure 1, and the key elements are summarized below.

Data Collection

The key to being able to produce reasonable analysis products is using reliable inputs, which can be verified in the field. In order to complete analyses in various subareas of the region, the information listed below by source is required:

- Current daily and peak-hour counts.
- Intersection traffic control data.
- Signal timing and phasing information.
- Proposed increment of land development.
- Transportation network improvements for the study area included in the comprehensive plan.
- Intersection and roadway link lane geometry.
- Additional intersection and link volume data.

Refinement of the Regional Model in the Subarea/Validation

The regional travel model was developed as a tool for use in the long-range regional transportation planning process. In conducting subarea studies, additional detail is needed in the regional analysis tool, or an entirely new travel forecasting tool must be prepared. Based on the local confidence in the regional model, it was determined that, with some refinement in the study area, the model would provide an excellent resource in conducting the subarea analyses. Following model refinement in the study area, validation of the subarea model was required to ensure that the model reacted reasonably in the study area and that the integrity of the regional model was maintained. The validation step was essentially divided into two different types of tests:

- Document the ability to simulate base year volumes. Through the subarea validation process a link daily assignment to actual daily count deviation of approximately 10 percent is recommended as the outside target for the subarea.
- Document that refining the network and zone structure in the subarea does not adversely impact the regional model validation in other areas. Outside the subarea, the refined model assignments for the base year and the final version of the regional model validation should not show significant differences. Through this test, the ability to maintain the integrity of the validated and accepted regional model is demonstrated.

Developing Future Traffic Volumes

Through application of the refined regional model and input of the increment of development associated with a specific growth concept,
average daily traffic (ADT) volumes for a future year would be prepared. For each of the intersections in the study area, peak-hour turning movements would be developed through one or a combination of the following steps:

- Apply current peak-hour percentages and link directional splits to the forecasted future ADT. Using an interactive propensity model, develop turning movements for each of the intersections in the study area.
- Apply a regional model derived change percentage/increment to current count data. The increment of change would be a function of the change between model application base year assignments and future year assignments.
- Apply a control-point expansion method. Develop peak-hour turning movements at a defensible control point using one of the methods listed above, then work away from the control point, smoothing intersection turning movements using a combination of model derived change and current count data.

Traffic Operations Analysis/Alternatives Analysis

Traffic operations at currently signalized intersections, intersections which meet signal warrants based on forecasted volumes and those intersections that city staff believe would be signalized, would be conducted on an isolated and corridor basis. For selected subareas or portions of subareas, a micro-scale corridor simulation analysis may be warranted.

For those intersections which, in the current and/or future conditions, are observed to operate at an unacceptable level of service, various improvement concepts were evaluated based on city guidelines for acceptable improvement. Potential improvement concepts include:

- Adding left and/or right turn lanes.
- Adding through lanes.
- Modifying the signal timing and/or phasing to improve the efficiency of the operations.
- Providing alternate corridors to which traffic from a congested corridor is diverted.
- Modifying the flow directions to convert two-way flow to one-way.
- Reducing vehicle demand through enhancement of TDM strategies or increasing the non-auto mode split.

Documentation

Throughout the subarea analysis process, documentation for defensible decision making is essential. The following list summarizes the general documentation needs throughout the process:

- Memorandum identifying the data collection needs/requirements for the specific study area.
- Memorandum documenting the assumptions and findings of the subarea TranPlan model validation.
- Memorandum of the forecasted daily and peak-hour traffic within the study area.
- Memorandum summarizing the current and future transportation plan network traffic operations.
- Draft report documenting the inputs, assumptions, methods, and results of the transportation system improvement alternatives analysis.
- Final report which incorporates the draft document material and comments by city staff.

SUBAREA ANALYSIS ASSUMPTIONS

The purpose of this chapter of the subarea study process documentation is to provide a summary of the key assumptions applied in conducting analyses of various subareas throughout the local planning area. The following assumptions were used throughout the subarea study analysis:

- Generally, intersection turning count data is not collected on a systematic as is link count data. Thus, for any subarea, intersection turning movement data from a number of years may
need to be blended together to obtain a “base” condition set of intersection data. This process is referred to as smoothing. Through the smoothing process, individual approach movements are adjusted such that the outbound intersection counts at an upstream location, match those of the complementary downstream approach total.

- Focusing versus windowing: In general, there are two methods for adding the detail to the regional model. These concepts are:
  - Windowing the subarea from the regional model: In this method, the subarea network and trip table are extracted from the regional model. The trip interaction between all zones external to the study area and study area zones would be aggregated to the roadways at the limits of the study areas. These roadways would essentially become the refined subarea model external stations. The level of traffic at the subarea external stations would be fixed for a specific period, even if changes to the internal subarea network or trip table developed as part of the subarea study, and would, in reality, modify the travel patterns of users.
  - Focus on the subarea within the regional model: Through this method additional detail is added to the traffic analysis zone (TAZ) system and the network within the study area. The model area outside of the subarea would remain as it was in the validation. As the TAZs are subdivided, the numbering system must be updated, which results in incompatibilities in the number system with the validation run. A benefit of conducting the subarea modeling through focusing is that the resulting application is much more dynamic in the level of interaction with the remainder of the region than results from the windowing application.

Determination of the most appropriate application is a local decision.

- Traffic operations mitigation analysis: For intersections in a specific study area where the results of the current and/or future traffic on the current and/or transportation plan network result the following operating conditions, improvement concepts are to be evaluated as part of the subarea planning process:
  - Overall intersection operations of LOS D operations or worse.
  - Individual lane group operations of LOS E or worse.
  - Unacceptable queue length.
  - Corridor travel time.
  - Corridor operating speed during the peak period.
  - The improvement alternatives evaluated should be multi-faceted, crossing a broad range of types, including:
    - Adding a traffic signal where one does not exist and where the peak hour warrant was met.
    - Providing transportation systems management types of improvements which may include adding left and/or right turn lanes and modifications to signal operations.
    - Including an alternate transportation corridor to which volume could be diverted.
    - Adding corridor through capacity by increasing the number of through lanes.
    - Creating transit system enhancements that have the potential to reducing vehicle traffic through increasing the mode split.
    - Modifying the land development concept to reduce the level of peak hour vehicle tip generation in the congested corridor.
    - Documentation of the subarea analyses should take the form of:
      - Technical memorandum documenting the current traffic count data, lane geometrics, signal timing data and traffic operations.
      - Technical memorandum documenting refinements incorporated into the TranPlan travel demand model, including a summary of the refinement validation results.
      - Technical memorandum documenting the initial mitigation analysis.
      - Draft report documenting the inputs, assumptions, methods and results of the transportation system improvement alternatives analysis. A potential outline for the draft report, including a list of potential tables and figures is included in the Appendix.
      - A final subarea study report which incorporates comments by city staff.
  - Meetings: The typical subarea analysis study should require approximately five to six milestone meetings. These include:
    - Project initiation and data transfer.
    - Model refinement and revalidation findings.
    - Future daily and peak-hour traffic on the transportation plan network.
    - One to two mitigation alternatives meeting/workshops.
    - Review of the draft report.

FINDINGS

An initial pilot test of the subarea planning process was conducted in the North 27th street area of Lincoln. Over the past six to seven years the area has experienced significant development as retail and residential centers. Through the initial application of the subarea process, a number of refinements were incorporated. These along with a number of findings as to the reasonableness of the process are documented below:

- Incremental decision making in the analysis process is critical to maintaining a timetable. There are a number of milestones throughout the process where decisions as to the desired course of action require consensus by the participants. A lack of consensus, at least consent, will likely result in rehashing old issues and will make establishing a defensible solution more difficult.
- For most small to medium-sized metro areas, regional travel model focusing is likely the most desirable process for developing horizon year forecasts (when a regional model is available). The time required to complete a model run in even medium-sized metro areas (500,000 to 750,000) is approximately one to two hours. The level of flexibility and regional interaction associated with maintaining an intact regional model significantly outweighs the time savings associated with a smaller window dataset.
- The control point smoothing technique resulted in the most defensible set of horizon year intersection turning movements.
- Corridor simulation is an important element in the process. Many communities are expanding a one dimensional level of service analysis as a measure of effectiveness in comparing various alternatives or defining unacceptable corridor operations. Simu-
lation programs currently offer the most comprehensive capabili-
ties to compare an expanded set of measures of effectiveness, including:
• Corridor delay.
• Corridor travel time.
• Corridor operating speed.
• Fuel consumption.
• Air quality.
Iowa DOT Weather Information System to Support Winter Maintenance Operations

DENNIS BURKHEIMER

Understanding and interpreting weather information can be critical to the success of any winter snow and ice removal operation. Knowing when, where and what type of deicing material to use for a particular winter weather event can be a challenge to even the most experienced veterans. Knowing where to find the weather information needed to make decisions and what information to use can also be difficult. The Maintenance Division of the Iowa Department of Transportation has taken a number of steps over the past ten years to provide supervisors and operators with the weather information and training they need to make better snow and ice operational decisions. A fifty-site Roadway Weather Information System coupled with a satellite delivered weather information system at nearly every maintenance garage have been sources for real-time weather information. Extensive training has also been included in the process as an important element to understanding how to use this weather information for making operational decisions. The department also provides similar weather information to cities, counties, and the general public through the Internet and rest area kiosks in an effort to educate others on how the weather information can help with their snow and ice operations and provide general traveler information to help motorists make informed decisions on travel.

INTRODUCTION

The winter maintenance techniques and technologies used today are far different than those used in Iowa less than a decade ago. A few years ago, the standard operational practice was to treat roads with a 50/50 mix of sand and salt after receiving the first call from local law enforcement officials warning that roadway conditions were getting slippery. Often, trucks were not dispatched until a preset amount of snow had already accumulated on the surface of the roadway, which may have lead to potentially hazardous travel conditions for the public. The standard of operation at the time was purely reactive. Today, the Iowa Department of Transportation is taking a more proactive approach to winter maintenance by attempting to have equipment, materials, and personnel on the roadway in advance of or at the very beginning of a winter precipitation event to try to reduce the amount of time the roadways are slippery. Anti-icing, the practice of applying chemical deicers before a storm, has been used in Iowa over the past five years, and today the entire 3,200 lane miles of the interstate system plus an additional 3,000+ lane miles of non-interstate roadways are anti-iced when conditions permit. The move to a proactive snow and ice operation was needed because traffic was increasing annually at all hours of the day and society’s expectations were also increasing as the demand for quick delivery of goods and services became a benchmark for customer service. Many businesses also moved to a “just-in-time” delivery of goods and services which meant they no longer maintained large quantities of raw materials or finished goods on hand and were dependent on the transportation for quick delivery of goods. An economic analysis conducted by Standard and Poor’s DRI indicated that it would cost the state nearly $68 million dollars a day in economic losses if the roadway system were shutdown because of a winter storm. (1)

To meet the customer expectations for driving conditions during and after winter storms, the Iowa DOT felt that the use of liquid deicing chemicals would be an important element to enhance snow and ice removal, began purchasing new equipment to support the use of more liquids, and invested time and effort in training operators and supervisors about adopting the proactive stance for snow and ice operations. One of the key elements to the success of this transition was having accurate weather information in the hands of the decision-makers. With reliable and accurate weather information, supervisors could determine when, where, and what materials to apply to the roadway providing for a more efficient snow and ice operation and improved traveling conditions for motorists.

ROADWAY WEATHER INFORMATION SYSTEMS (RWIS)

The department purchased and installed its first RWIS site in Des Moines in 1989 from Surface System, Inc. (SSI). The site consisted of three pavement sensors that collected real-time pavement temperatures and salt concentrations, and a subsurface probe that measured temperatures under the roadway, air temperature, relative humidity, and wind speed and direction. The system was monitored at the local maintenance garage using DOS-based computer software that accessed the weather station with an early version of laptop and telephone modems. Top speed at that time for transfer of data was 1,200 bits per second. In the following years, a number of sites were installed each year, and as new hardware and software were developed, changes were made to the network topology to provide access to the weather information at the local garage and surrounding garages in a timely manner. A central server would call each of the sites every hour to collect current information and, at the same time, drop-off current weather information from the other sites in the state. This data collection process created large cost for long distance telephone calls and the information was often late in arriving to the decision-makers.

By 1999, the department had installed fifty RWIS sites throughout the state, and although most of the weather sensors remained basically the same, the data collection process and software used...
to view the information changed to reduce long distance charges and to provide faster and easier-to-use information. The communication link between the weather tower and the garage has remained the same with the use of modems of telephone or radio links, but now, virtually all calls are local calls with nominal monthly fees, and access rates have been increased to 56,000 bites per second. Once the weather data is moved from the tower to the workstation in the garage, information is now moved back to headquarters using the department’s high speed Local Area Network (LAN). Many of the site garages are connected to the LAN by T-1 lines, while the rest are connected with dedicated 56kps lines with plans to upgrade to T-1 lines in the near future. The central collection center of the system consists of two servers used to collect the information from the fifty sites: one server for archiving data and one server for Web access.

In 1989, the concept of using computers in maintenance facilities was still relatively new, and the DOS-based software used to access the weather information was not easy-to-use for many supervisors and operators. A large amount of training effort at that time was geared toward understanding computers and the software used to access the information, rather than understanding how to use the information to make decisions during winter storms. Today, the system data are available in Hypertext Mark-up Language (HTML) which allows users to access the data with any Internet browser. The department felt that many future applications would be developed using HTML and that using it for accessing RWIS data would reduce the amount of training required for using the software and allow for more concentrated effort on understanding how to use the information for decision-making.

OTHER WEATHER INFORMATION

Having access to local roadway weather information from RWIS was very important for making decisions but access to other weather information was needed to understand the larger weather picture and the impact of approaching weather on operations. In 1994, the department leased satellite delivered weather systems from Data Transmission Network (DTN) for nearly all of its maintenance garages. The systems provide near real-time radar and satellite information along with current and forecast information from across the United States. This information provides supervisors and operators with the ability to watch winter storms as they develop, to check conditions before, during and after a storm to predict the impact on their roadways, and also to use the system in conjunction with their local RWIS data to help make informed decisions. These systems were equipped with only eight buttons to help users find the weather information they needed and were considered to be a very powerful, easy-to-use resource for weather information.

In 1996, with the success of the DTN systems at maintenance garages, the department decided to move data from the RWIS onto the DTN so that access to RWIS information could be available to all maintenance employees in one easy to use box. An agreement was also brokered between DTN and SSI to provide RWIS and DOT forecasts to other government entities in Iowa through the DTN systems for $35 per month with the proceeds split among the three parties. This allowed the department to share RWIS and forecast information to all snow fighters in the state in the hope that it would provide uniformity between systems for the traveling public.

The Internet was also identified as a valuable source of weather information and access was provided to all maintenance garages in 1999. The Internet has also proven to be an excellent resource for other snow and ice information that can be used for training and research.

WEATHER INFORMATION FOR THE PUBLIC

The department felt that providing the traveling public with real-time weather information was an important element for making important travel decisions. It was believed that if motorists could see impediments in the direction of their travel they may seek shelter or make changes in their travel plans.

In 1996, the department installed DTN systems in all rest areas along the Interstate system mounted behind plexiglass and located near the entry. The systems were designed to provide 15 weather and travel related screens to the viewer, changing every 10 seconds. The screens were pre-selected for their ability to provide travelers with near real-time weather information that included radar, satellite, current and forecasted road condition reports, and current weather observations such as temperatures, winds, snowfall and others. The system screens are modified in the summer months to provide more information on roadway construction and summer weather events such as tornadoes, hail, or flooding. One page in the system is being used by the local National Weather Service office to provide motorists with real-time weather watches and warning alerts as they occur.

In January 2000, the department opened a new internet web site called Weatherview (http://www.weatherview.dot.state.ia.us) that combines weather information from the fifty RWIS stations with information from thirty-three Automated Weather Observation Stations (AWOS) used at airports and managed by the department. This site provides current weather information from all reporting sites, is updated approximately every thirty minutes, and provides links to the National Weather Service forecast site and the road condition report operated by the Iowa State Patrol. In one storm, the site received over 10,000 hits from people seeking information. Plans are currently being developed to provide additional enhancements to this site by the winter of 2000-2001, providing the public with more information to help with their travel plans.

TRAINING

Providing weather information at the garage level was a very important to helping move from a reactive snow and ice operation to one that is more proactive and also important to providing training on how to understand and use the information to make operational decisions. When RWIS was first installed, training on how to access the information was provided to the few garages that had systems, but the vast majority of garages received little if any training on RWIS and weather. Once more garages received RWIS sites and the information became more widely available, more training was provided on how to access the information, plus a once-a-year, two-day training session on how to use the information for operational decisions was presented by SSI. The training was excellent but may have been too technical for the users in the audience. In 1997, the department began a “train-the-trainer” program that provided training to a core group of 26 RWIS users from around the state that were expected to
provide training and support to garages in their areas. This would allow more one-on-one training at the garage level and would also help reduce the anxieties often associated with classroom training. It also provided local garages with a peer that understands how to use RWIS and other weather information to make operational decisions and provide guidance on how to get the most from the weather systems.

A six-part video series, covering a number of topics related to winter maintenance, was also developed recently for operators and supervisors. The final video in this series concentrates on weather information and resources available to the users and talks about how to use that information to make operational decisions.

FUTURE

Traffic counts and customer expectations are both expected to increase in the future which will require maintenance forces to search for better and faster ways to remove snow and ice from the roadways. The department continues to research new equipment, deicing chemicals, technologies, and techniques to help meet the demands for better service. Efforts are also being made to provide decision-makers with more accurate weather forecasting through efforts with Iowa State University on a new frost forecast model and participation in a multi-state, FHWA-sponsored operational test of advanced weather forecasting called FORETELL. FORETELL is expected to deliver more accurate weather and roadway specific forecasts at greater resolutions than ever before.

The department is also investing in Global Positioning/Automated Vehicle Location system technology that will automatically track resources and roadway conditions. The system is designed with sensors on snowplows to measure pavement and air temperatures as well as plow and wing positions, truck speed, truck location, and the amount and type of materials being used. This technology may be combined with current efforts with WeatherView to provide the public with the locations of snowplows to help with their travel planning and road condition assessment.

REFERENCES

FIGURES 2 and 3  Data transmission sample displays available at maintenance garages and interstate rest areas

FIGURE 4  Public access to near real-time weather information from the department run Roadway Weather Information System along roadways and the Automated Weather Observation Stations at airports. Links at this site will take users to the current road condition report operated by the Iowa State Patrol and forecast information provided by the local National Weather Service office.
The Advanced Transportation Weather Information System (ATWIS)

MARK S. OWENS

The Advanced Transportation Weather Information System (ATWIS) project, first titled Short-Range Weather Forecasting Decision Support within Rural Advanced Traveler Information Systems, was designed to provide a current road and forecasted weather report to the traveling public and commercial vehicles across approximately 875 interstate miles within North Dakota and South Dakota. This prototype project was to investigate how to merge information and current technologies from both state and private industry to provide in-vehicle decision support data for the traveler. In its fourth year of operations, now covering 9,600 road miles across North Dakota, South Dakota, and Minnesota including interstate, U.S., and state trunk highways, the ATWIS program has grown far past its original goals and objectives. The expansion of the in-vehicle traveler information system has developed an interest in the application of the basic technology to other areas of transportation. This growth has truly created an over-arching weather information system for transportation far beyond the in-vehicle system. However, unlike efforts to adapt current governmental weather forecast products designed for general public safety and air travel, the Advanced Transportation Weather Information System was conceived and designed to provide information specifically for ground transportation, its users and maintainers. This federally funded demonstration and operational project was designed to last only five years with federal funds in a research environment. From the very beginning forward thinking visionaries, both locally and in Washington D.C., set the necessary objectives and plans in place to ensure that this project would begin the process of becoming a commercialized and self-sustaining program by the end of the fifth year. The goal of taking one of the many ITS research projects across the nation from dream to commercial reality by creating value from the application of technology is ready to take its steps out of the world of research and into the world of business. This paper examines the development and operational history of the nation’s first, and currently only, multi-state Advanced Traveler Information System (ATIS). The history will review how and why certain decisions were made during both development and demonstration of the project. We will examine the commercial application of the technology, as the system has grown outside its primary objectives, and the direction a commercial partner has taken ATWIS, or more precisely, its technology base into the business world. While an in-vehicle information system was the driving force that led to the creation of this technology, it spawned additional products and services, through one of its commercial partners, that have increased the accuracy and reduced the overall cost of site-specific weather information to both the traveling public and departments of transportation.

INTRODUCTION

In 1995, an effort was initiated to develop and demonstrate the utility of an in-vehicle traveler weather information system in an effort to put a safer transportation system into effect for the North Great Plains. While traveler information systems have existed across the U.S. in urban areas providing traffic related information, no models existed for the testing and deployment of a rural system designed to provide the travelers with in-vehicle road conditions and weather forecasts for site-specific decision making during their trip.

While much of the technology required to operate this project existed within the current operational research environment, additional integration of computer applications and weather observations was required. These new technologies required combining the technologies of weather analysis/forecasting with the computer representations of spatial and attribute information and developments on refining an infrastructure for collecting, processing, and disseminating information in a framework that permits concept validation. Major changes were needed in the type, location-specific abilities, and timeliness of current forecasts provided to the traveling public.

OPERATIONAL HISTORY

These changes included merging current technologies in weather analysis, weather forecasting, telecommunications, and road condition monitoring to produce short-term site-specific forecasts, together with the development of a rapid and timely dissemination method to each user group (see Figure 1). The requirements of such a system were 24-hour-per-day operations for timeliness, research in mesoscale weather prediction modeling for more finely detailed site-specific forecasts, a central database location accessible by the public, and clear direct lines of communication between the operational forecast center and all weather and road condition data sources available.

This large amount of data fusion required a Decision Support System (DSS) designed to manage data for timely dissemination of short term site-specific nowcasts/forecasts. The DSS evaluation of complex information makes it possible to identify a specific travel corridor and immediately assess and forecast weather conditions.

While weather observations provide valuable current conditions for the travel corridor and must be used to adjust forecasts when necessary, the value of weather information to travelers is greatest when it provides forecasted conditions for a later segment of the travel path. Model forecast products for discrete segments of the travel corridor are generated and updated regularly to produce nowcast products (forecasts from current time to six hours into the future) which reflect the changes to the model projections as based on hourly weather analyses.

Other weather data includes road weather observations from sites across South Dakota and North Dakota. The acquisition of these data is coordinated with the respective DOTs. These data also pro-
vide information on status of road surfaces as it relates to water or ice coverage. Integration of weather information with road attributes yields the capability to simultaneously discern the weather and its potential impact on traffic flow.

Since the work of this project was directed primarily towards assessing the feasibility of generating useful weather information for safe and efficient travel while en route, it was important that a means be available to distribute this information to vehicles. To facilitate a broader dissemination of the information during the demonstration period, cellular communications were selected in order to increase the test base, eliminate additional special equipment cost to the user, and establish a wireless interface for future applications. The University of North Dakota (UND) developed a forecast distribution procedure based upon coded weather information, which was interfaced to a computer telephony system (CT) using interactive voice response (IVR). Relationships were developed with cellular service providers across North and South Dakota. These include all cellular communications bands and as well as the new PCS bands. Over the course of the project, the road miles covered expanded, and the technology advancement within the telecommunication industry brought new companies to the project.

Considerable cost in programming was required initially to activate a special switch (#7233) or (#SAFE) at each cell location across the region. This switch allows the user to dial (#7233) or (#SAFE) to access the CT system (see Figure 2). On average, one minute and 30 seconds later (1:30) the user has the road condition/weather information they need to make a decision. The cellular companies absorbed this programming expense and advertised the special access number. This supplemented the advertising provided by the states in the form of blue information signs along the affected routes (see Figure 3).
On November 1, 1997, the road miles covered by ATWIS increased to 3,200 miles across North and South Dakota. On November 1, 1998, the project increased to more than 4,800 road miles across the two states, and the technology began to transfer to a commercial partner. Through this partner, the system expanded into Minnesota in early 1999, for an additional 2,400 road miles. The ATWIS system now covers three states, for a total of 9,600 road miles (see Figure 4).

![FIGURE 4 Total road miles for the Dakotas in 2000](image)

**Traveler Interface**

After the travelers have answered three to four questions about their location, they receive a site-specific road conditions report (as reported by DOT) and a six-hour weather forecast. This forecast is designed especially for their lane and direction of travel on the roadway reaching out approximately 60 miles in front of the reported location.

While small changes were made to the message content for the traveling public, in February of 2000, after requests from the public to add something in the message dealing with city location, passive navigational information was included. At the end of the message the traveler has the option to repeat the last message, perform another inquiry, listen to a list of roadways in the #SAFE system by state, or leave a question, comment, or recommendation for the operations team. Each comment or question, when a name or phone number is provided, receives a response within two business days. The sample message that follows illustrates the current message with the latest changes underlined.

> "The following road conditions report and weather forecast is sponsored in part by the North Dakota Department of Transportation. For travelers on North Dakota Interstate 94 eastbound from mile marker two hundred seventy-two traveling toward Bismarck North Dakota, traffic speeds are reduced due to poor visibility. Roadway is snow-covered. The Forecast until nine o’clock Central Time this Tuesday evening: Skies will be overcast becoming mostly cloudy. Visibility will be less than one-quarter mile changing to near zero with blowing snow. There will be frequent moderate snowing ending. Winds will be two changing to thirty-five miles per hour gusting to forty from the northwest. Temperatures will range from eight to ten degrees decreasing to minus two to minus six degrees."

While the project goal was en route information to the traveler, the Departments of Transportation requested additional products from the project. A simple text message, 48-hour forecast, specifically designed for each transportation district in both states, is now delivered via e-mail, as well as the World Wide Web. The project began providing eight daily district-specific forecasts for North Dakota, 13 for South Dakota, and a statewide forecast on October 1, 1996. A simple text message forecast was prepared for the state and each transportation district.

**Customer Response and Use**

During the operational demonstration period, user acceptance and use of service was an area of interest to the project. Two types of data were collected by the #SAFE research team and the UND Bureau of Governmental Affairs, acting as an independent evaluator of user acceptance for the project. The UND Bureau of Governmental Affairs conducted user acceptance and option surveys during 1996-1997 and again during the 1997-1998 winter driving seasons. The research team collected, compiled, and analyzed user statistics through the capture of request data by the traveling public.

**The Bureau of Governmental Affairs-Public Use**

Two surveys of cellular telephone owners, conducted during the first six months of operation in the spring of 1997 and again during the winter of 1997-1998, produced quite similar findings. There were no major inconsistencies in the findings from the two survey years. The margin of error for the two surveys is approximately +/- 2.5 percent. The surveys yielded the following results:

- Highway signs were the most frequently reported method of awareness to the #SAFE system, followed by radio/TV advertising.
- Both mail and telephone survey instruments, used during the winter time frame, revealed that 55% of drivers were aware of #SAFE as a direct result of exposure to highway signs, followed by exposure to radio/TV advertising at 27.8%.
- The most important question dealt with whether the traveler believed they would benefit from #SAFE in the future. An overwhelming 94.3% believed in the future safety benefits of #SAFE.

During both years, the Bureau of Governmental Affairs conducted evaluations on the use of the district weather forecasts by transportation department maintenance crew supervisors. Transportation department maintenance crew supervisors were almost all daily consumers of weather information. Almost all used the daily weather forecasts, and most used the forecasts in their planning activities. They rated the forecasts as accurate.

- 95% stated the daily forecasts were helpful in planning.
- 75% stated they altered planning or assignments as a result of the daily weather forecasts.

There was universal agreement, both from cellular telephone owners and maintenance crew supervisors, that the advanced transportation weather information system is beneficial and could benefit travelers and maintainers during periods of bad weather.
The research team performed several evaluations of the statistical data available from the computer telephony. These data include what road segments the travelers requested, the time of day, the number of requests per call, hour, day, or month, and what happened during storms or blizzards.

During this time, a number of questions were answered by reviewing when, where, and for what segments the general public accessed the system. As first expected, the greatest use occurred during bad weather. The 1996-1997 winter driving season turned out to be one of the worst winters on record with 11 Blizzards hitting the test region. These blizzards resulted in double the average snowfall amount, major flooding, and, for the first time in history, closure of the entire interstate system of North Dakota, not once, but twice.

The research team fully expected to see the program thrive during the winter season but believed usage would drop to nothing during the summer and the early morning hours year round. But after the first three years of operations, the traveling public has proven that they want to decide what information is important to them in their decision-making process, and they want the information available at anytime. The system has been accessed every hour of the day and every day of the year. Moreover, the majority of users requested two reports during a single call rather than just the area in their direction of travel.

The #SAFE system cost, as with all research projects, began very high for technology development and integration, at about $650,000 per year for what would be considered very little road miles (4,800). Advances in both forecasting and data management technology reduced the cost to around $250,000 per year. The annual operating cost of the #SAFE was reduced by 38% by the second year of operation, while the #SAFE commercial partner cut operational cost by another 50% by May 1999. The cost has stabilized within a research environment for an operational system (see Table 1).

### TABLE 1 Cost Examples of Research, Technology Improvements, and Commercial Activities

<table>
<thead>
<tr>
<th>Road Miles</th>
<th>Cost per Mile/Month</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beginning of Research Period</td>
</tr>
<tr>
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</tr>
<tr>
<td>2000</td>
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</tbody>
</table>

The addition of the ATWIS commercial partner produced further advancements to the forecaster interface, database systems, and computer telephony, while allowing the system to grow. The ATWIS technology has reached the point where annual growth is not required to build the system. Instead, the system operates more efficiently when an entering state incorporates all state highways at once. The lower operational cost of the system is important for two reasons: construction and annual operations.

Past experience in North and South Dakota proves that the cost recovery methods planned require two to three solid years of public use statistics to develop the market. As the system grows, as well as knowledge and use of the system by the public, statistics will begin to demonstrate the systems valuable services and benefits available to other industries. After the first full year of operation, with reasonable, statistical, and verifiable use, some additional savings could begin to enter the system. As this is a totally new business model and market only estimates can be made at this time as to additional revenue generation through sponsorship activities and yellow page advertisements. These activities would become an ongoing once they are begun, resulting in both income and expenses relating to these activities. Revenue generation would be split 35% advertising, 25% contract acquisition, and 40% cost recovery designed to reduce the overall cost to the state. It is expected that some road miles will be too rural to generate sponsorship income and will require that the state remains lead sponsor. Only time will tell exactly how sponsorship and yellow pages activities will affect the overall cost of the system.

### ATWIS GROWS UP

While the project was originally designed to demonstrate the merging of technology for en route information, the project’s technology transfer company expanded the use and enhanced the basic technology to produce site-specific forecasts for transportation as well as other industries. The company has created a number of products for internet delivery spanning several different industries. The original technology along with the enhancements to the integration technology has greatly reduced the cost associated with providing site-specific individual forecasts for key decision-makers in the transportation, utilities, agriculture, insurance, and emergency management industries to mention just a few. The company’s enhancements of the basic technology seeks to reach an economy of scale in weather prediction and analysis that greatly reduces the cost per forecast while improving the overall accuracy and detail.

The World Wide Web-delivered product allows access to selected sites around the state, providing hour-by-hour forecasts of a number of parameters including pavement temperatures. The map shows the locations of the site-specific forecasts in relationship to current transportation districts as well as the highway system (see Figure 5). Once a site is selected, a twelve-hour table appears, providing an hour-by-hour forecast of air temperature, pavement temperature, dew point, relative humidity, wind speed, wind direction, precipitation rate, precipitation type, precipitation accumulation, snow accumulation, and the likelihood of frost formation.

The number of hours and update cycle is selected by the transportation department, as well as table or graph format for the display. In Figure 6, the table design was selected to provide for easy and quick downloading and printing. Additionally, 12 hour and 24 hour forecast maps for temperature, winds, and precipitation, and current
Recent enhancements have provided additional benefits to the customer by providing information in a table or graphic display, custom configurable by the user. Both the table and the meteogram provide the customer with the ability to select only those parameters needed at that point in time. Depending on the time of year, specific weather conditions, or operation, the user can select parameters that are most valuable to their decision-making process and view those values together. Transportation, agriculture, and construction decisions during any season often depend on certain weather conditions in order obtain the best results from operations. Figure 7 is an example of this product.

CONCLUSION

Two conclusions become very obvious after reviewing all present data of this project. First, the merging of multiple information systems into a seamless architecture for decision support in a rapidly changing environment is possible. And second, not only is the traveling public ready for in-vehicle information systems, but actively seeks it out considering the limited advertising the #SAFE system has produced.

While the merging of technology and multiple information systems can reduce the cost of producing detailed weather forecasts, the human element is still a vital part of the forecasting system. Continued efforts in refining the detail and economy of scale are needed to produce a highly dependable and inexpensive product available to general public from home, office, or car at no cost, or nearly no cost, to the public.
Hollow-Cylinder Tensile Tester for Asphaltic Paving Mixtures

GHAZI G. AL-KHATEEB AND WILLIAM G. BUTTLAR

Obtaining fundamental mechanical properties of asphalt paving mixtures is a key component of performance-related mixture design and production control. Different testing modes, including the Superpave Indirect Tensile Test (IDT) and the direct tension mode, are used to obtain fundamental properties of asphalt mixtures, such as creep compliance and tensile strength. In this study, a hollow-cylinder tensile tester (HCT) was developed and used to obtain such properties for asphalt mixtures at low temperatures. The HCT is a compact, portable, and operationally simple surrogate test device for the IDT. In the HCT test mode, an internal pressure is applied to the inner cavity wall of the hollow cylinder through a flexible membrane, which produces a hoop (tangential) tensile stress in the cylinder. Closed-form equations are presented to determine creep compliance and tensile strength for thick-walled cylinders under ideal conditions. In the event of eccentrically-cored specimens (non-uniform wall thickness) or partial loading of inside hollow cylinder wall, three-dimensional finite element correction factors are presented, which can be used to obtain accurate measures of creep compliance and tensile strength from raw measurements. Two asphalt mixtures, a dense-graded surface course mixture (9.5-mm top aggregate size typical Illinois limestone) and a polymer-modified (PG70-34) sand-asphalt mixture, were tested in this study using both the HCT and IDT. Delrin plastic was also tested as a reference material using the HCT for validation purposes. Although the HCT is still in the early prototype stage, the preliminary results seem very reasonable when compared to the IDT. As expected, differences in tensile strengths between the IDT and HCT were observed due to differences in specimen geometry, size, and stress states. Key words: hollow cylinder, creep compliance, tensile strength, Superpave IDT, tensile test mode.

INTRODUCTION

Flexible pavements compose about 94% of the over two million miles of paved roadways in the United States. Asphalt mixture design must consider both climatic effects and vehicular loading. Asphalt mixtures behave very differently across the typical range of in-service pavement temperatures. Its behavior ranges from elasto-viscoplastic behavior at higher temperatures to nearly elastic, brittle behavior at very low temperatures. Accordingly, different pavement cracking is obtained at different temperature ranges, which require specific fundamental properties for each range, to serve as inputs for performance-related mixture design systems, such as Superpave. At low temperatures, creep compliance and tensile strength are the main fundamental properties of the asphalt mixture required by Superpave. Asphalt mixture creep compliance at low in-service pavement temperature (<0°C) is known to be strongly related to pavement cracking (1).

The Superpave IDT (2, 3) was developed in the Strategic Highway Research Program (SHRP) to obtain creep compliance and tensile strength for asphalt mixtures at low temperatures. However, the Superpave IDT is not a truly practical mixture design tool due to its cost and lack of portability. Therefore, the HCT device was developed to address these disadvantages by providing a simple, inexpensive, and portable test method for obtaining creep compliance, tensile strength, and dynamic modulus at low and intermediate temperatures.

OBJECTIVES

The main objectives of this study are:
· To evaluate the feasibility of the HCT as a surrogate test device for the IDT in obtaining fundamental properties for asphalt mixtures
· To assess the accuracy and repeatability of a prototype HCT

LITERATURE REVIEW

Previous studies dealing with the hollow cylinder testing mode have been reported in the literature. Most of these studies were conducted in the geotechnical field and mostly to obtain compressive, shear, and torsional properties for granular materials. Richardson et al. (4) developed the Nottingham Repeated Load Hollow-Cylinder Apparatus (NRLHCA) to characterize the inherent anisotropy and the anisotropic shear strength properties of fine-grained Leighton Buzzard sand. They concluded that the inherent anisotropy due to the preferred particle distribution induced during the sample preparation procedure affects the ultimate shear strength properties of the material.

Hight and Saada (5, 6) used hollow cylinders to determine compressive, shear, and torsional properties of asphalt concrete. Alavi and Monismith (7) tested hollow cylinders of 228.6 mm (9 in.) and 177.8 mm (7 in.) outer and inner diameters, respectively, under compression along the axis of symmetry. The testing was conducted at moderate and high in-service temperatures (4°, 25°, and 40° C) to evaluate the viscoelastic response characteristics of asphalt-aggregate mixes under dynamic axial and shear loads. The general conclusions from the evaluation were that asphalt concrete can be treated as a linear viscoelastic material for the stress range investigated, the stiffness moduli in both axial and shear loading are essentially independent of stress, and that the interchangeability of time and temperature can be used to define the stiffness response of mixes at a specific temperature over a wide range of time. Crockford (8) also tested hollow-cylindrical specimens under shear and compressive loading to study the effect of principal-plane rotation on permanent deformation in flexible pavements.

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From the literature search conducted, it is clear that the hollow cylinder test mode has never been used to specifically determine the tensile properties of asphalt paving mixtures at low temperatures.

CONCEPT AND KEY ISSUES OF THE HCT

In the HCT test mode, an internal pressure is applied to the inner cavity of a hollow-cylinder specimen to obtain fundamental tensile properties (creep compliance and tensile strength) for the asphalt mixture (Figure 1). The constitutive equations for thick-walled cylinders found in Timoshenko et al. (9) or Ugural et al. (10) are given below. The use of thick-walled cylinders is necessary for asphalt paving mixtures since aggregate size-to-wall thickness ratio is of concern.

\[
\sigma_t = \frac{a^2 p_i}{b^2 - a^2} \left(1 + \frac{b^2}{r^2}\right)
\]

\[
\sigma_r = \frac{b^2 - a^2}{a^2 - b^2} \left(1 - \frac{b^2}{r^2}\right)
\]

Where:
- \(\rho_i\) = Internally-applied pressure
- \(\sigma_t\) = Tangential (hoop) stress, which is tensile
- \(\sigma_r\) = Radial stress, which is compressive
- \(a\) = Inner radius of the hollow-cylinder
- \(b\) = Outer radius of the hollow-cylinder

Tensile properties of asphalt paving mixtures can be obtained using different test modes including direct tension, indirect tension (such as the Superpave IDT-Figure 2), beam testing, and finally the HCT test mode presented in this paper. The only mode with pure tensile stress state is the direct tension mode. However, the direct tension mode has several disadvantages including difficulty in gripping and testing brittle materials in tension, which sometimes results in failure near grips (at the ends) due to stress concentrations. A fundamental measure of tensile strength is a difficulty in beam testing due to neutral axis shifting and stress redistribution after crack initiation. The use of beams instead of cylindrical specimens is also a disadvantage shared by both direct tension and beam testing modes.

FIGURE 1 Concept of the Hollow-Cylinder Tensile Test (HCT)

The HCT test mode has several advantages over other tensile test modes. One of these advantages is the simplicity and portability of the test device. Utilization of gyroratory specimens is also an advantage of the HCT mode. Another advantage of the HCT is the lack of stress concentration at the point of load application, since the HCT involves the application of a uniform internal pressure to produce tension. Some key issues, however, have to be considered in the HCT test mode including: (1) particle size-to-wall thickness ratio; (2) density gradients and broken particles in gyroratory-compacted specimens; (3) tensile strength determination; and (4) containing the pressurizing bladder and accounting for compressibility of the membrane, fittings, and fluid. Some of these key issues have been discussed in details in a previous study (11). In this paper, the validity and repeatability of a prototype HCT test device (Figure 3) is addressed through testing of two asphalt mixtures and a Delrin plastic reference material.

MATERIALS AND EXPERIMENTAL METHODS

Materials

Two asphalt mixtures were produced in this study. The first was a modified sand-asphalt mixture, which was designed for an upcoming demonstration project at the Peoria Regional Airport in central Illinois, to serve as a strain-tolerant reflective crack control interlayer. A PG70-34 polymer-modified binder and a 4.75 mm (No.4) maximum aggregate size gradation were used to prepare this mixture. The second mixture was designed to resemble a standard highway sur-
face mixture used in Illinois. An AC-20 (PG64-22) binder and a 9.5 mm top aggregate size limestone gradation were used to produce this mixture. Specimens were compacted to design air void levels (3% for the sand-asphalt interlayer, and 4% for the surface mixture) using a Brovold/TestQuip Superpave gyratory compactor (SGC). Three hollow cylinder replicate specimens were produced for each mixture at each temperature from the SGC specimens, using a coring machine that produces specimens of 4.025 inch (102.24 mm) inner diameter. The outer diameter is 5.906 inch (150 mm), which is standard for SGC specimens. A Delrin plastic hollow cylinder with the same dimensions was also fabricated to serve as a reference standard.

Testing Methods

Creep tests were conducted at three low temperatures, -20º C, -10º C, and 0º C for a duration of 100 seconds, following Superpave IDT test protocols outlined in AASHTO TP-9 specifications. Strength tests were conducted at –10º C for asphalt mixtures. For Delrin, testing was performed at room temperature (22.2º C). Only creep testing was performed on the Delrin material. Piston movement and applied pressure were monitored in the HCT mode. In the creep tests, a constant internal pressure was applied to the inner wall of the hollow cylinder using a control pressure mode, and the vertical movement of the loading ram was monitored throughout the 100-second test. Two-inch strain gages for testing of asphalt mixtures and 0.5-inch strain gages for testing of Delrin were used as a secondary measure of the tensile strains at the midpoint of the inner wall of the specimen. A constant rate of ram displacement was applied to the specimen in strength tests until failure occurred. Pressure of the cavity pressurizing fluid was monitored throughout the test with a pressure transducer.

Formulas presented in Buttlar et al. (11) were used to obtain creep compliance and tensile strength from raw test measurements. In short, the hoop stress on the inner wall of the HCT specimen is a function of applied pressure in the inner cavity and specimen geometry. The ratio of hoop stress to applied cavity pressure is approximately 2.65; however, the aforementioned formulas allow a more accurate computation of hoop stress based upon the specimen dimensions, eccentricity of the inner cavity, and percent of the inner wall loaded pertaining to each specimen.

Strain was measured using two methods. One method involves estimating hoop strain on the inner wall by virtue of the increase in cavity volume in response to pressure. The increase in cavity volume causes the loading to charge piston. Of course, it is necessary to account for the amount of ram displacement caused by bulk fluid compression, compression of the pressurizing membrane, and movements of fittings, seals, etc. This is accomplished by periodically running a companion test on a very rigid cylinder to determine the magnitude of these effects. A steel hollow cylinder was fabricated for this purpose.

PRELIMINARY RESULTS

Creep Compliance

Initial validation of the HCT was performed using the Delrin plastic reference cylinder. Creep results for Delrin were used to compare strain-gage-based tensile strains and volume-based tensile strains for a material with well-defined properties. Volume-based strains were found to be consistent with strain gage measurements over a range of applied cavity pressures (Figure 4) and loading times (Figure 5). Both measurement systems yielded realistic estimates of Delrin modu-
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However, this series of tests indicated a problem in the flexible seals used to contain the pressurizing membrane, which could lead to more significant errors in tests performed on asphalt concrete specimens. Since the new seal design was not completed at the time of this phase of the test program, specimen strain via strain gage measurements were reported in the ensuing analysis.

FIGURE 4 Strain-gage tensile strain vs. volume-based tensile strain for Delrin at 100-sec. and 22.2 degrees C on outer wall over range of cavity pressures

FIGURE 5 Volume-based tensile strains and strain-gage tensile strains vs. time for pressure of 135 psi and at 22.2 degrees C

FIGURE 6 HCT/IDT creep compliances vs. time at 0 degrees C for polymer-modified sand mixture

FIGURE 7 HCT compliance vs. IDT compliance at 100-sec loading time for polymer-modified sand mixture

FIGURE 8 HCT creep compliance vs. time for polymer-modified sand mixture (3 replicates per temperature)

Figures 6 and 7 present a comparison of creep test results performed on the polymer-modified mixture using the HCT and IDT. The HCT creep compliances were found to match closely with IDT creep compliances at higher loading times, while a slight, yet consistent over-prediction of compliance was noted at shorter loading times. Follow-up testing is planned to study this effect in greater detail. One possible explanation for the differences noted would be that the shape of the applied creep load (step function) for the IDT and HCT tests performed were in fact different. Figure 8 shows increasing creep compliance with higher temperatures and longer loading times from HCT measurements, as would be expected. As shown in Figure 8, creep compliance measurements on replicate specimens showed good repeatability at each of the three test temperatures.

Tensile Strength

Unlike fundamental measures of modulus or compliance, which are, by definition, independent of test mode, tensile strength is generally dependent upon specimen geometry, size, and stress states. For instance, the ratio of direct tensile strength of Portland cement concrete (PCC) to flexural strength ranges from 0.30 to 0.77, and the ratio of the direct tensile strength to splitting strength can be as low as 0.41.
for PCC (13). In the IDT, a distinction is made between ultimate tensile strength and true tensile strength. The surface mounted sensors on IDT specimens are used to determine the time of “first failure,” by identifying the time and corresponding load at which the maximum difference between vertical and horizontal deformations on each side of the specimen is reached (12). True tensile strength is found to be, on average, about 80 percent of the strength based upon ultimate load.

Similarly, it was found that first failure does indeed occur on the inner wall of HCT specimens, where tensile stresses are highest (Figure 9), and additional pressure is required to completely fracture the HCT specimen. After failure, HCT specimens are typically split vertically on one side. Figure 9 shows typical data from an HCT strength test. For this specimen, the ultimate pressure occurred at 5.9 seconds (t-ult), while the failure time from the strain-gage measurement on the inner wall was 4.6 seconds (t-fail). An average value of 7.11 MPa for the HCT tensile strength was obtained when the pressure at t-ult was used, whereas, a 4.73 MPa average tensile strength was obtained when the stress at the t-fail was used. Figures 10 and 11 present comparisons between HCT and IDT strengths. The HCT strengths taken at t-fail are closer in magnitude to IDT strengths than those taken at t-ult, yet are still higher in magnitude. However, the HCT strengths taken at t-ult show better repeatability. This is probably because the strain gages used on the inside wall of the HCT specimens only spanned across 1/6th of the inner circumference.

A comprehensive testing program is underway, which will employ crack detection foils, additional strain gages, and a greater variety of materials in an effort to gain a better understanding of failure behavior in the HCT. The testing program will also seek to quantify the accuracy and precision of the HCT device, particularly as a function of maximum aggregate size. In addition to creep and strength tests, the HCT will also be used to measure dynamic modulus over a range of temperatures.

**SUMMARY AND CONCLUSIONS**

A hollow-cylinder tensile tester (HCT) was developed to obtain fundamental tensile properties for asphalt paving mixtures at low and intermediate temperatures. The HCT was designed to be a simple and portable surrogate test for the Superpave IDT. Preliminary tests have now been performed on asphalt concrete and Delrin plastic reference specimens. The following conclusions can be drawn based upon the findings of this study:

1. The HCT device has many potential advantages over conventional tensile test modes for obtaining fundamental tensile properties of asphalt mixtures. The HCT test mode is capable of inducing tension on brittle materials, such as asphalt mixtures at low temperatures, with relatively minimal stress concentrations.
2. HCT tests conducted to date suggest that accurate, fundamental measures of creep compliance are achievable with the HCT.
3. The HCT yields tensile strengths that are higher than the IDT, particularly when the breaking pressure (at t-ult) is used to compute tensile strength.
4. More testing is still needed to validate the accuracy and repeatability of the HCT. In particular, a new bladder sealing system and a better understanding of the failure behavior are needed.
ACKNOWLEDGEMENT

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REFERENCES

Effects of Sample Preconditioning on Asphalt Pavement Analyzer Wet Rut Depths

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Moisture damage of asphalt mixes, better known as stripping, is a major distress affecting pavement performance. AASHTO T283 has historically been used to detect moisture susceptible pavements through the determination of a tensile strength ratio (TSR). Results from AASHTO T283 have been inconsistent. As a result there has been increased interest in finding an alternative test. Preliminary indications reveal that loaded wheel rut testers, such as the Asphalt Pavement Analyzer (APA) have the potential to detect moisture susceptible mixtures. To date, no standard test methodology has been developed. The objective of this study was to evaluate the effects of sample preconditioning on APA rut depths. Eight different mixes from seven project sites were evaluated with the APA. Samples were tested using four different preconditioning procedures: dry, soaked, saturated, and saturated with a freeze cycle. The results were compared with TSR values as well as other aggregate tests. The results indicate that the APA can be utilized to evaluate the moisture susceptibility of asphalt mixes. Additionally, the results indicate that harsher preconditioning of saturation and saturation with a freeze cycle did not result in increased wet rut depths. Using only dry and soaked conditioning appears to be adequate.

INTRODUCTION

When the adhesive bond between asphalt and aggregates is loosened or weakened by the action of moisture, we say that stripping has occurred. The damaging effects that can result include rutting and cracking due to shear forces. Although the phenomenon of stripping has been acknowledged for over 50 years, being able to predict the moisture susceptibility of aggregates has not been adequately solved. Part of the attention of the Strategic Highway Research Program (SHRP) was focused on determining a test method to evaluate the moisture damage potential of aggregates. This research was not completely successful. The recommendations from SHRP were to continue using AASHTO T283, “Resistance of Compacted Bituminous Mixture to Moisture Induced Damage.” Besides the occasional inability of AASHTO T283 to accurately determine moisture susceptibility, the test is also time intensive (three to four days to complete). Thus, a test method that would accurately predict stripping potential and take hours rather than days to complete would be attractive to highway agencies and contractors alike.

Research by the Colorado DOT (1) and the Georgia DOT (2) has shown that loaded wheel testing devices can be used to identify moisture sensitive mixes. Because rutting is one of the symptoms of stripping, developing a test method with these devices is very logical. Additionally, the loaded wheel device used in this study, the Asphalt Pavement Analyzer (APA), has the ability to test samples while they are submerged in water providing a more direct simulation of water-asphalt interaction.

The objective of this study was to evaluate the effects of sample preconditioning on APA rut depths. The tensile strength ratios (TSR), methylene blue values, and sand equivalents of the samples were also evaluated and compared with APA rut depths. Additionally, the viability of using of the APA in predicting moisture susceptible mixes was evaluated.

THE EXPERIMENT

Materials

Eight different mixes from seven project sites in Kansas were used. The initial intent was to have at least two mixes in each of the “good,” “fair,” and “poor” TSR categories without any anti-stripping agent being applied. In other words, a couple of the mixes should easily pass T 283 (good, TSR > 90), a couple of the mixes should have TSRs in the 75 to 85 range (fair), and a couple of the mixes should have TSRs less than 70 (poor). However, in the end, two mixes had TSRs in the 90s, four mixes that had TSRs between 75 and 85, and two mixes that had TSRs between 70 and 75.

Aggregates and asphalt cement were obtained from each project and samples were compacted at the optimum asphalt content to 7% ± 0.5% VTM using the Superpave Gyratory Compactor (SGC). The asphalt cement was either a PG 58-22 or a PG 58-28. Seven of the eight mixes were surface mixes and one was a binder mix. Two mixes were Superpave mixes (mix designation S). The Kansas Department of Transportation (KDOT) made all of the samples, except for those used at Site 7. The University of Kansas made Site 7 samples. Table 1 shows a summary of the material characteristics and mix designation of the eight mixes.

Asphalt Pavement Analyzer (APA)

The APA holds six SGC compacted cylindrical samples (approximately 150 mm x 75 mm) for testing simultaneously. The air and water bath temperatures of the APA can be controlled. Air
temperatures and water bath temperatures of 40°C were used. Rutting is attained by cycling 0.44 kN (100 lb.) loaded wheels on rubber hoses that have air pressures of 690 kPa (100 psi). After an initial zero-reading is made, the APA can be set to cycle as many times as desired. For this study rut depth measurements were obtained at 500, 1,000, 2,000, 4,000, and 8,000 cycles.

Test Plan

Preconditioning

Generally, two samples from each project site and at each condition state were tested in the APA. Four preconditioning states were tested. The first preconditioning state was accomplished by placing the samples in the APA at a chamber temperature of 40°C for four hours prior to running the APA. This condition state is referred to as 40°C dry. The second preconditioning state was accomplished by soaking the samples in a 40°C water bath for 2 hours prior to running the APA. In this condition state, the samples were tested in the APA while submerged in 40°C water. This condition state is referred to as 40°C soak. In the third preconditioning state, the samples were vacuum saturated in accordance with AASHTO T283 and then placed in a 60°C water bath for 24 hours. Next, the samples were placed in the APA’s water bath at 40°C for two hours and then tested in the APA while submerged in 40°C water. This condition state is referred to as 40°C saturated. In the fourth preconditioning state, the samples were treated the same as the third state, using the optional freeze cycle of AASHTO T283. As in the previous two condition states, the samples were placed in the APA’s water bath for two hours at a temperature of 40°C and then tested submerged in the APA in 40°C water. This condition state is referred to as 40°C freeze.

TSR, Methylene Blue, and Sand Equivalent Testing

The KDOT provided TSR values, methylene blue values, and sand equivalents. Methylene blue values were not available for Site 7. The results are shown in Table 1.

Data Analysis

After testing in the APA, the rutting data at 8000 cycles was analyzed using a two-way analysis of variance (ANOVA) in which rut depth was the response variable (or Y variable) and project site and condition state were the two effects (or X variables). Additionally, the Tukey-Kramer method was used to determine where there was a statistically significant difference between the means of the response variables. Finally, comparisons of TSR values, methylene blue values, sand equivalents, rut depths, and rut ratios were completed. Rut ratio is defined by the following equation:

$$\text{Rut ratio} = \frac{\text{Rut Depth at Condition State X}}{\text{Rut Depth at 40°C Dry}}$$

RESULTS

Analysis of Variance (ANOVA)

A two-way ANOVA was performed using the four condition states, and the results are shown in Table 2. The results clearly show that the rut depth variation was due to the effects of the whole model as opposed to chance. The F ratio for the whole model was 25.42 and the probability of a greater F value occurring if the variation of the rut depth resulted from chance alone was less than 0.0001 (Prob.>F). The results also show that project site alone, condition state alone, and project site - condition state interaction all had significant effects upon the variation of the rut depth.

<table>
<thead>
<tr>
<th>Source</th>
<th>Degrees Freedom</th>
<th>Sum Squares</th>
<th>Mean Square</th>
<th>F Ratio</th>
<th>Prob. &gt;F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site</td>
<td>7</td>
<td>160.58</td>
<td>22.94</td>
<td>74.0</td>
<td>0.0001</td>
</tr>
<tr>
<td>Conditioning</td>
<td>3</td>
<td>30.90</td>
<td>10.30</td>
<td>33.2</td>
<td>0.0001</td>
</tr>
<tr>
<td>Site * Cond.</td>
<td>21</td>
<td>55.13</td>
<td>2.62</td>
<td>8.4</td>
<td>0.0001</td>
</tr>
<tr>
<td>Error</td>
<td>30</td>
<td>9.39</td>
<td>0.31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>61</td>
<td>256.00</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

A statistical comparison using the Tukey-Kramer test was completed on the means of the main effects of site and condition state. The Tukey-Kramer test compares the actual difference between group means with the difference that would be significantly different (4). The difference needed for statistical significance is called the least significant difference (LSD). The results of this comparison test on the sites are shown in Table 3. The results indicate that the majority of the sites had significantly different rut depths.

<table>
<thead>
<tr>
<th>Grouping*</th>
<th>Mean Rut Depth (mm)</th>
<th>Site</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>7.95</td>
<td>Site 2</td>
</tr>
<tr>
<td>B</td>
<td>5.93</td>
<td>Site 1</td>
</tr>
<tr>
<td>C</td>
<td>5.17</td>
<td>Site 6B</td>
</tr>
<tr>
<td>C &amp; D</td>
<td>4.83</td>
<td>Site 4</td>
</tr>
<tr>
<td>D &amp; E</td>
<td>4.36</td>
<td>Site 7</td>
</tr>
<tr>
<td>E</td>
<td>4.08</td>
<td>Site 5</td>
</tr>
<tr>
<td>F</td>
<td>2.83</td>
<td>Site 3</td>
</tr>
<tr>
<td>F</td>
<td>2.44</td>
<td>Site 6A</td>
</tr>
</tbody>
</table>

* Means with the same letter not significantly different.

TABLE 1  Material Characteristics of Samples by Site

<table>
<thead>
<tr>
<th>Location</th>
<th>Mix Designation</th>
<th>PG Grade</th>
<th>VTM</th>
<th>TSR</th>
<th>Methylene Blue</th>
<th>Sand Equivalent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site 1</td>
<td>SM-1T</td>
<td>58-22</td>
<td>7.0</td>
<td>82.5</td>
<td>5.5</td>
<td>78.0</td>
</tr>
<tr>
<td>Site 2</td>
<td>BM-2A</td>
<td>58-28</td>
<td>6.7</td>
<td>73.5</td>
<td>14.0</td>
<td>79.5</td>
</tr>
<tr>
<td>Site 3</td>
<td>BM-2</td>
<td>58-22</td>
<td>6.8</td>
<td>84.5</td>
<td>6.5</td>
<td>69.5</td>
</tr>
<tr>
<td>Site 4</td>
<td>BM-2A</td>
<td>58-28</td>
<td>6.7</td>
<td>98.2</td>
<td>29.0</td>
<td>77.0</td>
</tr>
<tr>
<td>Site 5</td>
<td>SM-2C</td>
<td>58-28</td>
<td>7.4</td>
<td>92.5</td>
<td>19.5</td>
<td>77.5</td>
</tr>
<tr>
<td>Site 6A</td>
<td>BM-1</td>
<td>58-22</td>
<td>6.6</td>
<td>83.1</td>
<td>8.0</td>
<td>68.0</td>
</tr>
<tr>
<td>Site 6B</td>
<td>BM-1</td>
<td>58-22</td>
<td>6.7</td>
<td>77.2</td>
<td>10.0</td>
<td>61.5</td>
</tr>
<tr>
<td>Site 7</td>
<td>BM-1</td>
<td>58-22</td>
<td>7.0</td>
<td>74.8</td>
<td>*</td>
<td>76.0</td>
</tr>
</tbody>
</table>

*Values not available.
The results of the Tukey-Kramer test on the preconditioning states indicated that the means of the 40°C dry, 40°C saturated, and 40°C freeze condition states were not significantly different. This result was somewhat unexpected. It means that the AASHTO T283 preconditioning had little effect upon the rutting results. As shown in Table 4, the 40°C soak preconditioning had the greatest rut depth followed by the 40°C saturated, followed by the 40°C dry, followed by the 40°C freeze, which had the least amount of rutting. A one-way ANOVA completed on each of the eight sites indicated that the probability of a greater F value (Prob>F) occurring by chance alone was less than 0.05 for each site. Thus, the effect variable, conditioning, was a significant factor in the variability of the rut depth data.

**TABLE 4 Comparison of Group Means for Sample Conditioning, Tukey-Kramer Test**

<table>
<thead>
<tr>
<th>Grouping</th>
<th>Mean Rut Depth (mm)</th>
<th>Sample Conditioning</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>5.88</td>
<td>40°C Soak</td>
</tr>
<tr>
<td>B</td>
<td>4.29</td>
<td>40°C Saturated</td>
</tr>
<tr>
<td>B</td>
<td>4.23</td>
<td>40°C Dry</td>
</tr>
<tr>
<td>B</td>
<td>4.09</td>
<td>40°C Freeze</td>
</tr>
</tbody>
</table>

* Means with the same letter not significantly different.

**Correlation Analysis**

As discussed previously, the TSR value generated by AASHTO T283 is the current accepted measure for moisture susceptibility. However, in Europe there are several other aggregate tests that are used in the evaluation of aggregates including the methylene blue and sand equivalent tests (1). Therefore, these two tests were included in the analysis to determine if any correlation exists. The comparison includes the rut depths at 8000 cycles with 40°C dry and 40°C soak conditioning and the rut ratio of the 40°C soak rut depths to 40°C dry rut depths as defined by equation 1. The correlation coefficients between the pairs were poor, generally less than 0.5. The best correlation was between TSR and the methylene blue test (r=0.70).

**Threshold Value**

Table 5 ranks the eight sites from best to worst in each of the test categories. The site number is provided first followed by the test parameter in parenthesis. There is little consistency in ranking between the test results. The TSR and methylene blue test results were fairly consistent, and the rut depth results at 40°C dry and 40°C soak were also fairly consistent. However, the rankings from the sand equivalent results and the rut ratio results do not appear to correlate with any of the other categories.

As shown in Table 5, the APA was not able to identify all the sites with TSR values below 80%, the usual specification criteria. However, AASHTO T283 is not infallible either. All sites with TSR values less than 80% had soaked rut depths greater than 5.00 mm. Two sites with TSR values above 80%, Sites 1 and 4, had soaked rut depths greater than 5.00 mm as well. Site 4 (TSR = 98.2%) is unstable with a dry rut depth of 5.93 mm and should not be used. Site 1 had a TSR of 82% but a wet rut depth of 10.6 mm and a rut depth ratio of 1.7, indicating moisture damage potential. A threshold value of 5.00 mm for 40°C soaked preconditioning differentiated between mixes with low TSR’s (< 80%) and mix instability from those with satisfactory TSRs, greater than 80%.

**CONCLUSIONS**

1. The effect of the AASHTO T283 sample conditioning on rut depths did not yield significant differences. It appears that the conditioning by saturation and the optional freeze cycle of AASHTO T283 are not necessary to evaluate moisture susceptibility of asphalt mixes by APA rut depths. As was indicated earlier, the saturated conditioning was performed in accordance with AASHTO T283, and this resulted in saturation levels of 60% to 70%, higher than that measured in the soaked samples. It is possible that the higher saturation levels resulted in excess pore water pressure being developed during the cyclic loading. This excess pore water pressure could help support the load resulting in reduced rut depths when compared to the soaked samples.

2. The APA rut depths of the 40°C soak conditioning were significantly greater than the other conditioned rut depths. With additional refinement, a method to utilize the APA in evaluation of moisture susceptibility of asphalt mixes could be developed. Preliminary results indicate that a threshold value of 5.00 mm for 40°C soak rut depths would be appropriate.

3. The sand equivalent and rut ratio results do not correlate well with the TSR values. However, the methylene blue values do have a fairly good correlation with the TSR values.

**TABLE 5 Comparison/Ranking of Test Results by Site***

<table>
<thead>
<tr>
<th>TSR</th>
<th>Methylene Blue</th>
<th>Sand Equivalent</th>
<th>40°C Dry Rut Depth</th>
<th>40°C Soak Rut Depth</th>
<th>Rut Depth Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 (98.2)</td>
<td>4 (29.0)</td>
<td>2 (79.5)</td>
<td>6A (1.83)</td>
<td>3 (3.08)</td>
<td>3 (0.95)</td>
</tr>
<tr>
<td>5 (92.5)</td>
<td>5 (19.5)</td>
<td>1 (78.0)</td>
<td>3 (3.25)</td>
<td>6A (3.60)</td>
<td>6B (0.99)</td>
</tr>
<tr>
<td>3 (84.5)</td>
<td>2 (14.5)</td>
<td>5 (77.5)</td>
<td>5 (4.73)</td>
<td>4 (1.01)</td>
<td></td>
</tr>
<tr>
<td>6A (83.1)</td>
<td>6B (10.0)</td>
<td>4 (77.0)</td>
<td>1 (3.88)</td>
<td>7 (5.00)</td>
<td>7 (1.08)</td>
</tr>
<tr>
<td>1 (82.5)</td>
<td>6A (8.0)</td>
<td>7 (76.0)</td>
<td>7 (4.63)</td>
<td>6B (5.53)</td>
<td>5 (1.35)</td>
</tr>
<tr>
<td>6B (77.2)</td>
<td>3 (6.5)</td>
<td>3 (69.5)</td>
<td>6B (5.58)</td>
<td>4 (6.00)</td>
<td>2 (1.36)</td>
</tr>
<tr>
<td>7 (74.7)</td>
<td>1 (5.5)</td>
<td>6A (68.0)</td>
<td>4 (5.93)</td>
<td>2 (8.50)</td>
<td>6A (1.97)</td>
</tr>
<tr>
<td>2 (73.5)</td>
<td>N/A</td>
<td>6B (61.5)</td>
<td>2 (6.25)</td>
<td>1 (10.60)</td>
<td>1 (1.70)</td>
</tr>
</tbody>
</table>

* Site # followed by test parameter in parenthesis.
N/A = Data not available.
REFERENCES


VMA as a Design Parameter in Hot-Mix Asphalt

WALTER P. HISLOP AND BRIAN J. COREE

Current research at Iowa State University on behalf of the Iowa Department of Transportation has focused on the volumetric state of hot-mix asphalt (HMA) mixtures as they transition from stable to unstable configurations. This has traditionally been addressed during mix design by a minimum voids in the mineral aggregate (VMA) requirement, based solely upon the nominal maximum aggregate size. The current research addresses three maximum aggregate sizes (19mm, 12.5mm, and 9.5mm), three gradations (coarse, fine, and dense), and combinations of natural and manufactured coarse and fine aggregates. Specimens are compacted using the Superpave Gyratory Compactor (SGC), conventionally tested for bulk and maximum theoretical specific gravities, and physically tested using the Nottingham Asphalt Tester (NAT) under a repeated load confined configuration. The results clearly demonstrate that the volumetric conditions of an HMA mixture at the stable/unstable threshold are influenced by the maximum aggregate size, gradation and aggregate shape and texture. The currently defined VMA criterion, while significant, is seen to be insufficient by itself to correctly differentiate sound from unsound mixtures. Under current specifications, many otherwise sound mixtures are subject to rejection solely on the basis of failing to meet the VMA requirement. The results of the current research project suggest a new set of volumetric design parameters that explicitly take into account such factors as aggregate gradation, shape, and texture.

INTRODUCTION

In Superpave, the volumetric design of asphalt mixtures requires consideration of air voids, voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA). The percent air voids is used as the basis for selecting the asphalt binder content. VMA is defined as the sum of the volumes of the air voids and the unabsorbed binder in the compacted specimen. VFA is the percentage of VMA containing asphalt binder. It is widely accepted that these volumetric properties are useful in predicting hot-mix asphalt pavement performance. Excessive air voids or VFA and inadequate VMA suggest potential durability problems. Insufficient air voids or excessive VFA indicate potential rutting problems.

Over the years, mix designers have established standards of maximum and/or minimum limits on these volumetric properties to exclude poor performing asphalt mixes. The current Superpave guidelines are shown in Tables 1 and 2. The VFA requirements originated with the Corps of Engineers in the late 1940s (1). The VMA requirements date back to 1959, when Dr. Norman W. McLeod first proposed a relationship between “critical” minimum VMA and nominal maximum aggregate size for dense graded mixtures (2). In 1962, the Asphalt Institute dropped the VFA requirements in favor of a minimum VMA requirement in their Marshall mix design guidelines (3).

The Asphalt Institute reinstated a VFA requirement in 1994 in conjunction with the minimum VMA requirement (4).

In Superpave, meeting McLeod’s minimum VMA requirement is a deciding factor on whether or not an aggregate blend can be used. In recent years, some researchers have presented concerns that these minimum VMA requirements are too restrictive and may rule out economical mixes with acceptable performance properties (5). Others point out that evaluating and selecting the aggregate gradation to achieve VMA is the most difficult and time-consuming step in the Superpave mix design process (6). Others suggest it is not applicable to all asphalt mixtures and propose refinements to it (7).

<table>
<thead>
<tr>
<th>Nominal Maximum Aggregate Size</th>
<th>Minimum VMA, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5 mm</td>
<td>15.0</td>
</tr>
<tr>
<td>12.5 mm</td>
<td>14.0</td>
</tr>
<tr>
<td>19 mm</td>
<td>13.0</td>
</tr>
<tr>
<td>25 mm</td>
<td>12.0</td>
</tr>
<tr>
<td>37.5 mm</td>
<td>11.0</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Traffic, ESALs</th>
<th>Design VFA, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 3 x 10^6</td>
<td>70–80</td>
</tr>
<tr>
<td>&lt; 1 x 10^6</td>
<td>65–78</td>
</tr>
<tr>
<td>&lt; 3 x 10^6</td>
<td>65–78</td>
</tr>
<tr>
<td>&lt; 1 x 10^7</td>
<td>65–75</td>
</tr>
<tr>
<td>&lt; 3 x 10^7</td>
<td>65–75</td>
</tr>
<tr>
<td>&lt; 1 x 10^8</td>
<td>65–75</td>
</tr>
<tr>
<td>&lt; 3 x 10^8</td>
<td>65–75</td>
</tr>
</tbody>
</table>

RESEARCH OBJECTIVES

The research objective was to determine the validity of the minimum VMA requirement vs. nominal maximum aggregate size required in Superpave volumetric mix design. The project sought to fulfill three specific objectives:

1. To establish a laboratory method by which the transition of an asphalt paving mixture from sound to unsound behavior may be credibly identified and measured.
2. To use that method to identify and evaluate statistically the effects of aggregate-related factors on the critical state of such mixtures.
3. To derive a predictive relationship relating critical state (e.g., critical VMA) to aggregate-related properties such as nominal maximum aggregate size, gradation, shape, and texture.

This paper presents the results of objectives 1 and 2. Work is in progress on objective 3.
EXPERIMENTAL TEST PROGRAM

Experimental Design

The test matrix used in the study is presented in Table 3. As shown, a total of 36 blends were studied; nine gradations comprised of four different aggregate blends. The four blends selected were:
1. Manufactured: Each gradation is 100 percent crushed material.
2. 50-50 Blend: Each gradation is a blend of 50 percent crushed and 50 percent natural material on each sieve size.
3. Manufactured Fine-Natural Coarse (MFNC): The material passing the #4 sieve was 100 percent crushed and the material retained 100 percent natural. The coarse (natural) aggregate was washed to make sure the P200 (75 mm) material was entirely from the crushed aggregates.
4. Natural: Each gradation is 100 percent natural material.

Comparing these four blends should distinguish the effects of gradation and shape for both the fine and coarse aggregates. For example, comparing the manufactured with the MFNC would emphasize the effects of the coarse aggregate shape and texture. Comparing the MFNC with the natural blends would examine the shape and texture effects of the fine aggregate to be examined.

For each blend, two specimens at 4, 5, 6, 7, and 8% asphalt content were fabricated and tested to determine the critical state volumetric properties. These properties were then compared with the values shown in Tables 1 and 2 to “validate” the Superpave guidelines.

Materials

Asphalt Binder

While the binder was not intended to be a variable in the study, it was important that it be of a typical performance grade used in Iowa. Jebro, Inc., of Sioux City, Iowa supplied ten 5-gallon pails of a conventional PG58-28 binder. The binder test results and specification requirements are listed in Table 4.
Once the aggregates were selected and sufficient quantities were obtained, each aggregate needed to be characterized using conventional tests. The aggregates were dried and sieved prior to testing. Once this was done, the aggregate gradations used in the study could be selected. For the project, three nominal maximum aggregate sizes (19mm, 12.5mm, and 9.5mm) were used. For each nominal maximum aggregate size, fine, dense, and coarse gradations were carefully blended in the laboratory. The gradations used in the study are shown in Table 5.

### LABORATORY TESTING

#### Specimen Preparation

For the most part, Superpave mix design procedures (AASHTO TP4-93) and applicable British testing standards for the Nottingham asphalt tester (NAT) were followed. However, to expedite testing and conserve materials, two important changes were made.

1. **Specimen size:** The NAT equipment limits specimen height to approximately 115–120 mm in height, which would seem adequate for normal SGC specimens of 4500–4700 g. The Transport Research Laboratory (TRL) researchers who worked with the NAT used specimen heights of 60 mm and 90 mm (8). The Superpave specimens are considerably larger than these and would require some care in mounting the linear variable displacement transducers (LVDTs) used to measure axial deformation. Once the decision was made to use smaller specimens, it was decided to target a specimen height of 80 mm, which would also conserve asphalt and aggregates. It was found that a batch weight of 3375 g would provide the right specimen geometry and optimize materials.

2. **Unsawn/polished specimen ends:** The TRL researchers recommend cutting and polishing the specimen ends prior to testing in the NAT, as they will reduce end effects and can be specified and reproduced by independent laboratories. However, since the same specimens would be used to determine the theoretical maximum specific gravity, it was determined that cutting and polishing would be omitted. The SGC compacted specimens were examined for levelness and seemed adequate. Of course, prior to testing they were liberally coated with a silicon-teflon grease to mitigate end effects.

Mixing and compaction temperatures of 147°C and 135 °C were used throughout the study. A two-hour short-term aging period was used prior to compaction. All specimen tested were compacted to $N_{\text{design}} = 109$, which corresponds to the heaviest traffic levels used in Iowa.

#### Testing Procedures

Repeated load triaxial tests were conducted using the Nottingham asphalt tester (NAT). The NAT is widely used in Europe for testing asphalt mixtures—most commonly in a repeated load axial configuration. It has recently been modified to apply a confining stress; this is done through a vacuum as shown in Figure 1(8, 9). All tests were conducted at 45 °C.

The NAT repeated load triaxial test procedures are summarized as follows:

1. Place specimen in environmental test chamber 130 minutes prior to test to consistent test temperature.

### Table 5 Aggregate Gradations

<table>
<thead>
<tr>
<th>Sieve Number (mm)</th>
<th>9.5 mm NMS</th>
<th>12.5 mm NMS</th>
<th>19.0 mm NMS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fine</td>
<td>Dense</td>
<td>Coarse</td>
</tr>
<tr>
<td>19.0</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>12.5</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>9.5</td>
<td>95</td>
<td>95</td>
<td>95</td>
</tr>
<tr>
<td>4.75</td>
<td>80</td>
<td>65</td>
<td>55</td>
</tr>
<tr>
<td>2.36</td>
<td>60</td>
<td>47</td>
<td>36</td>
</tr>
<tr>
<td>1.18</td>
<td>45</td>
<td>34</td>
<td>25</td>
</tr>
<tr>
<td>0.600</td>
<td>32</td>
<td>26</td>
<td>17</td>
</tr>
<tr>
<td>0.300</td>
<td>22</td>
<td>19</td>
<td>12</td>
</tr>
<tr>
<td>0.150</td>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>0.075</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>
2. Grease top and base platen with silicon-teflon grease.
3. Place HMA specimen on base, slide rubber membrane over it, and secure with O-ring.
4. Place top platen on specimen, roll membrane up over edge, and secure with O-ring.
5. Place in center of load frame and lower load cell onto the ball seating.
6. Place the LVDTs on the support arms and zero them out.
7. Enter in appropriate test data (e.g. specimen height, name, etc.) and start test.
8. Begin the test after a two minute conditioning—1800 cycles (one second load duration, one second recovery) taking one hour at 300 kPa axial load, 17 kPa confining stress.

The repeated load triaxial test provides realistic loading conditions in that it simulates traffic and confines the HMA laterally. Recent improvements and innovations (such as the NAT equipment) have made it user-friendly, expedient, and easily adaptable into a Superpave based mix design program.

**ANALYSIS OF RESULTS**

The first step of the analysis was to determine the critical transition asphalt content of the compacted HMA mixture based on a visual analysis of the NAT results. To show how this was done, the test results for the three 19 mm NMS crushed aggregate blends is shown in Figure 2. Examining the plot the critical asphalt content of the three mixes was estimated as 6.6 for the coarse, 6.3 for the dense, and 6.9 for the fine-graded mix. Five of the mixes did not become unsound over the range of asphalt contents used in the study. For
each of the 31 mixes that became plastic, the volumetric properties were calculated at the critical point. Whereas McLeod specified VMA at 5 percent air voids and Superpave at 4 percent air voids, the critical VMA was defined at whatever air content the mix became unstable.

### TABLE 6 Comparison of Observed Critical VMA Values with Superpave Requirements

<table>
<thead>
<tr>
<th>Nominal Maximum Aggregate Size</th>
<th>Observed VMA Average Value</th>
<th>Observed VMA Standard Deviation</th>
<th>Minimum Required VMA</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5 mm</td>
<td>13.5</td>
<td>1.5</td>
<td>15</td>
</tr>
<tr>
<td>12.5 mm</td>
<td>12.3</td>
<td>1.1</td>
<td>14</td>
</tr>
<tr>
<td>19 mm</td>
<td>11.2</td>
<td>1.7</td>
<td>13</td>
</tr>
</tbody>
</table>

### VMA & Nominal Maximum Size

The critical VMA values where the mixtures became unsound are plotted in Figure 3. Table 6 compares the average values with Superpave’s minimum VMA requirements. As can be seen, the minimum VMA requirements based on nominal maximum aggregate size appear to fit the data trend reasonably well, however it is seen that the measured values are typically less than the Superpave criteria.

Therefore, from a practical viewpoint, this gives credence to the complaints that the VMA requirements are too restrictive, as is shown in Figure 3, where only three mixes meet the Superpave VMA requirements. Given the difficulties in achieving Superpave’s minimum VMA requirements, it is worth examining how other aggregate properties, e.g., gradation and surface texture, effect critical VMA. This involves using analysis of variance (ANOVA) to identify the significant aggregate-related effects upon critical VMA.

In selecting what factors to include in the ANOVA analysis, it was hypothesized that critical VMA would be determined not only by nominal maximum aggregate size, but also by crushed coarse aggregate content, crushed fine aggregate content, and some indicator of the gradation curve. For the latter, the fineness modulus (ASTM C-136), was selected.

The ANOVA results are presented in Table 7 and show that nominal maximum aggregate size becomes insignificant when the fineness modulus of the aggregate blend is included as a factor. This makes sense as the fineness modulus encodes information about the nominal maximum aggregate size of the blend.

### Other Volumetric Properties

#### Air Voids

In Superpave, the design asphalt content is determined at 4 percent air voids ($N_{design}$). Prior to Superpave, Marshall mix design allowed a range of 3–5% air voids. For the mixes tested, the results suggest a lower limit of 2.6% air voids (S.D. = 0.65) at the critical state.

#### Voids Filled with Asphalt

The mixes studied were compacted to $N_{design} = 109$ gyrations, which, under Superpave (Table 2), corresponds to a traffic level of 3–10 million ESALs. Following Table 2, this limits VFA to 65–75 percent. For the mixes tested, the critical VFA appears to occur at about 78 percent (S.D. = 5.8).

### CONCLUSIONS

The results presented here are of a preliminary nature and based on performance testing equipment that is still in the developmental stage. Work is ongoing to develop an equation relating critical VMA to aggregate properties to refine the Superpave requirements. Based on the results achieved so far the following conclusions are presented:

1. Specifying a minimum VMA requirement for asphalt paving mixtures based on nominal maximum aggregate size may be unrealistic.
2. Fineness modulus (shape of gradation) and crushed coarse and fine aggregate content (surface texture) appear to be much more robust indicators of critical VMA.
3. Current minimum VMA requirements may rule out mixtures that perform satisfactorily as some have suggested.

### FURTHER WORK

The next step in the research is to analyze the results statistically to develop an equation

$$VMA_{crit} = f(FM (Gradation), %\text{ crushed coarse}, %\text{ crushed fine}) + \epsilon.$$  

This equation will then need to be applied to field and laboratory data, for both well and poorly performing mixes for validation.
ACKNOWLEDGEMENTS

This research was sponsored by the project Development Division of the Iowa Department of Transportation and the Iowa Highway Research Board (TR-415). The authors wish to express their gratitude to the Center for Transportation Research and Education (CTRE), the Iowa Department of Transportation (IADOT), and the Asphalt Paving Association of Iowa (APAI) for their support.

REFERENCES

An Application of ITS for Incident Management in Second-Tier Cities: A Fargo, ND Case Study

Shawn Birst and Ayman Smadi

Congestion on urban freeways, which adversely affects the economy, environment, and quality of life, continues to be a major problem in the United States. Minor incidents, such as minor traffic accidents, stalled vehicles, and special events, account for the majority of urban freeway congestion. Due to the problems associated with freeway incidents, many large metropolitan areas have implemented Incident Management Systems (IMS) to alleviate congestion and safety problems associated with incidents. These systems provide motorists with timely and accurate information to avoid incident locations. These systems have been implemented mainly in large urban areas; however, little is known about the possible benefits in smaller urban areas (second-tier cities). This study examined the feasibility of implementing IMS in small/medium size urban areas using a case study of the I-29 corridor in Fargo, ND. The INTEGRATION simulation model was used to estimate the potential benefits of an IMS which employs Advanced Traveler Information Systems (ATIS) and Advanced Traffic Management Systems (ATMS). The case study analysis revealed that the combination of ATIS and ATMS provided the most favorable network benefits under a 20-minute incident. The IMS reduced incident travel times by 13 percent (city arterials), 28 percent (freeways), and 18 percent (overall network); average trip times were reduced by 20 percent (overall network); and average speeds increased by 21 percent (overall network). Key words: incident management, second-tier cities, simulation models.

INTRODUCTION

The purpose of this study is to examine the benefits of using Intelligent Transportation Systems (ITS) for incident management in second-tier cities. ITS incorporates existing and emerging technologies in areas including telecommunications, computer sensing, and electronics that provide real-time transportation information (1). These ITS technologies provide information to manage transportation, resulting in increased efficiency and safety of the surface transportation system and dramatically improving the travel options and experiences of the motorists (2).

Background

In 1992, the Texas Transportation Institute (TTI) estimated that the 50 largest urbanized areas in the United States lost over $48 billion due to congestion, a 9 percent increase from 1991 (3). Incidents, such as traffic accidents, stalled vehicles, construction and maintenance, special events, and adverse weather conditions, account for nearly 60 percent of all traffic congestion in the United States (4). This congestion has the following negative implications:

· Lost productivity and less personal time with family, hobbies, etc.
· Increased pollution levels and wasted fuel consumption from slower vehicles and stop-and-go conditions.
· Safety issues related to increases in crashes due to driver frustration, aggressive driving, risky maneuvers, etc.

Incident Management

Incident management is a coordinated and planned program that controls, guides, and warns the motorists of traffic problems in order to optimize the safe and efficient movement of people and goods. Incident management involves the cooperation of multiple agencies, such as government officials, police, highway patrol, fire and rescue services, emergency medical services, hazardous material crews, and towing services, to facilitate non-recurring congestion problems on freeway systems (5). Incident management systems attempt to reduce the detection, response, and clearance times of incidents, thereby reducing the delays experienced by motorists affected by the incident. Incident management also provides traveler information to warn motorists that an incident is ahead and to take an alternative route if one is available. The diverted traffic will reduce the demand on the road segment where the incident occurred, causing less delay to the motorist on this segment.

ITS in Incident Management Systems

ITS technologies were developed to increase the efficiency and safety of the transportation system. Incident management systems primarily incorporate two components: Advanced Traffic Management Systems (ATMS) and Advanced Traveler Information Systems (ATIS). The ultimate goal of ATMS is to provide traffic control strategies that are capable of adjusting to the existing traffic demands through surveillance technologies, data pro-
cessing, and communications (5). ATMS also can detect incidents using inductive loop detectors or video-image detectors. Based on the incidents’ characteristics, ATMS can select a proper response, such as diverting upstream traffic away from the incident and adjusting traffic signal timing plans on alternative routes. In addition, the incident information can be provided to motorists using ATIS so they can adjust their travel routes.

ATIS collects and distributes traffic information for both “pre-trip” and “en-route” travelers (5). Several forms of communication technologies can be used including variable message signs (VMS), highway advisory radios (HAR), local radio and television broadcasts, websites, and kiosks. ATIS provides current and near future traffic conditions, which allow motorists to change their plans or routes to minimize their travel times.

**Traffic Simulation Models**

Simulation models, primarily those capable of analyzing both arterial and freeway networks, provide effective tools for analyzing the performance of existing and proposed corridors. These models offer great flexibility for evaluating various infrastructure and operational alternatives under different conditions—all without altering existing facilities or disrupting traffic. Several freeway simulation studies have been performed to evaluate incident management protocols and strategies, including:

- Santa Monica, CA (I-10),
- Houston, TX (US-59),
- Arlington County, VA (I-66),
- Fort Worth, TX (I-35W), and
- Orlando, FL (I-4).

The freeway simulation studies varied in case study location, incident occurrence and duration, and optimization strategies. However, each study found that implementing various forms of ITS in incident management systems benefitted the transportation network. The studies evaluated several components of ATMS and ATIS, and estimated several quantitative system improvements, such as (6, 7, 8, 9):

- Freeway and arterial speed increases of 15.3 and 10.1 percent, respectfully (Santa Monica, CA);
- Total network delay time ranging from 22.9 to 27.4 percent for the small network and 14 to 16 percent for the large network (Houston, TX);
- Diversion efficiencies as high as 298 percent for the small network and 79 percent for the large network (Houston, TX);
- Total system delay reductions of 63 percent (Arlington County, VA), and
- Total network delay savings of 1 to 11 percent, freeway delay savings of 300 to 500 percent (Fort Worth, TX).

**IMPORTANCE OF RESEARCH**

The case studies in the previous section analyzed freeway corridors in several large urban areas across the United States (note: no applications in smaller urban areas). Traffic levels, trip characteristics, and the availability of alternative routes are distinctly different in smaller urban areas; however, it is expected that similar benefits may be realized in small-to-medium size metropolitan areas. These benefits must be quantified and accurately examined to support ITS deployment decision in these areas.

**OVERVIEW OF THE METHODOLOGY**

Developing a complete and functional incident management system requires extensive planning, communication, and coordination among several entities to determine proper protocols for possible incident scenarios throughout the implementation area. Further, deploying ITS components generally requires significant up-front expenditures that must be carefully analyzed to justify their implementation. Therefore, this study develops a methodology to evaluate potential user benefits that can be used in a benefit-cost analysis of IMS utilizing ITS technologies (Figure 1).

**FIGURE 1 Proposed methodology flowchart**

The evaluation is based on comparing key Measures of Effectiveness (MOE), such as travel time, trip time, and speed. The MOEs are estimated using traffic simulation models for several cases that represent the network under current conditions and with ITS deployment. The base cases represent the case study’s current or existing conditions (road geometry, traffic volumes, turning movements, and signal timing plans). The ITS cases use the base case network and traffic levels, but employ various ATIS and ATMS elements to encourage drivers to take alternative routes and to implement adequate signal timing plans adequate for the new distribution of traffic.

The INTEGRATION simulation model (developed by Dr. M. Van Aerde) was selected for this case study for a number of reasons, including the following:

- It is capable of modeling ITS components, such as ATIS and ATMS,
- It provides several types of quantitative output, such as travel time, trip time, speed, vehicle emissions, and fuel consumption, and
- It provides graphical output, which allows the user to “fine-tune” or adjust the model to reflect actual traffic conditions and compare different scenarios.
CASE STUDY

Fargo is the largest city of the four-city Fargo-Moorhead (F-M) metropolitan area, which had a population of approximately 166,000 in 1996. Numerous freeway incidents occur within the F-M metropolitan area, as a result of special events, traffic accidents, inclement weather, and material spills. A majority of the incidents that occur in the F-M area are traffic accidents. An average of 108 crashes per year occurred on the freeway system within the F-M metropolitan area between 1994 and 1996. This information may be used for examining potential benefits of IMS.

The analysis was conducted on a portion of Interstate 29 (I-29) and Interstate 94 (I-94), which are predominantly used for local traffic. Four of the area’s heaviest traveled arterials are included in the corridor, which will provide motorists with diversion routes during incident occurrence. An incident occurrence will be simulated on a northbound segment of I-29 (Figure 2).

Evaluation Scenarios

The simulation analysis can be grouped into two cases: 1) base cases and 2) ITS enhanced cases. Comparing the simulation output of the two groups will determine whether using ITS technologies can benefit motorists during incidents in second-tier cities. The scenarios that were used in this study are as follows:

- Scenario 1: Base Case (without incident occurrence, ATIS, or ATMS),
- Scenario 2: Incident Base Case (without ATIS or ATMS),
- Scenario 3: ATIS Case, and
- Scenario 4: ATIS and ATMS Case.

The simulation period for all cases has a duration of 1 hour and 40 minutes. The following list describes the simulation timeline:

- The network is “loaded” with off-peak traffic demands (5 min).
- Traffic demands increase to simulate AM peak hour traffic conditions (60 min). Traffic demands will return to off-peak conditions for the remainder of the simulation.
- A one-lane-blocking incident occurs on a northbound segment of I-29 at the beginning of AM peak hour traffic demands (20 min).
- Traffic returns to normal conditions (recovery period). The recovery period was determined by inspecting the on-screen graphics of the simulation model (63 min for the incident base case). Although the recovery period will be shortened by implementing ITS strategies, for comparative purposes, the simulation length for all of the scenarios will remain the same.
- All generated vehicles are allowed to reach their final destination (12 min).

Base Cases

The base cases utilize 1996 traffic volumes, road geometry, and traffic control. Two base cases will be simulated for this case study. The first base case, Scenario 1, will simulate current traffic conditions within the corridor during the AM peak hour without
an incident and is primarily used for validation. The incident base case, Scenario 2, simulates current conditions with an incident, but without providing traveler information or traffic management. Scenario 2 will serve as a baseline for comparing the effectiveness of the other scenarios (i.e., the effectiveness of the ITS enhancements).

### ITS Enhanced Cases

The ITS enhanced cases will examine the operational benefits of ATIS and ATMS deployment for the base network with the same traffic conditions as the incident base case. Traveler information regarding the incident will be provided to freeway motorists entering the incident location and will be in the form of current link travel times for all possible routes. Link travel time information, via VMS devices, will provide the vehicles with a minimum path based on the current network traffic conditions for 180 seconds. Freeway vehicles only will divert to alternative routes if it is more beneficial to do so.

ATMS will be utilized to accommodate the diverted traffic from the freeway to the city arterials, and to reduce the impact of freeway incidents on both the freeway system and the city street system. Simply diverting traffic off the freeway will only benefit the freeway while creating a large burden to the city street network. Therefore, ATMS strategies will incorporate optimized signal timing plans for the city arterials similar to Split Cycle Offset Optimization Techniques (SCOOT). This process will include optimizing cycle lengths, phase splits, and offsets along diversion routes, which will eventually bring the diverted traffic back to the freeway. The cycle lengths, phase splits, and offsets will be optimized every 5 minutes with the cycle lengths ranging from 60 seconds to 120 seconds.

### Case Study Results

Three Measures of Effectiveness (MOE) values were compiled from the simulation model and included 1) total travel time for the freeways, city arterials, and total network; 2) average trip time for the O-D demands; and 3) average speed of the total network. The incident base case (Scenario 2) output served as a baseline for comparing the other scenarios to determine the effectiveness of the ITS cases (Table 1).

### Travel Time

Travel time refers to the time it takes a vehicle to traverse a given link. The total network travel time refers to the summation of all link travel times throughout the corridor. The total network travel times were broken down into total freeway and city arterial travel times to determine how each facility type reacts to the given scenarios.

Travel time generally decreased as the amount of ITS deployment increased. When compared to the incident base case (Scenario 2), both Scenarios 3 and 4 provided freeway travel time reductions of 26 and 28 percent, respectively. Since Scenario 4 also incorporated ATMS strategies, city arterial travel times were reduced by 13 percent. Scenario 4 provided greater network benefits regarding travel time than the base case without an incident (Scenario 1) since the existing traffic signal system is not operating at the optimal timing plans.
Trip Time

Trip time is defined as the time it takes each vehicle to complete its trip. The average trip time values were obtained by summing the network O-D trip times by the total number of vehicles that were generated by the model. When compared to the incident base case (Scenario 2), the case that only provided traveler information (Scenario 3) reduced trip times by 10 percent. Providing traveler information along with adjusting the signal timing plans to reflect the current traffic demands (Scenario 4) enhanced the corridor’s performance by reducing trip times by 20 percent. Similar to the travel time output, the ATIS/ATMS case outperformed the base case (Scenario 1).

Travel Speed

Average travel speeds for every link were calculated by the simulation model. As experienced in the two previous MOE comparisons, the ITS cases provided higher average speeds. The ATIS scenario (Scenarios 3) provided 8 percent higher average speeds, while the ATIS/ATMS strategies (Scenarios 4) provided an increase of 21 percent. The ATIS/ATMS case provided higher average speeds than those encountered by the base case. The higher speeds can be attributed to the reduction of traffic congestion, which was caused by stop-and-go traffic under incident conditions and by poor traffic signal coordination.

CONCLUSIONS

The operational effectiveness of using ITS in incident management was evaluated for several scenarios involving different levels of ITS implementation. The evaluation was based on comparisons of three MOEs (which included travel times, trip time, and speed) for each of the analysis cases. The ITS enhanced cases provided significant user benefits to both the freeways and city arterials, therefore, enhancing the overall operational efficiency of the transportation system. When compared to the incident base case, the combination of ATIS and ATMS provided reductions in network travel time and network trip time of 18 and 20 percent, while increasing the average network speed by 21 percent. The analysis also revealed that implementing ATIS/ATMS strategies during incidents improved traffic conditions to such an extent that it was more effective than the base case without an incident. To achieve significant benefits for the case study corridor, the ITS cases did not need to divert a lot of the freeway traffic to the arterials. In fact, the best ITS case (Scenario 4) only diverted eight percent of the traffic that would have used the incident location to the arterials.

An important benefit of incident management relates to safety improvements. A study determined that 20 percent of all crashes occur upstream from an incident (12). The traffic levels of the ATIS/ATMS case returned to normal conditions 13 minutes earlier than the incident base case, thus reducing the potential of secondary incidents.

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REFERENCES

Public Safety Operations and Traffic Management - A Force for the Future

ROBERT B. FRANKLIN, JR.

There are a number of converging forces that will allow an improved and even more efficient interconnect between public safety operations and those management systems used by the traffic management community to address incident management and congestion mitigation. In the rural environment, increasing the efficiency of this interconnect will also materially assist in the deployment of Automatic Crash Notification (ACN) and in the provision of more timely traveler information. Overall, fleet efficiencies and economy of operation, incident zone safety and improvements in the time required to restore traffic, are all areas that benefit from this close interconnect. From more rapid detection, verification and response to incidents, to minimization of the time required to transport the injured to a medical facility, interconnection promises great benefits with minimal incremental expense required to link the systems. While, to date, there has been resistance to this interconnection of public safety operations and traffic management, proliferation of the cell phone has done much to bring the systems together. From the traffic engineer’s perspective, the cell phone has become one of the fastest incident notification devices available. From the Public Safety perspective, emergency call takers are facing a daily increase in the number of emergency notification calls transmitted by cell phone. Over 90% of these are traffic related; the cell phone has become the enabling technology for ACN. Coupled with this is the imminent emergence of a number of technologies that will provide the ability to accurately, rapidly, and easily locate these wireless devices. Thus, the cell phone is rapidly becoming the driving force to bring the two functions together. This paper highlights some of the methods and levels of interconnect; discusses the pluses and minuses against a backdrop of real-world examples and lays out a potential path forward. Key words: Automatic Crash Notification, cell phone, interconnect, public safety, wireless geolocation.

TRAFFIC MANAGEMENT TODAY

There is a movement afoot - one that is going to benefit traveler, traffic engineer, and first responder alike. The traditional walls that have separated the traffic management and public safety communities are starting to crumble, and we will all be the benefactors.

To this point in time, development of the operating systems supporting these functions has been conducted in almost a totally independent mode. The same can be said for their mode of operation; exchange of real-time information was at best scanty. A major causative factor is that the various Departments of Transportation (DOT) have historically focused on managing the pavement infrastructure, not operating it. There has been almost a total focus given on “managing” the paving, widening and resurfacing projects; “operating” was not in the lexicon. These have been the “vote-getters” that have competed so successfully over the years for tight budgetary dollars. When it came to addressing the flow of traffic most of the action took the place of passive design measures. This has been particularly true with respect to the freeway and major arterials. With the advent of the Intelligent Transportation System (ITS) movement, this has begun to change. Now the DOTs are beginning to actively control the flow of traffic and are implementing region-wide incident management and congestion mitigation plans. The Advanced Transportation Management Systems (ATMS) currently being deployed in most major metropolitan areas are generally based on a solid ability to rapidly and accurately detect incidents and congestion. That this information could be of value to other governmental agencies has not been, nor is it being contested now. Unfortunately the cost of deploying a detection network with its supporting communication system has caused most ATMS deployments to be done in an incremental or phased approach. Thus even in some of the larger ATMS deployments such as Atlanta, Houston, or Seattle, a significant part of the freeway or interstate system is not yet included; coverage of the major arterials is even more patchy and coverage on the rural system essentially does not exist. This means that the ability to provide information about traffic conditions is still sketchy or non-existent for significant portions of the infrastructure.

TECHNOLOGY SHIFT

While no one in public safety contests the potential value of real-time traffic conditions and road report information to their dispatch and operations systems, the current maturity or lack of coverage of the deployed ATMS does not force the issue; that is about to change. Since the introduction of the Enhanced 911 system or E911, all wireline 911 calls received at the Public Safety Answering Point (PSAP) are accompanied by Automatic Number Identification/Automatic Location Identification (ANI/ALI) data. The same is not currently true with wireless or cellular phones (only number identification is available). To correct this, the Federal Communication Commission (FCC) has mandated that by October 2001 the cellular industry must be able to determine and transmit cellular phone location with a minimum of 125m accuracy when a cellular phone makes an E911 call. The wireless industry has jumped on the bandwagon and soon will be able to not only meet, but also far exceed this mandate. Current projections are that the location accuracy will be in the 20m range plus the direction of travel and speed may also be available. For some technical solutions to this FCC mandate, the estimates concerning the accuracy for direction of travel are plus or minus 5 degrees and for speed, plus or minus 3 MPH. Given that there are almost 80 million wireless phones in operation in the United
States it can be readily extrapolated that there are essentially 80 million new traffic detectors available to the traffic management community for incident management and congestion mitigation purposes. This is not to say that there will be an overnight quantum leap in capabilities. Additional algorithm work will be required to adapt the location, velocity, and direction input from wireless phones, but the potential is boundless. This potential for using the location data is the first major impact of the cellular phone on the current way of doing business for both the traffic engineering and public safety communities.

Another major impact is the sheer volume of emergency calls now being received from cellular phones by the 911 system. In 1998, the Cellular Telephone Industry Association estimated that there were approximately 98,000 calls made on wireless phones to a 911 system every day. Other statistics indicate that 90% of these are traffic related and estimates indicate that from 25% to 50% of the number of callers do not know their location. Prior to the cellular phone’s explosion, most traffic related incidents were reported by traffic enforcement officers or by callers using wireline devices. There were a few calls with more accurate information about the details of the incident. Now there are instances of as many as 75-100 calls being received about the same traffic-related incident. This has placed a tremendous burden on the public safety call-takers. There is no thought that establishment of an interconnect between traffic management and public safety systems will mean that a call will not be answered. But by tying the two systems together, corroborating information from the traffic detection network can be made available to the call-takers, thus allowing an informed dispatch decision to be made much more quickly and efficiently. Where this has happened there has been a marked increase in the effectiveness of the First Responders and a corresponding decrease in the overall system-wide 911-response time.

The third area of impact has been the availability of the phone to initiate emergency calls. This availability is now taking a major leap forward because of the cell-phone’s ability to support Automatic Crash Notification (ACN). Current estimates are that Telematic/Mayday Devices will be deployed in up to 4 million vehicles in the next three years; General Motors alone estimates that they will have one million subscribers for their OnStar™ system by 2001. Given this type of growth projection, it is easy to forecast the cost of implementation going down. This cost decrease will cause the number of deployed devices to go up and thus, will produce faster incremental growth in the number of geo-located wireless devices available for the information stream. While there are no apparent links to the traffic management system, every new wireless device mounted in or transported by a vehicle becomes another potential source of information to the traffic management community.

**DO WE HAVE A WAY AHEAD?**

This is not to say that everything is rosy. There are fundamental philosophical design issues that may impact the viability of cellular location as a detection methodology. Remembering that the FCC ruling is focused on providing information to the public safety function, the ability to use a wireless device as a detection device in an ATMS has not been a consideration.

There are two basic technical solutions available to meet the FCC requirement - handset-based or network-based. The handset-based solution relies on Differential Global Positioning System (DGPS) technology, and while it will provide very accurate location data (potentially down to a car-length when the Federal Railroad Administration completes the deployment of the National Differential GPS network in 2003, location data is probably only going to be available when an emergency call is made.

On the other hand, network-based solutions will be of benefit as they are going to produce location information on a fairly regular (generally 15-minute) basis as a function of tracking the device to facilitate call routing. This polling feature is what allows cellular roaming; the cellular system needs to know which cell a phone is in at any point in time. Given the projected density of the cellular phone population, particularly in the urban and near-urban areas, there should be a sufficient number of devices providing location based reports to formulate a fairly accurate depiction of traffic conditions being experienced by vehicle mounted or transported cellular phones.

Using the network solution, there is not a requirement to track a phone over the 15-minute period to generate direction and speed information. Rather, all three of the attributes - position, direction and speed - are gathered each time that the device is polled. This is the area where algorithms must be prepared, for example, to sort out pedestrian traffic from vehicle traffic in the downtown areas and, most important, eliminate all privacy information from the process.

What is not yet in the works is the ability to track a phone to generate Origin/Destination (OD) information. This is not a requirement to satisfy the FCC mandate and would entail considerably more software development.

There are additional problems. There are currently 5,500 Public Safety Answering Points (PSAP) that service 911 calls. There is going to have to be a fairly serious effort made to accommodate the information coming directly from the wireless systems and coming from the traffic management system.

There are a number of joint public safety/traffic management operation centers where the two functions have no more connectivity than being located in the same facility. Interconnection can achieve “virtual” co-location despite geographic separation.

Some people feel that placing both functions in the same location will achieve the majority of the potential benefits. I would argue that this is not the case, for in these instances most of the coordination is manual as well as spasmodic and ad hoc. This does not produce the same level of benefit, as does the basic level of interconnection. Interconnection can achieve “virtual” co-location despite geographic separation.

**A COMMON LEXICON**

For purposes of this discussion we will categorize the three transfer products in ascending order of technical complexity, as voice, video and data while the interconnect will generally be defined as falling into one of four levels. Remember that the issue is interconnection, not co-location.

Level One is the bare minimum level of interconnect and may be no more complex than an off-hook or ring-down circuit connecting the traffic management center with the PSAP. This is the “voice-only” level and is quickly and easily established with no impact on the current computer system at either end since it is totally “man-in-loop”. There are no other requirements beyond adjusting Standard Operating Procedures within the two systems to ensure
that calls are made in a timely fashion. Implementation of Level One will begin to develop the close working relationship that can and should be fostered between these two Public Service systems. It will provide more rapid corroborating information to the public safety call-taker and will allow the traffic managers to respond to incidents far more rapidly.

But using the axiom that “a picture is worth a thousand words,” Level Two involves the ability to pass control of Closed Circuit TV (CCTV) camera views between the two systems. Almost every ATM systems has a large number of cameras installed as part of the deployment. These are used by traffic managers to verify indications generated by the installed detection system. Similarly, more and more of the larger metropolitan public safety agencies are installing security cameras in the Central Business Districts and high population density areas such as shopping centers, stadiums and airports. By being able to pass control of these cameras to the other partner in an interconnected system, the area of visual coverage and accuracy of response is markedly improved. One major concern is who has access to these tools and who controls them during an incident. Public Safety operations personnel may want to have the ability to zoom in on an accident scene to assist the on-scene incident commander while the traffic managers may want to zoom out and pan the camera in an effort to implement the proper traffic management response. Both requirements are valid and will take some close cooperation to work the issue. Beyond the control issues are the cost of procurement and maintenance. With an integrated management plan focusing on a team approach, these issues can be worked out to the betterment of all.

Level Three is the exchange of data from screen to screen. If you placed the typical public safety call-taker’s screen next to a traffic incident manager’s screen, you would observe a significant commonality of data elements for all traffic-related incidents. Being able to simultaneously fill in data fields on both systems with a “single keystroke” will produce significant savings in the time necessary to arrive at implementation decisions for both systems. For the most part this is not a technically challenging problem. The only area of concern is to be sure that the requisite “firewalls” are in place to protect the privacy of information that is an integral part of the public safety operation but not required by the traffic management community. In actuality, the hard part will be to get the appropriate intergovernmental agreements in place to allow this exchange of data. There is an understandable reluctance on the part of public safety to have any information contained within their system exposed to other agencies without the same safeguards.

The fourth and most comprehensive level is the automatic cross population of the two system’s databases. There are a number of technical concerns to address, but none are insurmountable given today’s technologies. The biggest concern stems from the geo-location system. As can be readily understood, the public safety community relies on street addresses for geo-location. This reliance is caused by the fact that the majority of the initiating action for their operations originate based on a call from an individual citizen who currently has no way of determining his location in terms of Latitude and Longitude. The current exceptions to this rule are the calls originating out of the ACN systems since they are based on DGPS. When the geo-location mandate is fully implemented, the public safety Computer Aided Dispatch (CAD) systems are going to have to be able to do the geo-location cross-reference from the Latitude/Longitude system to street addresses since the incoming calls will carry a mixture of geo-referencing methodologies. This is not a trivial task.

It is envisioned that the majority of these systems will be GIS based, which raises the additional concern about the spatial errors that are induced when transferring location data from one GIS application to another. Errors of as much as 300 meters are not uncommon when transferring from one GIS application program to another. Generally this is not as much of a problem in the rural environment, but can raise significant hurdles to interconnection of urban systems. Currently, the only way to overcome this issue, if it exists for a particular locale, is for all of the agencies contemplating joining an interconnected system to agree to a common GIS platform. Although this is not the most satisfactory solution, it is one that may be required given the current level of variation between platforms.

This variation is probably the single most significant technical stumbling block when trying to interface with existing public safety CAD systems, which tend to be very proprietary in nature. This problem may diminish in the future as new or replacement CAD systems are installed.

The other major problem, which is more emotional than technical, is the issue of jurisdictional autonomy. When information is to be shared or exchanged between two or more governmental agencies, there is the understandable worry about increasing exposure to increased liability. Since many of the larger ATM systems are run by a state-level agency and the public safety CAD by a local agency, there is an understandable reluctance to interconnect.

Suffice to say that this can be eliminated technically with a high degree of assurance that information can be protected using a combination of procedures and software privileging. The hardest institutions to convince will be law enforcement, but if everyone has a clear understanding of the design, protections and, most importantly, the benefits of establishing an interconnect, it can be done.

SUMMARY

Where is this whole movement going? There are more and more examples of successful interconnects. The Cellular Telecommunications Industry Association, the Association of Public Safety Communications Officials (APCO), and National Emergency Number Association (NENA) have been fairly active in support of the development of the wireless location program, but have backed off a bit in the face of the industry’s lethargic response, and more importantly, because of the FCC’s sluggish push toward implementation. This could have been a major stumbling block to the quick implementation of the wireless implementation technology. The whole movement may have seen a revitalization because on 15 September, 1999 the FCC adopted revisions, which require 100 percent compliance NLT 31 December 2004, to its wireless E911 rules affecting the hand-set based solutions.
These revisions have the potential of rapidly accelerating the geolocation of wireless phones but it may well be in a direction that may not produce the majority of the benefits to the ITS community. Given the cost of network solutions as opposed to the steadily reducing cost of the hand-set based solutions, the wireless industry does not see the value of the business case in a public safety only application. The revenue stream is legislated and not subject to providing additional returns on their investment.

Therefore, if the ITS community is to benefit from this emerging technology, we must make our needs known to the wireless industry now before the commitment to the hand-set technology is too far advanced. We may be facing the VHS-BETA marketing decision all over again. At the time BETA was the far better solution but it was relatively expensive so the industry settled on VHS and the rest is history.

When you consider the cost of installing and maintaining a detection network throughout a region as compared to the probable cost of acquiring the data from the wireless industry, or if you consider the fact that the deployment will be almost overnight, the value of being able to avail ourselves of this new technology should be readily apparent. ITS America is watching this program very closely and is sponsoring several efforts to try to forge the bond with public safety as quickly as possible by working with the national public safety associations. This top-down push will go a long way toward accelerating the process but just as much value can be achieved at the local level. That is our job.
The Basics First: Baltimore Regional Information Exchange System (IES)

JOERG "NU" ROSENBOHM, EILEEN SINGLETON, AND BOBBIE SHARMA

Many transportation planners and facility operators across the country face the same problem – the inability to access information about real-time conditions, planned events, and historical data in neighboring jurisdictions or other agencies. This common problem is being addressed in a recent project initiated by the Baltimore Metropolitan Council (BMC, staff to Baltimore’s Metropolitan Planning Organization) and participating member agencies. In order for agencies to share relevant recurring and non-recurring incident information and other transportation-related information, procedures for sharing data and information must first be established. Initially, these procedures will be based on using existing forms of communication. A plan is being developed to gradually add electronic sophistication as time and budget allow. The first step in developing information sharing procedures was to conduct a survey to identify available and desired information, and data needs (electronic and non-electronic descriptions of equipment, procedures and events [recurring and non-recurring]). The survey results have been used to match data needs with existing data exchange methodologies. In the near term, these methodologies include telephone, fax, electronic mail, and scheduled mailings, while mid- to long-term approaches include more automated systems (i.e., integrated computer exchange systems). Determining methodologies that allow tying these future high-tech systems into the architecture defined within the Maryland Statewide Information Exchange System (IES) is being developed to address this need. The primary purposes of this project are to identify information that is desired by, and available from, participating agencies, and to identify appropriate methodologies to share information among participating agencies. This project involves the development and initial deployment of the Baltimore Regional Information Exchange System (IES). In order for agencies to share relevant recurring and non-recurring incident information and other transportation-related information, procedures for sharing data and information must first be established. Initially, these procedures will be based on using existing forms of communication. As part of this project, an attempt has been made to include steps and procedures to allow the gradual addition of electronic sophistication as time and budget allow.

This project seeks participation of all regional stakeholders, including those that may not typically be included, but who could benefit from the system, such as emergency responders, school bus operators, and mass transit and para-transit operators. The project may be somewhat unique in that it starts as a “low-tech” solution by establishing information-sharing procedures using existing technologies. The long-term goal of the project is to develop an electronic information clearinghouse that contains transportation information or information that will assist transportation professionals including planners and facility operators.

INTRODUCTION

Many transportation planners and facility operators across the country face the same problem – the inability to access information about real-time conditions, planned events, and historical data in neighboring jurisdictions or other agencies. The problem includes not only the actual lack of data, but also the lack of procedures to exchange available data (i.e., contacts within neighboring agencies and jurisdictions, the format of the data, and knowledge of the types of data that may be available from other agencies). Often, there is also a lack of knowledge of data and data formats among the different departments within an agency or jurisdiction.

The resolution of this issue arose as a high priority during work on Phase I of the Metropolitan Baltimore Intelligent Transportation Systems (ITS) Early Deployment Plan (EDP). This common problem is being addressed in a project initiated in Phase II of the EDP by the Baltimore Metropolitan Council (BMC, staff to Baltimore’s Metropolitan Planning Organization) and participating member agencies. This project involves the development

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PURPOSE / PROJECT DESCRIPTION

Since 1996, the Baltimore region has been working on the ITS EDP effort made possible through a grant from the Federal Highway Administration. BMC has been the lead agency, and Transportation Corridor Consultants (TCC) has provided consultant support. There has also been participation from:

- transportation planning and public works departments of the local jurisdictions in the region which includes Baltimore City and Anne Arundel, Baltimore, Carroll, Harford, and Howard Counties
- Maryland Department of Transportation and its modal administrations (State Highway Administration, Maryland Transportation Authority, Mass Transit Administration, Maryland Aviation Administration, Maryland Port Administration, Motor Vehicle Administration)
- Federal Highway Administration
- Maryland State Police
- local emergency responders

Phase I of the EDP revealed that there is a great need in the Baltimore region for the establishment of procedures to enable agencies to share real-time, non real-time and historic transportation-related information. In Phase II, the Baltimore Regional IES is being developed to address this need. The primary purposes of this project are to identify information that is desired by, and available from, participating agencies, and to identify appropriate methodologies to share information among participating agencies. These methodologies will include low-tech meth-
ods in the near-term and high-tech methods in the mid- to long-term. Additionally, steps will be provided to guide agencies to connect to other advanced systems such as the new Maryland statewide CHART II system and the I-95 Corridor Coalition Information Exchange Network (IEN).

This project will focus resources on plans and projects that can be implemented in the near-term, while laying a strong foundation to support future projects. TCC’s involvement with the development of the CHART II architecture as well as the IEN will help to ensure consistency with the regional ITS architecture defined in the EDP and with other proposed and existing statewide systems. The knowledge of these and other projects will benefit the Baltimore metropolitan area agencies by applying previous experiences and preventing similar mistakes.

The Baltimore Regional IES will build on what is currently available, in terms of information and communication systems, while preparing the agencies for more sophisticated systems in the future. In terms of this project, information is defined as electronic and non-electronic descriptions of equipment, procedures, and events (recurring and non-recurring).

The project began with the determination of the currently available information and data that each participating agency and jurisdiction could provide. Additionally, the project sought to determine the desired data and information that an agency or jurisdiction (“entity”) would like to receive from other entities. In terms of communications, the existing conditions of both communications media and existing procedures also needed to be determined. At the onset of the project, a decision was made to develop and distribute a sophisticated survey rather than conducting interviews. It was thought that by having participants complete the survey, they would learn more about what information exists and is needed by the entity. Each participating jurisdiction or agency selected one person to oversee completion of the survey for their entity. The return rate was over 90% indicating the strong support of the participants and their understanding that a data exchange system can only work with sufficient input information.

The major project tasks include: survey development and completion, survey analysis, implementation plan development, implementation, and evaluation. The first two major tasks have been completed, and the implementation plan development is currently underway. The tasks are described in detail in the following paragraphs.

**Survey Development and Completion**

The first step in this project included the preparation of a comprehensive survey that inquired about the anticipated information needs and information provisions of each participating agency. The survey was developed with the help of the project’s consultant team and the participating jurisdictions and agencies. In order to facilitate a high return and due to the lengthy format, the survey was structured in sections that could be answered by different departments and agencies within a jurisdiction. Completed surveys were returned to the BMC, checked for completeness, and forwarded to the consultant, who analyzed the responses.

The following information was collected for each jurisdiction/agency and department:

- Types of information available (traffic data such as volume/occupancy/speed/classification, upcoming events, construction activities, incidents, messages from variable message signs and highway advisory radio, closed-circuit television images, maintenance activities, etc.)
- Current information sharing practices (procedures that are in place to contact other agencies about the types of information as well as frequency of data exchange practices)
- Communication systems/technologies available (to implement the procedures)

This part of the inventory was used to establish the basis for the development of the Baltimore Regional IES. Naturally, the communications systems and technologies available could and should not be limited to the currently utilized means. Thus, private entities, such as Internet Service Providers (ISPs) and communications service providers (i.e., Bell Atlantic), have been considered in the analysis/inventory because these entities could provide missing links to enable a more complete Information Exchange System.

In addition to the information and data that could be provided by each agency, each entity was also asked to provide information for each of the following:

- which roads are operated by which agencies,
- locations of transportation facilities (such as traffic operations centers, toll facilities, and airports),
- locations of police barracks,
- locations of public works maintenance facilities, and
- relevant information.

The map is being prepared using geographic information system software. When the map is completed, the electronic map file will be distributed to all participants.

**Survey Analysis**

The results of the survey responses were reviewed for completeness and compiled into a comprehensive report. This report was structured to include an overview section and the general findings, as well as participant-tailored sections. These participant-relevant sections included listings of the existing conditions, equipment (both field and center), and data exchange procedures. Additionally, an attempt was made to identify desired exchanges with respect to the agency/jurisdiction that would provide the desired data, the types of desired data, and the update frequencies.

While the survey responses did indicate the data types, the formats of those data types, and the data type update frequencies, they did not match these items, i.e., indicate which data type is updated at what frequency and then stored in what format. As a result, several assumptions were made about formats and update frequencies for some data types. Additionally, many of the survey responses did not indicate the agencies, jurisdictions, and departments from which a particular entity would like to receive data. Therefore, it was assumed that an entity would like to receive data from its neighboring agencies as well as from state-level agencies that could provide relevant data.

The existing conditions were exhibited in a large matrix that allowed viewing the currently existing data exchange procedures and the data types between entities.

The survey analysis report was presented to all participants asking for a detailed review and the provision of comments that would verify the exhibited contents and especially the assumptions made.
Parallel to this request, the participants were informed about the need to match the data formats and update frequencies with the data types without which the data exchange procedures could not be developed effectively. Comments were incorporated into the final Survey Analysis Report.

Implementation Plan Development

Based on the survey analysis results, the project team developed a draft Implementation Plan that addressed issues such as communications requirements, equipment needs, and estimated operations and maintenance (O&M) costs. The Implementation Plan also repeated several of the assumptions already stated in the survey analysis to emphasize that the verification of those items are of high importance.

Since this project attempts to provide an introduction of ITS and information sharing protocols to the participating entities, the implementation plan provided two main aspects:

1. Steps and procedures to implement an initial IES and
2. Steps and procedures to advance to an expanded IES.

The initial IES will be based on low-tech communication devices such as fax, phone, scheduled mailings and e-mail. For the purpose of data exchanges, a set of spreadsheet-software based forms have been developed that allow each entity to decide what its preferred communications mode is for each data type (i.e., fax, voice, mailing or e-mail attachment). The forms have been kept simple so only the most important information is transmitted; however, the forms can be modified by users to suit their needs.

For each communications mode, communications requirements, equipment needs, and estimated O&M costs have been provided.

The establishment of the communications requirements included the determination of ‘lowest common denominator’ to establish a common basis for data exchange. For each data type that is proposed to be exchanged between entities, a communications media was proposed, i.e., incident information should be transmitted via phone, construction schedules should be transmitted via e-mail, fax, or mailings.

The equipment needs for each agency were based on the inventory and the communications needs previously identified. Since this project is designed to introduce an initial information sharing system, the emphasis was on defining the requirements for a low-cost, low-tech near-term implementation. Where the appropriate communication equipment for this near-term implementation was not available, a detailed list of the information sharing equipment needed for implementation was developed. The list contained equipment type, function, example make/model and the associated estimated cost. Additionally, an outline of the equipment needs and a description of the anticipated steps to achieve a smooth transition to middle- to long-term high-tech technologies was included.

Operation and maintenance requirements have been identified and annual cost estimates were provided to inform the agencies about the ongoing O&M costs of the system.

The expanded IES will be based on an automated system, which may also use the above-mentioned forms. Since it was a declared goal to determine the feasibility of connecting the IES with the Maryland CHART II system (currently under development), the expanded IES implementation aspects were comprised of investigating and estimating costs for three different scenarios:

1. CHART II workstation limited to viewing and querying features
2. CHART II server and workstation to allow viewing and querying as well as providing field device data to other interested parties.

This would involve development of system interface software.

3. A separate system.

The third alternative was determined to be cost prohibitive. Option 1 could be used for agencies that do not operate and maintain field equipment, while Option 2 should be used for all other entities. Further implementation recommendations could not be made because the CHART II system development is not at the point to provide detailed connection requirements. For example, while CHART II will allow local jurisdictions to access state data, it is not yet known if CHART II will allow local jurisdictions to access data from other local jurisdictions. In order to facilitate integration of the Expanded IES with CHART II, the project team recommended that the IES participants get involved in the CHART II effort that seeks to establish the requirements for an archive server. This archive server feature of the CHART II system could serve as the basis to exchange data among the entities (especially since many of the participants are highly interested in archived data).

NEXT STEPS

Once the Implementation Plan has been finalized and accepted by all participants, each entity will deploy the IES according to the procedures established for that agency or jurisdiction. The project consultant will provide help to agencies requesting assistance with implementing the procedures and setting up the hardware.

Evaluation Plan Development and Effort

Parallel to these efforts, an evaluation plan will be created to identify a method for organizing the key findings into a concise format that can be read by a variety of audiences to form judgments on the overall merits of the project. Using previously developed performance criteria, a plan for evaluating the effectiveness of the deployment will be developed. The plan will include specification of performance objectives, data to be collected, a data collection schedule, analysis techniques, and evaluation criteria. The evaluation will likely be highly qualitative and will define the effect of the IES on each agency’s performance.

CONCLUSION

The purpose of this project is to introduce agencies, jurisdictions, and departments (and their staff) to the concepts of ITS and information sharing protocols. One of the main problems associated with ITS implementations is that many agencies dive headfirst into an implementation that involves technologies and concepts that are new to the implementing agency. This introduces a lot of stress and confusion for the personnel responsible for this implementation.
Another problem to be addressed in ITS implementations is the interagency cooperation required to implement and maintain the ITS project. In many implementations, institutional issues pose a problem that has to be addressed right at the beginning of a project.

The approach taken by this project sought to minimize institutional problems by inviting all of the stakeholders to participate in the development of the Baltimore Regional IES. Additionally, system users will learn about the most important aspects of ITS projects by relating solutions to familiar technologies and procedures.

One lesson learned from this project is that even a very comprehensive and detailed survey cannot replace personal interviews because only one-to-one interview settings allow participants to convey additional information and background information about the purpose of certain questions. Additionally, setting appointments for interviews elevates the importance of the survey and the responses. The downside of this approach is that it is very labor intensive, especially when numerous participants are involved, and therefore, may be cost prohibitive.
Development of a Decision Support Tool to Better Manage Alabama's Rural Public Transit Vehicles

MICHAEL D. ANDERSON, COREY N. DOSHIER, JASON D. MOODY, AND ADAM C. SANDLIN

In an ongoing effort to improve mobility and quality of life for Alabama’s citizens, researchers at the University of Alabama in Huntsville are working to improve the state’s ability to manage its rural transit fleet. This fleet management consists of a visual inspection for all state owned vehicles along with an examination of individual provider’s records to verify vehicle identification numbers and mileage as well as assess the overall condition of the vehicle based on appearance, passenger comfort level, and maintenance needs. The research team is developing a vehicle inventory database to track all of Alabama’s public transit vehicles and to be used as a decision support tool for the equitable acquisition of new vehicles and disposal of inadequate vehicles. This database will be used to establish a new record keeping system for all state owned vehicles. Specific functions for data entry, review, and reporting will be included with the database. This tool will allow access to information on all agencies in a user-friendly environment and benefit Alabama as an improved decision support tool providing potential cost savings associated with improved vehicle acquisition and disposal. Key words: public transportation, rural transit, fleet management, and GIS.

INTRODUCTION

Personal mobility is a vital component of an individual’s welfare and quality of life. However, in many rural areas of Alabama, a large portion of the state’s residents lack the resources or ability to provide for their own mobility and are dependent on the state’s rural transit program. The rural transportation program provides residents needed transportation services for trip purposes such as shopping, medical, and social/recreational.

The Alabama Department of Transportation (ALDOT) is responsible for providing new vehicles, as well as insurance and vehicle tags for the vehicles, and a portion of the agency’s operating costs. The vehicles are then provided to the local agencies, responsible for operation and maintenance.

The current funding for the state’s rural transit program allows for the procurement of approximately 50 to 60 vehicles, allowing for the replacement of about ten percent of the fleet per year. Unfortunately, the expected life for a rural transit vehicle is between five and seven years (or around 125,000 miles). This discrepancy is creating a situation where the local agencies are operating vehicles beyond the expected life as vehicles are aging faster than they can be replaced.

To compound the problem, many of the poorer counties providing rural transit service often require the vehicles to travel on unpaved roadways. Travel on these unpaved roadways rapidly deteriorates the vehicle operating condition. However, the expected vehicle life used to schedule disposal and procurement was based solely on vehicle mileage and age, making it difficult to equitably distribute newly acquired vehicles between counties with a differences in the percentage of paved and unpaved roadways.

Therefore, to assist the ALDOT in rural transit vehicle disposal and procurement equity, the University of Alabama in Huntsville has developed a decision support tool to better manage vehicle acquisition and disposal. The development of this tool was accomplished through a site inventory of all rural transit vehicles and the creation of a database system containing all rural transit vehicles in Alabama. This paper discusses the data collection procedure and development of the decisions support database, demonstrates some vehicle acquisition scenarios using the database, and comments on the possible reasons behind the different vehicle distributions. The paper concludes that the rural transit vehicle inventory was beneficial to Alabama, and the investment made will provide future cost-savings.

ALABAMA RURAL PROVIDERS

Alabama’s rural public transit system consists of 27 operators located throughout the state. Each operator is responsible for a specified number of counties, ranging from one county to nine counties with service provided in 52 of Alabama’s 67 counties. Currently, 640 rural transit vehicles are licensed and insured in Alabama. These vehicles are generally fifteen-passenger standard vans or cut-away chassis vehicles seating between 17 and 21 passengers (Figure 1 shows an example of a cut-away chassis vehicle used in Alabama for rural transit).

DATA COLLECTION EFFORT

The data collection effort involved an on-site inventory of all rural transit vehicles in Alabama. To assist in the data collection process, an inventory form was developed to allow the examiner to walk around the vehicle from the front driver’s side to the rear. Figure 2 shows the form. Some of the items collected on the form are the vehicle identification number, mileage, seating capacity, and vehicle type. In addition to these basic data elements, each of the inspectors was required to assign a condition rating for each vehicle. To ensure
consistency in the rating between data collectors and ALDOT, all members of the team inventoried one entire agency with the ALDOT present.

DATABASE DEVELOPMENT

The database developed to manage the rural transit fleet was designed to allow for new vehicle acquisition, annual updates, and vehicle disposals. Using the Microsoft Access Database program, a table was developed containing all required fields to support each of these three stages in a vehicle life. Then, separate data entry and report forms were developed to review, alter, or enter specific vehicle information. Examples of the forms for vehicle acquisition and disposal are shown in Figures 3 and 4.

FIGURE 1 Cut-away chasis rural transit vehicle

FIGURE 2 Inventory collection form

FINDINGS

The on-site inspections of all rural transit vehicles in the state revealed several interesting facts regarding rural transit in Alabama. First, there were over 40 vehicles discovered that were no longer operable for transit service. The inoperable vehicles were basically discarded by the agencies, however, they were not officially disposed of as a
reduction in fleet size would affect the operating budget amount provided to each agency from ALDOT. Second, there were 15 vehicles located that were licensed and titled by ALDOT, with vehicle title on hand, that were not claimed and operated by the respective agency intended to use the vehicle. In both of these cases, ALDOT realized a potential cost savings as these vehicles were subsequently removed from the state inventory list, saving approximately $60,000 or roughly $1,000 per vehicle for insurance and licensing.

The results of an examination of the percentage of vehicles in the fleet that would be replaced using the condition rating as the purchasing factor are summarized in Table 2.

Results of the inventory study show several unique vehicle acquisitions factors in Alabama. First, the Alabama-Tombigbee agency is in need of more vehicles than they would receive through an equitable purchase for all agencies, vehicle age or vehicle mileage. In addition, the 15 vehicles they would receive using the condition rating would essentially replace 58% of their operating fleet with new vehicles. One explanation for the additional vehicles to be purchased is that the Alabama-Tombigbee agency operates in Monroe and Clarke Counties which are large, scarcely populated counties offering home-to-work trips on unpaved roadways.

Second, the Baldwin County Commission experiences higher mileage with their transit vehicles in fewer years than any other agency in the state. There are two reasons behind this. First, they offer transit services to tourists visiting the Gulf Shores area, and second, Baldwin is one of the largest counties in the state. These reasons create a situation where Baldwin County vehicles have high mileage requiring replacement; however, they have not officially surpassed their useful life years.

Finally, Madison County Commission, which according to the condition rating would not receive any vehicles, has five vehicles that are among the state’s fifty oldest vehicles. This situation is possible because Madison County is one of the smaller counties in the state with a relatively small rural population and Madison County has a very few unpaved roadways in the county.

INVENTORY STUDY

To further demonstrate the usefulness of the database developed in this project, a case study was performed to examine the disposal and acquisition of vehicles based on mileage and age versus condition rating. Using the age and mileage criteria, new vehicle acquisition would be as shown in Table 1 (assuming 50 new vehicle acquisitions).

Using an equitable distribution providing the same number of new vehicles to each agency would allow each agency to acquire two vehicles per year (an annual total of 54 vehicles). However, twelve agencies are not in need of any new vehicles based on the condition rating and four agencies need only one vehicle.

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TABLE 1  New Vehicle Acquisitions

<table>
<thead>
<tr>
<th>Agency</th>
<th>Condition</th>
<th>Age</th>
<th>Mileage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alabama-Tombigbee</td>
<td>15</td>
<td>10</td>
<td>11</td>
</tr>
<tr>
<td>Lee-Russell COG</td>
<td>9</td>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>Blount County Commission</td>
<td>8</td>
<td>5</td>
<td>9</td>
</tr>
<tr>
<td>Northwest AL. COLG</td>
<td>5</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Baldwin County Commission</td>
<td>3</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>HELP INC.</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Exceptional Children</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Macon-Russell CAA</td>
<td>2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Lawrence County Commission (LCATS)</td>
<td>1</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Jackson County Commission</td>
<td>1</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Community Service of West Alabama</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Morgan County Commission (MCATS)</td>
<td>1</td>
<td>3</td>
<td>0</td>
</tr>
<tr>
<td>Madison County Commission</td>
<td>0</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>Escambia County Commission</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Covington County Commission</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
</tbody>
</table>

TABLE 2  Vehicle Acquisitions as a Percentage of Fleet.

<table>
<thead>
<tr>
<th>Agency</th>
<th>New Vehicles</th>
<th>Total Vehicles</th>
<th>% Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alabama-Tombigbee</td>
<td>15</td>
<td>26</td>
<td>58</td>
</tr>
<tr>
<td>Lee-Russell COG</td>
<td>9</td>
<td>34</td>
<td>26</td>
</tr>
<tr>
<td>Blount County Commission</td>
<td>8</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>Northwest AL. COLG</td>
<td>5</td>
<td>56</td>
<td>9</td>
</tr>
<tr>
<td>Baldwin County Commission</td>
<td>3</td>
<td>52</td>
<td>6</td>
</tr>
<tr>
<td>HELP INC.</td>
<td>2</td>
<td>14</td>
<td>14</td>
</tr>
<tr>
<td>Exceptional Children</td>
<td>2</td>
<td>11</td>
<td>18</td>
</tr>
<tr>
<td>Macon-Russell CAA</td>
<td>2</td>
<td>8</td>
<td>25</td>
</tr>
<tr>
<td>Lawrence County Commission (LCATS)</td>
<td>1</td>
<td>26</td>
<td>4</td>
</tr>
<tr>
<td>Jackson County Commission</td>
<td>1</td>
<td>11</td>
<td>9</td>
</tr>
<tr>
<td>Community Service of West Alabama</td>
<td>1</td>
<td>4</td>
<td>25</td>
</tr>
<tr>
<td>Morgan County Commission (MCATS)</td>
<td>1</td>
<td>33</td>
<td>3</td>
</tr>
<tr>
<td>Madison County Commission</td>
<td>0</td>
<td>21</td>
<td>0</td>
</tr>
<tr>
<td>Escambia County Commission</td>
<td>0</td>
<td>7</td>
<td>0</td>
</tr>
<tr>
<td>Covington County Commission</td>
<td>0</td>
<td>8</td>
<td>0</td>
</tr>
</tbody>
</table>

CONCLUSION

The results of this inventory study for the rural transit vehicles in Alabama identified many unique situations and recommendations. The study allowed for the identification of several vehicles that should no longer be part of the state’s insurance and licensing list. The study showed how a condition rating system could be used for replacement of vehicles that would be more equitable than a system based on mileage or age alone.

Possible additions to the database system that might improve equity in vehicle acquisition would be to incorporate the income of residents, average age of county residents, and vehicle ridership data. However, even without these additions, this project demonstrated that a relatively small investment made by the Alabama Department of Transportation has the potential to save money in future years as vehicles are removed from state lists and provides a condition-based system for equitable vehicle acquisition.

ACKNOWLEDGEMENTS

The authors of this paper would like to thank the Multimodal Bureau of the Alabama Department of Transportation for supporting this research effort and all the individual rural transit agencies whose time and commitment were necessary for completion of this project.
GIS-Based Itinerary Planning System for Multimodal and Fixed-Route Transit Network

Qiang Li and Carl E. Kurt

This paper introduces a multi-objective linear programming model for transit itinerary planning (TIP) with multimodal and fixed-route transit networks and presents an efficient two-phase TIP algorithm to find the optimal path that has the least combination cost from a given origin to a given destination. The algorithm recognizes the inherent nature of the multi-objective and time schedule constraint of TIP and considers trade-off among multiple optimization criteria in the path selection process. In particular, the algorithm of K shortest path problem with multiple time windows associated with time schedules is proposed in order to generate a set of path alternatives for evaluation and choice of the best path. A GIS-based Transit Itinerary Planning Decision Support System (GIS-TIPDSS) for assist passengers with itinerary decision making was developed. The GIS-TIPDSS was tested using data from a real transit network.

INTRODUCTION

Pre-trip transit information systems (PTIS) are the important component of advanced traveler information system (ATIS) and a means of alleviating the uncertainty about transit schedules and routes (1). Pre-trip information can cover a wide range of categories, including transit routes, maps, schedules, fares, and more. PTIS utilizes this information to support itinerary planning and helps travelers to make decisions on the itinerary path from given origin to destination, even if the trip involves multiple modes. The itinerary decision may involve transfers among different modes or routes and scheduling at the transfer points. This demands significant effort and time in a complicated multimodal transit network. PTIS allows travelers direct access to transit route information and improves travelers’ travel time by providing the optimal path between the origin and destination. PTIS may also attract more potential passengers to use the transit system because of its user-friendly interface and efficient routing.

In general, transit itinerary planning (TIP) is a multi-objective decision making process. The solution is rather complicated because of multiple decisions, multiple criteria, and uncertainty. The decisions depend on many factors such as in-vehicle travel time, walk time, transfer time, number of transfers, reliable schedules, etc. Because of the multi-objective nature of TIP and conflict criteria in problem solving, there may be no single optimal solution, but rather a group of potential best solutions, from which the decision maker selects the best compromise in an heuristic fashion.

In this paper, we propose a multi-objective linear programming model for TIP and a corresponding two-phase heuristic solution algorithm, which is built on a transit network representing a multimodal and fixed-route transit system. The two-phase algorithm is a heuristic solution of the multi-objective linear programming model. The first phase generates feasible path alternatives between given origin and destination points, with the aim of minimizing the total travel time including both travel and waiting time. The second phase is to evaluate these feasible paths and select the best path. The evaluation of the alternatives is based on a linear disutility function, which takes into account decision criteria such as number of transfer points, bus headway or frequency, total travel cost, etc.

One of the purposes of the research is to integrate the developed TIP model and geographic information system (GIS) technology. The GIS-TIPDSS was designed and implemented within a GIS environment, MapInfo for Windows. GIS plays a key role in integrating transportation data and analysis models. This paper describes the development of the GIS application system and related with components.

PROBLEM DEFINITION AND FORMULATION

Conceptually, TIP finds an optimal path from a given origin to a destination with specific departure or arrival times subject to certain constraints in a fixed-route transit network. The objectives of the routing problem may concern minimization of total travel time, number of transfers, number of modal changes, and trip cost. The time constraint considers that a vehicle arrives at a prescheduled transit station—called a time point—within a list of departure times and requires departure from the station at the next departure time. Therefore, the time constraint can be treated as multiple time windows bounded by scheduled departure time lists.

Modeling Multimodal Transit Network

A multimodal transit network may include bus, metro, and subway, where service routes are fixed and the departure or arrival at certain stations is scheduled in advance and generally not subject to changes. A derived network model must capture all possible fixed-route transit modalities and the interconnections among them. In particular, the network model must represent accessibility between any two modes or any two routes. The derived transit network consists of a set of nodes representing prescheduled stations, or time points, a set of
transit route segments connecting two nodes on the same route, and a set of transfer links connecting two nodes from different routes.

In addition to representing physical elements (e.g., route segments and transit stations) of the transit network, the network model also must consider the characteristics and activities of the fixed-route trips. Different virtual links, called transfer links, are added to the network graph to represent the following activities: access from traveler’s origin to a feasible transit station, waiting for a bus or a train, boarding/alighting a bus or a train, walking between two transit stations for transfer, etc.

Let $G = (N, L)$ be a multimodal transit network, where $N$ is the set of $n$ nodes and $L$ is the set of $m$ direct links $(i, j)$, connecting node $i$ to node $j$ in the network. Each distinct node represents a specific route station and is assigned a route ID. The network model includes two types of links, specifically:

1. Route links, which correspond to segments of transit routes between consecutive prescheduled transit stations on the routes.
2. Transfer links, which represent transfer time from one station to another, plus alighting/boarding time at the two stations.

Each link $(i, j) \in L$ has a weight associated with it given by the time (cost) required to travel or transfer from node $i$ to $j$. In particular, let $c_{ij}$ be the weight representing the sum of time (cost) traveling from node $i$ to node $j$ if the link $(i, j)$ is a route link, or the sum of the time spent on transferring and dwelling if the link $(i, j)$ is a transfer link.

Basically, we consider three attributes related to the travel cost, namely, in-vehicle riding time, average stopping time representing estimated stopping time at unscheduled transit stops or unexpected stops, and riding fare converted into time units. Similarly, by definition of transfer link, the transfer cost includes walking time from one station to another station and the boarding/alighting time. The representation of the transit network with weights allows one to search optimal paths, taking various costs into account.

**Formulation**

As mentioned before, the TIP, specifically, the two-point optimal routing problem with multiple time windows, can be formulated as a multi-objective integer programming problem given a set of optimal criteria and constraints. Basically, the following optimality criteria are considered in the mathematical programming model:

- minimizing total travel time, including riding and dwell time;
- minimizing total waiting time occurring at prescheduled nodes;
- minimizing path disutility cost caused by transferring activities.

Unfortunately, it is very difficult and expensive using current solution methods to deal directly with the multi-objective integer programming model. There are two major reasons for this. First, the problem formulation cannot be solved directly in a reasonable amount of time because the routing problem is an NP-hard problem, and there are no known polynomial time algorithms for the exact solution of the problem. Second, the optimality criteria include multiple objectives with complicated and even conflicting constraints. There may be no absolutely optimal solution; rather a best-compromised solution may be preferred. Therefore, a heuristic approach was taken instead of directly solving the mathematical programming models.

**HEURISTIC SOLUTION AND ALGORITHM**

Due to the considerable complexity of the TIP model, a heuristic solution is regarded as a natural approach for solving the problem. To develop the heuristic solution for the TIP, the TIP is considered as consisting of the following two major related sub-problems:

- A K-shortest path problem (KSPP) for generating a set of route alternatives between given origin and destination.
- A route choice problem, where the generated path alternatives are evaluated based on a route disutility function and the best compromise is selected.

The composite heuristics were developed to obtain the best-compromised solution by combining route’s alternative generation with route selection. The first step identifies $K$ shortest paths on the network between given origin and destination using a modified label setting technique. Then, the best-compromised path is selected in the set of the generated path alternatives based on the optimal criteria.

**Modified K Shortest Path Algorithm**

Using the mathematics notation in previous section, let $o$ and $d$ be given origin node and destination node, a path from node $o$ to node $d$ is an ordered sequence of nodes and links, namely, $(n_1 = o, n_2, n_3, ..., n_k = d)$, where $n_i \in N$, $(n_i, n_{i+1}) \in L$ for any $i = 1, 2, ..., k-1$. Let $P_{od}$ denote the set of paths from node $o$ to node $d$ in $G(N, L)$ and $T(P_{od}) = \{t(p), t(p), ..., t(p)\}$ denote the set of corresponding path times (costs). The objective of a single shortest path problem is to determine a path $P_{od}^* \in P_{od}$, for which $T(P_{od}^*) \leq T(P_{od}^*)$ holds for any path $P_{od} \in P_{od}$. If one requires not only to find the dominant shortest path but also to determine the second, third, ..., up to K-th shortest path between a given origin and destination, the shortest path model extends to calculate K shortest paths listed by non-decreasing order of the objective values. So, the KSPP is to determine a set of $P_{od} = \{P_{od}, P_{od}, ..., P_{od}\} \in P_{od}$ such that:

$$t(p_i) \leq t(p_{i+1}), \text{ for any } p_i \in P_{od}^k$$

$$t(p_k) \leq t(p_i), \text{ for any } p_i \in P_{od} - P_{od}^k$$

To tackle the KSPP with time constraint, a modified KSPP algorithm with multiple time windows was developed. The model identifies a set of feasible K shortest paths, which considers riding time and stopping time along a link, and waiting time at nodes.

Basically, this KSPP model is a modified version of label setting KSPP algorithm (2). Shier’s (3) study indicated that the label-setting algorithm required the least computation time for low density or sparse networks such as transportation networks. The modification of the KSPP model is related to addition of multiple time windows as time constraints. A vehicle that arrives at a prescheduled node requires departure only at one of scheduled departure time list. As a result, a waiting time may occur.

The modified label-setting algorithm solving the KSPP iteratively is presented in detail below. Assume for a given directed transit network $G(N, L)$, a timetable storing schedule list, a pair of origin $o$ and destination $d$, and a travel time matrix $T = \{t_i\}$ are available. For each node $i$, two k-vector labels, arrival time and departure time, denoted by
\[ A_i = \{a_i^1, a_i^2, \ldots, a_i^k\} \] and \[ D_i = \{d_i^1, d_i^2, \ldots, d_i^k\}, i \in \mathbb{N}, \] are attached. The entries of \( A_i \) and \( D_i \) are listed in increasing or at least non-decreasing order, respectively. The two \( k \)-vectors represent the earliest arrival and departure time, second earliest arrival and departure time, up to the \( k \)-th fastest path starting from node \( o \) to node \( i \). Each entry of the two \( k \)-vectors and corresponding node have two states, permanent or temporary. Initially, all entries are set to temporary. If all entries of \( k \)-vector of node \( i \) become permanent, the \( k \)-shortest paths from node \( o \) to node \( i \) are found and node \( i \) is set to permanent. In order to speed up the calculation and manage the labels efficiently, the minimum labels of \( D_i \) are maintained in a priority queue, specifically, a binary heap, \( Q = \{q_1, q_2, \ldots, q_n\} \), where \( q_i \) represents the minimum value of temporary labels associated with a node departure label vector. Once all entries of \( D_i \) vector become permanent, the search of \( k \)-shortest paths from the origin to destination is complete. The entry values of \( A_i \) indicate the \( k \) shortest travel times in increasing order.

**Algorithm Steps**

**Step 1:** Initialize network. For all node \( i \neq o \in \mathbb{N}, \) set \( A_i = \{\infty, \infty, \ldots, \infty\} \) and \( D_i = \{\infty, \infty, \ldots, \infty\}, A_o = \{s_t, \infty, \infty, \ldots, \infty\} \) and \( D_o = \{s_t, \infty, \infty, \ldots, \infty\}, \) where \( s_t \) is the starting time at node \( o \). Insert \( s_t \) into the priority queue \((q_1 = s_t)\) and set \( d_s^o = \infty \) permanent. Then, set all entries except entries of node \( o \) to temporary. The entry values of \( A_o \) indicate the \( k \) shortest travel times in increasing order.

**Step 2:** Let \( l = q_i \) associated with node \( n^* \), for each node \( j \) adjacent to node \( n^* \) :

1. If \( l + d_j^o < \infty \) replace the first encountered entry \( (a_j^m) \), which greater than \( l + d_j^o \), by \((l + d_j^o)\); 2b: If \( dt_t(j) < a_j^m \leq dt(j) \), replace the corresponding \( d_j^m \) in \( D_j \) with \( d_j^m \). Otherwise, \( d_j^m = \infty \), indicate that no scheduled departure time in time list is available when vehicle arrival at node \( j \) at time \( a_j^m \): 2c: If updated value of \( d_j^m \) is smaller than node \( j \)’s entry in the priority queue, replace node \( j \)’s entry by \( d_j^m \) and reorder the queue.

**Step 3:** Set the entry of \( A_{n^*} \) corresponding to \( l \) permanent. If all entries in \( A_{n^*} \) become permanent, remove the node \( n^* \) from the priority queue. Otherwise, replace node \( n^* \)’s entry in the priority queue by next remaining temporary label in \( D_{n^*} \) and reorder the queue.

**Step 4:** If the removed \( n^* = d \), stop iteration. Otherwise, go to step 2.

At the end of this iteration, the arrival time vector at node \( d (A_d) \) gives the minimum total travel time, second minimum total travel time, third, up to \( k^m \), required from origin \( o \) to node \( i \) by going through only the schedule and path feasible nodes.

**Paths Evaluation and Selection**

In the real world, transit passengers choose the best path by considering not only the usual link-based shortest criteria but also non-link based factors, which could affect their path choice, such as average headway (frequency) along the path, total number of transfer points along the path, and total path cost (e.g., fare). It is difficult and even impossible to implement the path-based attributes into the link-based cost functions for shortest path calculation in label setting shortest path algorithm. This is why the objective function is presented as three separate optimal criteria: link-based travel time, node-based waiting time associated with the specific entry link, and path-based disutility cost. After the \( K \) shortest path model takes care the first two optimal criteria, this evaluation algorithm will select a path with minimized disutility cost.

The algorithm for path evaluation and selection is simple. For each generated shortest path, the value of the disutility function associated with the path is calculated. The comparison between the values of the disutility is performed. Finally, the best optimal path is selected in the \( K \) generated paths.

**DEVELOPMENT OF GIS-TIPDSS SYSTEM**

Design of Transit Itinerary Planning Systems requires a computer tool such as GIS that can integrate and maintain large-size spatial transportation databases from different data sources and can conduct and support spatial and temporal analysis. Particularly, GIS has the ability to model and refine large-scale networks and control quality of information flow among various models. To integrate TIP model and GIS technologies, the functionality of a GIS system needs to be extended or modified. The key to the successful integration is the design of spatial network databases and associated management tools to meet the various spatial and temporal functions needs of TIP.

GIS-TIPDSS was implemented on a personal computer and intended for potential transit users. There are three main modules consisting of the GIS-TIPDSS: input module, TIP module, and output module. The input module includes preprocessing passenger information and loading transit network. The TIP module performs the two-phase itinerary planning and produces the best-compromised path. The output module includes post-processing the results from TIP module and displaying the best path. Input and output modules were designed and developed using the MapBasic provided by MapInfo. The routing module was coded using Visual C++.

The input and output modules provide interfaces between the GIS-TIPDSS and users. Since MapInfo provides the GUI development platform and common parameter standards, the user-friendly GUI is incorporated well into the GIS-TIPDSS system. The general principle of the GUI design is that the GUI utilizes the MapBasic program to maximize the use of built-in MapInfo utilities. The users or passengers can enter all the information pertaining to a desired trip such as origin/destination addresses and either arrival or departure times, etc. The input module processes the passenger’s input information. The process includes finding feasible and accessible transit stations closest to the given origin and destination and with the earliest departure time and then passing the info on routing module. Based on the time and date required for riding service, the input module will load available transit routes with corresponding time schedule table. The following are the basic steps:

**Step 1:** Locate the origin and destination on the map using the GIS address geocoding;

**Step 2:** Identify the nearest transit stations to the origin and destination locations;

**Step 3:** Check the feasibility of schedule time of the selected stations and determine the proper departure time at starting station.

The TIP module is the key component of the GIS-TIPDSS. It calculates and recommends the best-compromised path based on the
requirement of passengers. From the input module, the passenger’s spatial and temporal information is passed on the TIP module. An optimal path is produced from the TIP module.

The path planning proceeds as follows:
Step 1: Calculate $K$ shortest paths from the origin node to the destination node, minimizing in-vehicle riding time and waiting time. These paths are stored in ascending order with respect to total travel time along path;
Step 2: Set $k = 1$ and Set $p^* = \infty$; retrieve the $k^{th}$ shortest path, calculate the disutility function over the path, and then store the function value in $p^*$;
Step 3: Set $k = k + 1$. If $k > K$, go to step 4; otherwise, retrieve the $k^{th}$ shortest path, calculate the disutility function over the path, if the value of the function $< p^*$, replace the value of $p^*$ by the value of disutility function. Back to start of the step 3;
Step 4. Take $p^*$ as the optimal path, and stop.

The result $p^*$ from the TIP module is passed to the output module. The passengers get visualized best path between the given origin and destination on transit network map.

The GIS-TIPDSS was tested using a real transit network, Las Vegas Transit Bus System. The network consists of 4336 stops and 549 time points. The average number of transit running routes each day is about 85. The TIP algorithm was enhanced to consider more complicated scenarios because of the case study. The average time needed to find an optimal path solution from a given origin and destination was about 6 seconds. By testing the real network, it was demonstrated that the two-phase TIP algorithm is capable of producing an optimal path solution in a real multimodal transit network within a reasonable time frame.

CONCLUSION

In this paper, the TIP was modeled as a multi-objective routing-selection problem. The heuristic methodologies and algorithm were developed for generating the path alternatives and selecting a best path by evaluating the alternatives. The algorithm is a two-phase procedure, which is designed to simulate the decision making process for TIP. A GIS application for TIP to assist passengers with itinerary decision making was developed.

REFERENCE

Evaluation of Three Supplementary Traffic Control Measures for Freeway Work Zones

Nawaz M. Shaik, Kristen L. Sanford Bernhardt, and Mark R. Virkler

Controlling traffic in work zones to improve safety has long been a major concern for highway agencies. Three traffic control devices—white lane drop arrows, orange rumble strips, and the CB wizard alert system—were tested for their effectiveness in improving merging and reducing speed and speed variance at an interstate highway work zone in Missouri. Results of implementing the white lane drop arrows and the CB wizard alert system indicate decreases in the percentage of vehicles in the closed lane, mean speed, and speed variance. It also appears that the CB wizard alert system may be more effective than the white lane drop arrows. The CB wizard alert system in conjunction with the orange rumble strips did show similar reductions, but they were much smaller in comparison to the CB wizard alert system alone.

INTRODUCTION

Safety in work zones has been recognized as a significant problem for many years. The subject has received additional attention at times when improvement or rehabilitation of existing facilities is more prevalent than new construction. This shift in approach makes maintenance of traffic on highway facilities during repair or reconstruction critical. The closure of a lane on a four-lane high-speed facility during construction or maintenance activity creates many potential safety problems. Lane closures require the driver to make behavior adjustments, such as reducing speed and/or changing lanes. On high-volume facilities, problems often occur when two or more lanes of traffic must be warned sufficiently in advance so that motorists may travel safely through the one lane passing through the work zone.

Past studies of accidents in work zones have found a higher accident rate for work zones than for other sections of the road (1,2,3,4). The predominant factors contributing to work zone crashes appear to be failure to drive within the designated lane, failure to reduce speed, and failure to yield right of way, and the occurrence and severity of accidents has been related to both vehicle speed and speed variation (5).

The Manual on Uniform Traffic Control Devices (MUTCD) describes use of signs, signals, hand-signaling devices, channelizing devices, and deflection and attenuation devices along the approach to and within a work zone (6). In order to further reduce the number of crashes that occur in work zones, consideration should be given to additional traffic control devices. In an effort to improve the flow conditions approaching work zones, four states—Missouri, Kansas, Iowa, and Nebraska—joined together in a study of various additional traffic control devices. Three traffic control devices—white lane drop arrows, orange rumble strips, and the CB wizard alert system—were tested in Missouri and are described here. The hypotheses tested examined whether the devices alone or in combination reduced the mean speed of the traffic, reduced speed variance, and improved advance merging of the two lanes. This research also provided information about lane distributions, 85th percentile speeds, 15th percentile speed, 10 mph pace, percentage of vehicles in the 10 mph pace, and the percentage of vehicles below the speed limit.

OBJECTIVE

The primary objective of the study was to determine the effectiveness of the three traffic control devices located in the approach to a highway work zone. The devices are intended for use with stationary long-term work zones and with short-term moving projects. This research will help departments of transportation develop guidelines for selecting alternative traffic control devices for use in a work zone. The specific research tasks were:

1. To test and evaluate the effectiveness of the devices in reducing the average speeds and speed variance approaching the work zone;
2. To test the effectiveness of the devices in merging the traffic into one lane before the work zone starts;
3. To determine the opinion of drivers driving through the work zone about the CB wizard alert system;
4. To determine if these devices change the accident rate; and
5. To determine the ease of installation and removal and durability of the arrows and orange rumble strips.

This paper describes the devices, data collection procedures, results, and conclusions.

TRAFFIC CONTROL DEVICES

Orange Rumble Strips

A vehicle passing over the orange rumble strips experiences a bump, which alerts a driver to hazards ahead. The strips, which can be cut to length, are 4” wide and 0.15” thick. The orange color designates the construction site. Six sets of removable orange rumble strips were installed at locations approaching the work zone (Figure 1). Each set of strips contained six strips, which were placed on 10’ centers at the site farthest from the lane drop, 5’ centers at the next site, and 2’ centers at the 4 remaining sites. It was expected that the rumble strips
FIGURE 1  Schematic location of detectors and devices on work zone approach

would alert drivers still in the closed lane and approaching the lane closure to change lanes and reduce speed.

White Lane Drop Arrows

The white lane drop arrows were placed at a 45° angle to the travel direction. The arrows are approximately 7' long and slightly thinner than the rumble strips. The large size of the arrows and their white color provide a visual and aural feedback to the driver who passes over them. Three removable white lane drop arrows were installed near the beginning of the lane taper for the lane closure, as shown in Figure 1. It was expected these would alert drivers to change lanes and move to the open lane.

CB Wizard Alert System

The trailer-mounted CB wizard alert system broadcasts a work zone alert and information for advance warning about a lane closure on a CB radio channel. The wizard was placed approximately 6 miles (9.67 km) upstream of the lane closure and transmitted the following message when the right lane was closed: “This is the Missouri Department of Transportation. The right lane of Eastbound I-70 is closed ahead. Watch for slow or stopped traffic.” A similar message was transmitted when the left lane was closed. It was expected that this would bring an earlier lane change response by truck drivers and lower speeds upstream of the closure.

DATA COLLECTION

The field research was conducted on a highway with a 70 mph speed limit, but the posted speed limit approaching the work zone was reduced to first 60 mph and then 50 mph. The devices were tested at a stationary long-term work zone on eastbound Interstate 70 (I-70) near Columbia, Missouri. Interstate systems are similar throughout the United States, so the results may be representative of similar facilities in other states. The pavement related work at this site included cold milling, pavement repair and resurfacing. The average daily traffic was approximately 14,600 vehicles, with 25.6 % non-passenger vehicles in the eastbound direction of travel. The right lane of the eastbound highway was closed first, followed by the left lane.

Data were collected at four locations along the approach to the work zone, as shown in Figure 1, before any of the devices were in place and again after they were installed. Vehicle speeds, volumes, and vehicle classifications were collected in 15-minute intervals. Data for the white lane drop arrows and the CB wizard alert system were collected separately. Data for the orange rumble strips were collected while the CB wizard alert system was operating. Due to breaks in the pneumatic tubes, it was not always possible to collect data at all four sites during all time periods, but a minimum of 24 hrs of data were collected for each device tested.

The vehicles were grouped into passenger vehicles (2 axles), non-passenger vehicles (more than 2 axles), and all vehicles. The times of the observations were classified according to light conditions as day, night, and twilight (dawn to dusk). Finally, the levels of service in the closed and passing lanes were used to group the data into uncongested conditions, where both lanes had levels of service A, B, C, or D, or congested conditions, in which at least one of the lanes had level of service E or F.

The driver survey was conducted at a nearby truck stop, about 3 miles upstream of the lane closure. Surveys were conducted between approximately 9:00 am and 5:00 pm on several days. In addition, accident data were collected from one mile upstream of the first counter site through the end of the work zone, and observations were made regarding the ease of installing and removing these devices and their durability.

RESULTS

The traffic control devices were primarily intended to reduce traffic speeds, speed variability, and the percentage of vehicles in the closed lane. The data analysis examined the difference in the parameters before and after the devices were installed. The primary measures of effectiveness were lane distributions, speed mean, and speed variance; however, other parameters were also studied for significance in the evaluation of the traffic control devices. For the before and after studies, the analysis took into consideration the effects of time of day and class of vehicle. Due to the small amount of data for the dawn/dusk periods, a difference
would be extremely difficult to identify. A more detailed description of the data analysis and results appears in the complete report on this project (7).

**Percentage of Vehicles in the Closed Lane**

Table 1 summarizes the average changes in the percentage of vehicles in the closed lane for the four sites studied. When the white lane drop arrows and the CB wizard alert system were in place, the percentage of vehicles remaining in the closed lane decreased. However, the decrease associated with the white lane drop arrows was only significant at the two detectors farthest upstream from the arrows. Drivers at these two locations would not have seen the arrows yet, so either the effects of the arrows were propagated upstream, or some other factors caused the response while the arrows were in place. The white lane drop arrows were associated with greater decreases during the day than during the night, and they were associated with a greater reduction of passenger vehicles than non-passenger vehicles in the closed lane.

**TABLE 1 Change in Vehicles in the Closed Lane**

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Time</th>
<th>Traffic Control Device*</th>
<th>White Lane Drop Arrows*</th>
<th>Wizard System*</th>
<th>CB Wizard Alert System and Orange Rumble Strips*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>% (%)</td>
<td>% (%)</td>
<td>% (%)</td>
</tr>
<tr>
<td>All Vehicles</td>
<td>Day</td>
<td>-1.7 (20.8)</td>
<td>-2.9 (15.8)</td>
<td>+0.13 (2.95)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Night</td>
<td>-1.4 (7.1)</td>
<td>-3.1 (7.5)</td>
<td>-2.0 (11.7)</td>
<td></td>
</tr>
<tr>
<td>Passenger</td>
<td>Day</td>
<td>-1.8 (21.7)</td>
<td>-1.8 (12.0)</td>
<td>+1.44 (1.78)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Night</td>
<td>-1.7 (22.5)</td>
<td>-0.3 (0.3)</td>
<td>-1.5 (10.6)</td>
<td></td>
</tr>
<tr>
<td>Non-Passenger</td>
<td>Day</td>
<td>-1.0 (32.0)</td>
<td>-4.4 (29.8)</td>
<td>+0.1 (12.25)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Night</td>
<td>-1.8 (17.8)</td>
<td>-6.2 (44.0)</td>
<td>-2.5 (11.5)</td>
<td></td>
</tr>
</tbody>
</table>

*White lane drop arrows and CB wizard alert system were compared to no devices; CB wizard alert system and orange rumble strips were compared to CB wizard alert system alone.

The CB wizard alert system may be more effective during the day than during the night. The effect was greater on non-passenger vehicles than on passenger vehicles, which was expected, since the CB wizard alert system is used mostly by non-passenger vehicles. During the day, the percentage of vehicles in the closed lane during congested conditions increased at Sites 3 and 4, but this was not statistically significant.

When the orange rumble strips were added to the wizard system, the percentage of vehicles in the closed lane did not change. For the CB wizard alert system and orange rumble strips, there was a left-lane closure in operation. Table 1 shows an increase in the percentage of vehicles in the closed lane during the day and a decrease during the night. During the night, the CB wizard alert system and orange rumble strips had a similar effect on both passenger and non-passenger vehicles.

**Mean Speed**

A general trend of reduction in speed means was observed for the white lane drop arrows and the CB wizard alert system (Tables 2 and 3). Data for the CB wizard alert system and orange rumble strips show a small increase in the mean speed of the vehicles in the driving lane, which coincides with the increase in the percentage of vehicles in the closed lane during similar conditions. In general, the greater the reduction of percentage of vehicles in the closed lane, the greater the reduction in mean speed. Changes observed for both the white lane drop arrows and the CB wizard alert system were greater during the night than during the day, and there was a greater reduction in the mean speed in the passing lane than in the driving lane.

**TABLE 2 Change in Mean Speed During the Day**

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Lane#</th>
<th>Traffic Control Device*</th>
<th>White Lane Drop Arrows* (mph (%)</th>
<th>Wizard System* (mph (%)</th>
<th>CB Wizard Alert System and Orange Rumble Strips* (mph (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>All Vehicles</td>
<td>Dr Ln</td>
<td>-8.4 (14.0)</td>
<td>-5.6 (9.3)</td>
<td>0.6 (1.1)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ps Ln</td>
<td>-3.4 (5.4)</td>
<td>-6.0 (9.6)</td>
<td>-1.0 (1.8)</td>
<td></td>
</tr>
<tr>
<td>Passenger</td>
<td>Dr Ln</td>
<td>-9.8 (15.9)</td>
<td>-8.5 (13.5)</td>
<td>0.4 (0.9)</td>
<td></td>
</tr>
<tr>
<td>Vehicles</td>
<td>Ps Ln</td>
<td>-7.2 (10.9)</td>
<td>-9.4 (14.1)</td>
<td>-1.3 (2.2)</td>
<td></td>
</tr>
<tr>
<td>Non-Passenger</td>
<td>Dr Ln</td>
<td>-3.1 (5.4)</td>
<td>-3.3 (5.3)</td>
<td>0.5 (0.9)</td>
<td></td>
</tr>
<tr>
<td>Vehicles</td>
<td>Ps Ln</td>
<td>-4.8 (3.2)</td>
<td>-5.3 (9.0)</td>
<td>-5.3 (8.8)</td>
<td></td>
</tr>
</tbody>
</table>

*White lane drop arrows and CB wizard alert system were tested when the driving lane was closed; CB wizard alert system in conjunction with orange rumble strips was tested when the passing lane was closed.

**TABLE 3 Change in Mean Speed During the Night**

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Lane#</th>
<th>Traffic Control Device*</th>
<th>White Lane Drop Arrows* (mph (%)</th>
<th>Wizard System* (mph (%)</th>
<th>CB Wizard Alert System and Orange Rumble Strips* (mph (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>All Vehicles</td>
<td>Dr Ln</td>
<td>-9.7 (16.6)</td>
<td>-8.5 (13.8)</td>
<td>-1.4 (2.4)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ps Ln</td>
<td>-6.9 (10.9)</td>
<td>-19.6 (33.7)</td>
<td>-1.0 (1.4)</td>
<td></td>
</tr>
<tr>
<td>Passenger</td>
<td>Dr Ln</td>
<td>-9.6 (16.7)</td>
<td>-7.8 (12.6)</td>
<td>-1.6 (2.7)</td>
<td></td>
</tr>
<tr>
<td>Vehicles</td>
<td>Ps Ln</td>
<td>-6.6 (10.4)</td>
<td>-28.8 (46.2)</td>
<td>-1.4 (2.1)</td>
<td></td>
</tr>
<tr>
<td>Non-Passenger</td>
<td>Dr Ln</td>
<td>-16.2 (29.4)</td>
<td>-11.7 (19.3)</td>
<td>-1.0 (1.8)</td>
<td></td>
</tr>
<tr>
<td>Vehicles</td>
<td>Ps Ln</td>
<td>-3.7 (5.8)</td>
<td>-3.4 (5.4)</td>
<td>-0.1 (0.8)</td>
<td></td>
</tr>
</tbody>
</table>

*White lane drop arrows and CB wizard alert system were tested when the driving lane was closed; CB wizard alert system in conjunction with orange rumble strips was tested when the passing lane was closed.

*White lane drop arrows and CB wizard alert system were compared to no devices; CB wizard alert system and orange rumble strips were compared to CB wizard alert system alone.
It was expected that the CB wizard alert system would affect non-passerger vehicles more than passenger vehicles, but the data show a greater effect on passenger vehicles. It is interesting to note that, during the night, a 46.2% reduction in the mean speed of passenger vehicles was observed, compared to 5.4% reduction in the mean speed of non-passerger vehicles. Data from both the white lane drop arrows and the CB wizard alert system showed a greater reduction in the mean speeds of passenger vehicles than non-passerger vehicles.

On average, data for both the white lane drop arrows and the CB wizard alert system showed a reduction in the mean speed of about 10%. The speed reduction was greater in the driving lane than in the passing lane. The data for the white lane drop arrows indicate a greater effect on non-passerger vehicles during the night time than during the day. Adding the orange rumble strips to the CB wizard alert system resulted in small speed reductions.

Corresponding to the decrease in the mean speeds, there was an increase in the percentage of vehicles below the speed limit in all cases. The CB wizard alert system showed a much greater increase in the percentage of vehicles below the speed limit than the white lane drop arrows. There were similar reductions in the 85th percentile speed and 15th percentile speed, as discussed in the detailed report of the study (7).

**Standard Deviation of Mean Speed**

A small standard deviation of the mean speed for vehicles approaching a work zone is desirable. The changes in standard deviations were both positive and negative (Tables 4 and 5). The observed effect on standard deviation was greater during the night than during the day. The standard deviation in the driving lane increased during the day, and on further analysis (7), it was found that this increase was most apparent in passenger vehicles.

### TABLE 4 Change in Standard Deviation of Mean Speed During the Day

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Lane</th>
<th>Traffic Control Device</th>
<th>White Lane Drop Arrows</th>
<th>Wizard System</th>
<th>CB Wizard Alert System and Orange Rumble Strips</th>
</tr>
</thead>
<tbody>
<tr>
<td>All Vehicles</td>
<td>Dr Ln</td>
<td>+0.3 (3.8)</td>
<td>+1.7 (23.5)</td>
<td>-0.1 (2.7)</td>
<td>+1.9 (23.5)</td>
</tr>
<tr>
<td></td>
<td>Ps Ln</td>
<td>-0.3 (2.6)</td>
<td>-0.7 (8.4)</td>
<td>+1.2 (19.9)</td>
<td>-0.3 (2.6)</td>
</tr>
<tr>
<td>Passenger</td>
<td>Dr Ln</td>
<td>+0.4 (4.5)</td>
<td>+1.6 (22.7)</td>
<td>-0.2 (3.2)</td>
<td>+0.4 (4.5)</td>
</tr>
<tr>
<td>Vehicles</td>
<td>Ps Ln</td>
<td>+0.1 (2.4)</td>
<td>-0.7 (2.9)</td>
<td>+1.5 (19.3)</td>
<td>+0.1 (2.4)</td>
</tr>
<tr>
<td>Non-Passenger</td>
<td>Dr Ln</td>
<td>-0.6 (22.4)</td>
<td>-1.9 (50.0)</td>
<td>+0.4 (7.4)</td>
<td>-0.6 (22.4)</td>
</tr>
<tr>
<td></td>
<td>Ps Ln</td>
<td>-0.5 (8.3)</td>
<td>-1.2 (18.6)</td>
<td>+0.8 (22.9)</td>
<td>-0.5 (8.3)</td>
</tr>
</tbody>
</table>

*White lane drop arrows and CB wizard alert system were compared to no devices; CB wizard alert system and orange rumble strips were compared to CB wizard alert system alone.

Data for the CB wizard alert system indicate little effect on passenger vehicles, especially in the driving lane. Though the CB wizard alert system was associated with a lower standard deviation of mean speed of non-passerger vehicles during the day, the standard deviation increased at night. Even though data for both the white lane drop arrows and the CB wizard alert system show a greater effect on reducing the mean speeds of passenger vehicles, they show an increase in the standard deviations, which may be an indication of erratic maneuvers of drivers of passenger vehicles. The CB wizard alert system and the orange rumble strips data show an increase in the standard deviation of mean speed in the closed passing lane.

### Driver Survey Results

The responses to the driver survey questions are summarized in Table 6. The CB wizard alert system was installed a few miles in advance of the work zone; therefore, people driving into the work zone were more likely to hear the message than were people driving in the opposite (westbound) direction. The majority of the drivers understood all or part of the message, and 97.3% of the drivers felt the information they received was at least somewhat useful. The drivers surveyed were enthusiastic about using the CB radios to give warnings about work zones and lane closures.

### Accident Analysis

The time periods when the devices were in place were too short to indicate a statistically significant reduction in accidents. However, a sharp rise in accidents could indicate that the devices are hazardous.
### TABLE 6 Frequencies of Responses to Driver Survey Questions

<table>
<thead>
<tr>
<th>Items</th>
<th>Proportion</th>
</tr>
</thead>
<tbody>
<tr>
<td>What type of vehicle?</td>
<td>Heavy truck or trailer (87.8%)</td>
</tr>
<tr>
<td></td>
<td>Light truck/Van (8.1%)</td>
</tr>
<tr>
<td></td>
<td>Bus (1.6%)</td>
</tr>
<tr>
<td></td>
<td>Passenger car (2.4%)</td>
</tr>
<tr>
<td>Which direction of travel?</td>
<td>Eastbound (62.7%)</td>
</tr>
<tr>
<td></td>
<td>Westbound (37.3%)</td>
</tr>
<tr>
<td>How many years driving this type of vehicle?</td>
<td>&lt; 1 year (2.4%)</td>
</tr>
<tr>
<td></td>
<td>1-2 years (6.5%)</td>
</tr>
<tr>
<td></td>
<td>2-5 years (35.0%)</td>
</tr>
<tr>
<td></td>
<td>&gt; 5 years (56.1%)</td>
</tr>
<tr>
<td>How far in advance of a work zone are warning signs needed?</td>
<td>&lt; 1 mile (32.5%)</td>
</tr>
<tr>
<td></td>
<td>1-2 miles (14.6%)</td>
</tr>
<tr>
<td></td>
<td>3-5 miles (41.5%)</td>
</tr>
<tr>
<td></td>
<td>≥ 6 miles (38.2%)</td>
</tr>
<tr>
<td>Did you know about the lane closure before starting your trip?</td>
<td>Yes (41.5%)</td>
</tr>
<tr>
<td></td>
<td>Yes, but forgot (0.8%)</td>
</tr>
<tr>
<td></td>
<td>No (57.7%)</td>
</tr>
<tr>
<td>How did you find out about the lane closure?</td>
<td>Radio (4.3%)</td>
</tr>
<tr>
<td></td>
<td>CB radio conversation (47.1%)</td>
</tr>
<tr>
<td></td>
<td>CB radio recorded message (41.4%)</td>
</tr>
<tr>
<td></td>
<td>Word of mouth (5.7%)</td>
</tr>
<tr>
<td>Did you hear the message in the vehicle you are driving?</td>
<td>Yes (60.2%)</td>
</tr>
<tr>
<td></td>
<td>No (39.8%)</td>
</tr>
<tr>
<td>Did you understand the message?</td>
<td>Yes (64.8%)</td>
</tr>
<tr>
<td></td>
<td>Yes, but message not clear (31.1%)</td>
</tr>
<tr>
<td></td>
<td>No (4.0%)</td>
</tr>
<tr>
<td></td>
<td>No opinion (0.0%)</td>
</tr>
<tr>
<td>Do you find the information useful?</td>
<td>Very useful (39.5%)</td>
</tr>
<tr>
<td></td>
<td>Useful (57.9%)</td>
</tr>
<tr>
<td></td>
<td>Not useful (1.3%)</td>
</tr>
<tr>
<td></td>
<td>No opinion (1.3%)</td>
</tr>
<tr>
<td>Did you drive through a work zone with a recorded CB radio warning before?</td>
<td>Yes (36.1%)</td>
</tr>
<tr>
<td></td>
<td>No (63.8%)</td>
</tr>
<tr>
<td>How hazardous are interstate work zones compared to normal highway segments?</td>
<td>More Hazardous (55.3%)</td>
</tr>
<tr>
<td></td>
<td>About the same (34.1%)</td>
</tr>
<tr>
<td></td>
<td>Less hazardous (7.3%)</td>
</tr>
<tr>
<td></td>
<td>No opinion (3.2%)</td>
</tr>
</tbody>
</table>

No accidents were found to have occurred because of the technologies that were tested. The types of accidents that would be expected to occur due to the traffic control devices include:

- The white lane drop arrows were expected to help the drivers still in the closed lane to change lanes, which would be expected to cause a changing lane accident. No lane-changing accidents occurred when the arrows were placed on the pavement.
- The CB wizard alert system was expected to increase the driver awareness of the work zone well in advance and prepare drivers for the conditions ahead. Drivers would be expected to slow down and change to the open lane well in advance of the lane closure. If the CB wizard alert system were to cause an accident, it would be expected to be of the changing lane type. No changing lane accidents occurred when the CB wizard alert system was tested.
- The orange rumble strips were expected to warn drivers still in the closed lane and traveling at high speeds to slow down and change lanes. If the orange rumble strips were to cause an accident, it would be expected to be either a changing lane or out of control type accident. No accidents of either type occurred while the orange rumble strips were in place.

### Durability and Removeability

**Orange Rumble Strips**

The traffic control contractor’s first attempt to install the orange rumble strips occurred shortly after a light rain—the pavement surface appeared to be dry. The personnel laid out the strips, walked on the surface area of the strips to apply pressure, then rolled their pickup truck tires over the surface area. By the next morning and after a heavy rain, most of the strips had lost adhesion and had been removed from the pavement by traffic.

Approximately one month later, on thoroughly dry pavement, the contractor’s installation was successful. The strips were laid in place and a 200 lb (90 kN) roller was used to apply pressure (per the manufacturer’s instructions). The process, including a temporary lane closure and installation of the strips, required approximately three-and-one-half hours for a two-person installation team. The strips remained in good condition for eight days. Strip removal exhibited no particular difficulties and required approximately two hours (including a temporary lane closure) for a two-person team with no special tools.
White Lane Drop Arrows

The traffic control contractor’s personnel laid out the arrows, walked on the surface area of the arrows to apply pressure, then rolled their pickup truck tires over the surface area. The installation process, including a temporary lane closure, required approximately two hours for a two-person team. The arrows remained in good condition for seven days. Arrow removal required approximately two hours (including a temporary lane closure) for a two-person team with no special tools and exhibited no particular difficulties.

CONCLUSIONS

This study examined the effect of white lane drop arrows, the CB wizard alert system, and orange rumble strips on vehicle speeds, lane distributions, and vehicle conflicts at a long-term work zone in Missouri. Data indicate that the white lane drop arrows and CB wizard alert system were effective in reducing the speed of traffic approaching the work zone. Data for the CB wizard alert system in conjunction with the orange rumble strips show only small effects on vehicle speeds when compared to the CB wizard alert system alone. When compared to the white lane drop arrows, the CB wizard alert system was more effective. The results correlate well with the expectation that the use of CB radio is prevalent among non-passenger vehicles. Though it was not possible to test the orange rumble strips alone, they were tested while the CB wizard alert system was operating. The orange rumble strips did not bring about a large change with respect to the CB wizard alert system alone. The devices also showed a significant reduction in the percentage of vehicles below the speed limit—about 40% with the white lane drop arrows and up to 120% with the wizard system. The devices were not found to be hazardous, as they did not cause any accidents. They are easy to install and remove, and they worked for the lifetime of this work zone. Thus, the devices are easy-to-use with short-term work zones. Because the arrows and rumble strips were in place for approximately one week, durability for longer periods cannot be projected based on this study.

ACKNOWLEDGMENTS

This research was performed as part of the Midwest State Smart Work Zones Development Initiative (MwSWZDI) through the Mid-American Transportation Center (MATC). The project was overseen by the Missouri Department of Transportation (MoDOT).

REFERENCES

Midwest Smart Work Zone Deployment Initiative: Kansas’ Results

ERIC MEYER

During 1999, the Departments of Transportation from the states of Kansas, Nebraska, Iowa, and Missouri conducted a pooled-fund study of innovative devices designed to improve the safety and efficiency with which highway maintenance is conducted. In the state of Kansas, a total of nine devices were evaluated, including lighted raised pavement markers, CB-radio warning systems, and radar-triggered speed displays, among others. This paper gives an overview of the devices evaluated and summarizes the results of each of the evaluations. All of the products showed potential for improving work zone safety and operations. Some of the products require further development before they can be recommended for widespread deployment. The four products which seemed to show the most promise were orange removable rumble strips; the Vertical SafetyCade—designed to replace the reflektorized drum—a radar-triggered speed display, and an experimental configuration of Lightguard lighted raised pavement markers used to delineate a crossover in an interstate work zone. Speeds, lane distributions, and lane positions were used when appropriate to evaluate the effectiveness of each of the devices. In all cases, pneumatic hoses were used to collect the data. In most cases, one to two days of data were collected before and after device installation (or activation). Key words: work zone, maintenance, traffic control, speed.

INTRODUCTION

The State of Kansas ranks fourth in the country in public road mileage behind California, Texas, and Illinois. Of the more than 214,000 km (133,000 miles) of public roads in the state, the Kansas Department of Transportation (KDOT) is responsible for maintaining 15,450 km (9,600 miles). With that in mind, it should be no surprise that work zone safety is one of KDOT’s highest priorities. During 1999, KDOT joined with the DOTs from Nebraska, Iowa, and Missouri to evaluate innovative devices aimed at improving work zone safety. In Kansas, nine evaluations were conducted. For all evaluations, the devices were provided by the vendor at no cost to KDOT, and in exchange, KDOT funded the evaluation of the devices and the imminent publication of the results. The devices evaluated are shown in Table 1. The remainder of this paper contains a discussion of the data collection techniques used, followed by brief descriptions of the devices evaluated and a summary of the results from each evaluation.

<table>
<thead>
<tr>
<th>Product</th>
<th>Manufacturer</th>
</tr>
</thead>
<tbody>
<tr>
<td>SafetyCade Barricade</td>
<td>WLI Industries</td>
</tr>
<tr>
<td></td>
<td>1-800-323-2462</td>
</tr>
<tr>
<td></td>
<td><a href="http://www.wli-industries.com">www.wli-industries.com</a></td>
</tr>
<tr>
<td>Traffic Graphics Software</td>
<td>Professional Traffic Graphics</td>
</tr>
<tr>
<td></td>
<td>1-877-827-3279</td>
</tr>
<tr>
<td></td>
<td><a href="http://www.traffic-graphics.com">www.traffic-graphics.com</a></td>
</tr>
<tr>
<td>Removable Orange Rumble Strips</td>
<td>Advance Traffic Markings</td>
</tr>
<tr>
<td></td>
<td>1-252-536-2574</td>
</tr>
<tr>
<td></td>
<td><a href="http://www.trafficmarking.com">www.trafficmarking.com</a></td>
</tr>
<tr>
<td>Safety Warning System</td>
<td>MPH Industries, Inc.</td>
</tr>
<tr>
<td></td>
<td>1-800-835-0690</td>
</tr>
<tr>
<td></td>
<td><a href="http://www.mphindustries.com">www.mphindustries.com</a></td>
</tr>
<tr>
<td>Light guard RPMs</td>
<td>Lightguard Systems, Inc.</td>
</tr>
<tr>
<td></td>
<td>1-707-542-4547</td>
</tr>
<tr>
<td></td>
<td><a href="http://www.crosswalks.com">www.crosswalks.com</a></td>
</tr>
<tr>
<td>Wizard CB Alert System</td>
<td>Highway Technology/Trafcon</td>
</tr>
<tr>
<td></td>
<td>Industries, Inc.</td>
</tr>
<tr>
<td></td>
<td>1-717-691-8007</td>
</tr>
<tr>
<td>Interplex Solar Powered RPMs</td>
<td>Interplex Solar, Inc.</td>
</tr>
<tr>
<td></td>
<td>1-203-466-6103</td>
</tr>
<tr>
<td>Radar Drones</td>
<td>Speed Measurement Laboratories</td>
</tr>
<tr>
<td></td>
<td>1-800-617-4929</td>
</tr>
<tr>
<td>Speed Display</td>
<td>Speed Measurement Laboratories</td>
</tr>
<tr>
<td></td>
<td>1-800-617-4929</td>
</tr>
</tbody>
</table>

DATA COLLECTION

Three basic types of data were collected as appropriate for each application: vehicle speeds, vehicle lane positions, and lane distributions. When lane distributions were relevant, data were collected at points 152 m, 304 m, and 457 m (500 ft, 1000 ft, and 1,500 ft, respectively) upstream of the lane taper. Data were analyzed separately for passenger cars and trucks, as well as for daylight and nighttime conditions. Data collection periods were limited to one or two days before and after device installation, except for the radar drones and the speed display, for which a week of data was collected before and after deployment. All data collection was performed using pneumatic hoses. In order to remove the effects of platooning, only records with an associated headway of 5 seconds or more were considered, based on the Highway Capacity Manual’s recommendations for estimating percent time delay.
VERTICAL SAFETYCADE

The original SafetyCade Type II barricade was developed through the SHRP Program. The benefits of the SafetyCade over conventional barricades include better visibility, more positive guidance, greater portability, and improved recoverability. The product evaluated was a version of the SafetyCade, called the Vertical SafetyCade, designed to replace the standard reflectorized drum. Approximately half the width of the original version, this version is particularly applicable to sites where limited real estate is available. The benefits of this product over drums are similar to the benefits afforded by its predecessor. The collapsible frame allows the barricade to simply fold flat when hit by a passing vehicle. To restore the device, the main panel need only be brought upright, automatically locking in place. The model tested contained a single panel sign with a black-on-orange chevron. The chevron is intended to provide more positive guidance than reflectorized drums.

The SafetyCades were evaluated at the entrance to an interstate work zone, as shown in Figure 1. There were no statistically significant changes in either speeds or lane distributions, indicating that the Vertical SafetyCades were no less visible than drums. Observations of the test site by KDOT personnel before and after the deployment of the SafetyCades suggested that the positive guidance provided by the chevron panel was superior to the guidance provided by drums.

TRAFFIC GRAPHICS SOFTWARE

This product was a departure from the rest of the products evaluated. Rather than a roadside device or pavement marking, the Traffic Graphics Software is a comprehensive set of images for use in CorelDRAW, including macros, which help produce professional diagrams easily and quickly. The package was evaluated by two different areas within the DOT—the Bureau of Traffic Engineering and Public Relations. Both areas found the software easy-to-use and capable of generating complex traffic control diagrams quickly and efficiently. The traffic engineering area felt that while the software is powerful and easy-to-use, it is not necessarily superior to the CAD based software currently used for this purpose. The public relations personnel felt that this software would represent an improvement over their current methods of generating traffic control diagrams. One key difference between the two responses may be that the public affairs personnel had previous experience with CorelDRAW, whereas the traffic engineering personnel did not. The public affairs personnel felt the software was a good investment and would recommend its purchase if the decision was theirs to make. They did comment that the learning curve could be reduced by more intuitive organization of the component files. Currently, images are organized by a terminology drawn from the Manual on Uniform Traffic Control Devices (MUTCD)—a resource familiar to traffic engineers, but not to public relations personnel.

REMOVABLE ORANGE RUMBLE STRIPS

Removable orange rumble strips were tested and evaluated on a rural two-lane highway, at the approach to a work zone in which a temporary signal was in use and KDOT’s standard asphalt rumble strips were in place. The daytime data showed a statistically significant change in mean speeds and 85th percentile speeds downstream of the removable rumble strips for both passenger cars and trucks (95% confidence level). Because of the low volumes at night, analysis of data collected at night did not yield usable results.

Perhaps the primary benefit of the removable rumble strips is that they are easily installed and removed. The test installation went smoothly, and based on the test experience, three workers familiar with the installation procedures could probably install a full complement of strips in 30 minutes or less. To test the capabilities of the adhesive, no preparatory work was done to the pavement before installing the strips. While the adhesive was insufficient under the test conditions, the use of a blower to clear the installation area of loose particles would likely yield a satisfactory seal between the strips and the pavement. After two weeks, the strips showed no noticeable wear.

The thickness of the strips, 3.175 mm (125 mil), seemed insufficient to create noticeable, audible, and tactile warning to the driver, especially in trucks. However, the reductions observed in both the mean and the 85th percentile speeds indicate that the color of the strips alone is sufficient to have a positive effect.
Additionally, drivers have been observed crossing the centerline to circumvent standard asphalt rumble strips. Rumble strips that are less dramatic in their effect might serve the purpose of alerting the inattentive driver, while providing less impetus for drivers to leave their lane in an unsafe avoidance maneuver.

The qualitative analysis of the strips’ effectiveness from the driver’s perspective suggested that changes in the configuration tested could significantly improve the effectiveness of the device. The advantage afforded by the visible warning provided by the orange color of the strips was considered to be very significant by the KDOT Bureau of Traffic Engineering. KDOT is interested in conducting a subsequent evaluation in which strips with a greater thickness will be used, possibly as much as 6.35 mm (250 mil).

SAFETY WARNING SYSTEM

The Safety Warning System (SWS) is designed to inform drivers of an upcoming work zone through a message encoded in a radar signal broadcast from a trailer mounted transmitter. The system consists of two components: a transmitter that broadcasts messages encoded in a radar signal and an in-vehicle receiver capable of interpreting the messages. Many recent models of radar detectors are SWS compatible, and SWS receivers are available that do not function as radar detectors, making them legal for commercial vehicle operators. The SWS was deployed at a lane taper followed by a crossover at the entrance to a rural interstate work zone.

Speeds and lane distributions were collected at points 152 m, 304 m, and 457 m (500 ft, 1000 ft, and 1,500 ft, respectively) upstream of the lane taper, and speeds were collected at approximately the midpoint of the initial curve of the crossover. Data collection difficulties rendered the speed data prior to the taper unusable. Lane distributions showed no change with the deployment of the SWS transmitter. However, speeds within the crossover did show a statistically significant decrease (95% confidence level).

LIGHTGUARD LIGHTED RAISED PAVEMENT MARKERS

The Lightguard lighted raised pavement markers (RPMs) were deployed in the crossover at the same work zone entrance where the SWS was evaluated. One day of data was collected before the SWS was switched on, and a second day of data was collected before the Lightguard RPMs were lit. As mentioned previously, mean speeds decreased with the deployment of the SWS. When the Lightguard system was turned on (the SWS remaining active), an additional decrease in mean and 85th percentile speeds occurred. The change was statistically significant at a 95% confidence level for both trucks and passenger cars during both daylight and darkness conditions, though the more dramatic change occurred at night. The percentage of passenger cars passing within 30 cm (1 ft) of the edgeline decreased from 8.9 to 5.2 percent with the deployment of the RPMs, indicating that drivers were keeping closer to the center with the RPMs active. Percent changes for passenger cars at other distances from the edge line and percent changes for trucks at all distances from the edge line were not statistically significant at a 95% confidence level. Figure 2 shows the crossover delineated by the Lightguard RPMs. For comparison, Figure 3 shows another crossover in the same work zone where no lit delineators were installed.

WIZARD CB ALERT SYSTEM

The Wizard CB Alert System is a device intended to provide advance warning of work zone conditions to travelers via messages broadcast over CB channel 19. The device was deployed at a lane drop at the entrance to a typical interstate construction zone. Lane distributions showed no change with the deployment of the CB Wizard. Low traffic volumes and excellent visibility may have rendered this a poor site for evaluating the effectiveness of this device. KDOT is still interested in using this device in the future.
INTERPLEX SOLAR POWERED RAISED PAVEMENT MARKERS

The Interplex solar powered raised pavement markers were used to delineate a lane taper at an Interstate work zone entrance. The advantage of these RPMs is that they are easily installed (self-adhering) and require very little maintenance, being solar powered. However, the configuration evaluated was insufficient to impact driver behavior. No change occurred in lane distributions with the deployment of the RPMs. Compared to the Lightguard system, which requires a hard-wired power source, the Interplex RPMs were much less visible. Operating the RPMs in a flashing mode rather than steady burn might improve their effectiveness. Additionally, deploying more units with smaller spacings might provide better delineation. The ease of installation and low maintenance are noteworthy benefits, but a more effective configuration must be developed before these can be recommended for delineation of lane drops.

RADAR DRONES

Radar drones are intended to trigger radar detectors, causing those drivers to reduce their speed. Assuming that drivers using radar detectors tend to travel faster than the mean, this would reduce not only the mean speed but also the variation in speeds. Two radar drone units were deployed within a work zone approximately 1.6 km (1 mi) apart. Speeds were collected for four days prior to the deployment of the drones and for four days after deployment. Speeds were collected at a total of 10 points between the drones units. Some changes in the mean and 85th percentile speeds were observed, but no consistent pattern existed. One difficulty encountered was that the tractor batteries used to power the drones (one battery each) proved insufficient to maintain operation for the intended test period of one week. While the batteries were recharged during the week, it is suspected that the units were operational for only about half of the time during which the data was being collected. Consequently, no conclusions could be made regarding the effectiveness of the device from this evaluation.

SPEED DISPLAY

Following the radar drone evaluation, a radar-triggered speed display was deployed at the same site. The speed display evaluated comprised a back lit dynamic speed display, a standard speed limit sign posted above the display, and a strobe flash, all contained in a trailer mount. The strobe flash was set to activate when a vehicle’s speed exceeded 64 mph. During the operation of the speed display, statistically significant reductions occurred in mean speeds for both passenger cars and trucks during daylight and nighttime conditions. Near the device, the mean speed of passenger cars was reduced from 100.3 kph (62.3 mph) to 95.8 kph (59.5 mph), and the percent of passenger cars exceeding the posted speed of 60 mph (about 97 kph) dropped from 67% before the deployment of the device to 36% afterwards. Approximately 0.8 km (.5 mi) downstream from the device, the mean speed was reduced to 98.8 kph (61.4 mph), and the percent speeding was reduced to 60%. Compared to the effects on passenger cars, the effects on trucks were somewhat more pronounced at night and slightly less pronounced during the day. All reductions were statistically significant at a 95% confidence level.

CONCLUSIONS

All the devices evaluated showed potential for improving the safety and efficiency with which highway maintenance is performed, even though several caused no statistically significant change in the relevant quantitative evaluation parameters. The devices that were the most effective based on the quantitative data collected were the speed display and the Lightguard RPMs. Other devices whose effectiveness was not strongly reflected in the data, but which seemed to generate significant interest among KDOT traffic control personnel based on their own qualitative observations, were the SafetyCade Barricade and the orange removable rumble strips. All of these devices are commercially available, although the configuration used with the Lightguard RPMs was experimental.
The speed display is easily deployed, very mobile, and highly effective at reducing speeds. Speed reductions resulting from the deployment of the Speed Display were comparable to those occurring during active law enforcement.

The Lightguard RPMs resulted in substantial reductions in speeds and improvements in lane placement. Several enhancements might further improve the effectiveness of the RPMs, including a random flash mode for daytime operation and a sequenced flash (chasing) mode for nighttime operation.

The Vertical SafetyCade did not produce any significant changes in speeds or lane distributions. The chevron panel provides more positive guidance than standard reflectorized drums, which could result in reducing instances of vehicles encroaching on the work area.

The orange removable rumble strips did produce statistically significant reductions in speeds for both passenger cars and trucks, although the reductions were small. The potential for the strips to improve driver attention to the driving task while approaching the work zone is very promising. To maximize their effectiveness, the evaluated configuration should be altered and thicker strips should be developed or a double thickness used (i.e., one stripe on top of another).

REFERENCES

Evaluation of Two Strategies for Improving Safety in Highway Work Zones

ERIC MEYER

During 1999, the Departments of Transportation from the states of Kansas, Nebraska, Iowa, and Missouri conducted a pooled-fund study of innovative devices designed to improve the safety and efficiency with which highway maintenance is conducted. In the state of Kansas, a total of nine devices were evaluated. This paper discusses the two devices that showed the greatest potential for improving the safety of highway work zones, a radar-triggered speed display and Lightguard lighted raised pavement markers (RPMs). The devices are described as they were evaluated, and the results are discussed with respect to the effectiveness of the devices relative to the current practice in Kansas. The speed display was also compared directly with active law enforcement at the same site. Speeds were used as a measure of effectiveness for both devices. Lane position was also used to evaluate the effectiveness of the Lightguard RPMs, which were used to delineate a crossover. In all cases, pneumatic hoses were used to collect the data. Data were collected for four days before and four days after the deployment of the speed display. Only one day of data was collected before and after activation of the RPMs. Both devices produced significant reductions in mean and 85th percentile speeds (statistically and practically significant). The RPMs resulted in a reduction in the percentage of passenger cars tracking within 30 cm (1 ft) of the edge line. The reduction was statistically significant at a 95% confidence level, though practical significance is difficult to assess in this case. Both devices were evaluated at rural interstate work zones. Further evaluation is needed to determine to what extent, if any, the effects of the devices decrease over time in a context with a high percentage of repeat traffic, such as an urban freeway. Key words: work zone, maintenance, traffic control, speed.

INTRODUCTION

The Midwest Smart Work Zone Deployment Initiative is a pooled-fund study, initiated in 1999, involving the four states of Nebraska, Iowa, Missouri, and Kansas. The purpose of the study is to identify and evaluate innovative technologies applied to making highway work zone operations safer and more efficient for the traveling public, as well as maintenance workers. With more than 214,000 km (133,000 miles) of public roads, the State of Kansas ranks fourth in the country behind California, Texas, and Illinois (1). Being responsible for maintaining 15,450 km (9,600 miles) of these public roads (2), the Kansas Department of Transportation (KDOT) places a high priority on work zone safety. As part of the first year of the ongoing study, nine evaluations were conducted in Kansas during 1999. For all evaluations, the devices were provided by the vendor at no cost to KDOT, and in exchange, KDOT funded the evaluation of the devices and the publication of the results. A summary of the results from all nine evaluations is available in the paper entitled “Midwest Smart Work Zone Deployment Initiative: Kansas Results” (3). This paper discusses in more detail the evaluations of two devices that showed particularly high potential for improving safety in highway work zones.

The first device discussed is a commercially available radar-triggered speed display, provided by Speed Measurement Laboratories (1-800-617-4929, www.speedlabs.com). The speed display resulted in significant reductions in mean speed, 85th percentile speed, standard deviation, and percent of drivers exceeding the posted speed. These effects diminished downstream of the device, but remained at statistically significant levels for the 0.8 km (½ mi) over which speed data was collected.

The second device discussed is a system of lighted raised pavement markers (RPMs) provided by Lightguard Systems, Inc. (1-707-542-4547, www.crosswalks.com). While the individual RPM units are commercially available, the configuration and application evaluated were experimental. The lighted RPMs resulted in a significant reduction in speeds and an improvement in lane keeping by passenger cars.

DATA COLLECTION

All data was collected using pneumatic hoses connected to automatic traffic recorders. Speeds were collected using paired hoses with a 6 m (20 ft) spacing. Raw data (i.e., time stamped axle hits) were recorded and post-processed to obtain vehicle classification and per-vehicle speed data. Data was analyzed separately for passenger cars and trucks and for daylight and nighttime conditions. In order to remove the effects of platooning, only records with an associated headway of 5 seconds or more were considered, based on the Highway Capacity Manual’s recommendations for estimating percent time delay (4). Throughout the analyses, statistical significance was determined using a 95% confidence level.

RADAR-TRIGGERED SPEED DISPLAY

The speed display evaluated comprised a back-lit dynamic speed display, a standard speed limit sign posted above the display, and a strobe flash, all contained in a trailer mount. The strobe flash was set to activate when a vehicle’s speed exceeded 103 kph (64 mph). A second threshold speed could be set that activated an alarm horn. The horn would sound toward the construction zone to alert workers that a vehicle was approaching at a potentially reckless speed. A maximum speed could also be set for the display, discouraging drivers from competing to post higher speeds on the display. Only the strobe threshold was set for the evaluation period. The device is bulletproof to withstand substantial vandalism attempts. The device is camera-ready to allow
photo enforcement, although no camera was used in the evaluation (Photo enforcement is at this time prevented by state statute. In order for a citation to be issued, an offense must be witnessed by a law enforcement officer present at the time of the offense.).

**Test Site**

The evaluation was conducted in an 8 km (5 mile) construction zone on I-70 approximately 44 km (30 miles) west of Topeka, Kansas. The test was conducted using eastbound traffic during the second phase of a reconstruction project in which the eastbound lanes were closed, and two-way traffic was being carried in the westbound lanes. Originally, data was to be collected at ten locations in the vicinity of the device. Equipment failures resulted in usable data being obtained from only four of the collection points during the time the speed display was operating.

**Data Collection**

Prior to the deployment of the speed display, a week of baseline data was collected, followed by a week in which radar drones were deployed and more data collected. The week following the deployment of the speed display, the Kansas Highway Patrol (KHP) provided active speed enforcement for a total of 8 hours, recording the times during which an officer was present so that the corresponding data could later be identified. A fifth data set was included in the analysis, comprising the hour immediately following the departure of the KHP.

**Results**

Figures 1 and 2 show the speed distributions at data point 7 during the day, for passenger cars and for trucks, respectively. Figures 3 and 4 show similar data for data point 4. Vehicles passed over data...
points in reverse order, i.e., data point 10 was the farthest upstream of the data points, while data point 1 was farthest downstream. The speed display was deployed near data point 7 at a median crossing. KHP locations were not recorded, but it is likely they were observing from the same location, because the shoulders were not suitable for parking and the median crossing near data point 7 would be the best location from which to observe traffic.

The radar drone showed little or no effect on speeds or on the percent of drivers exceeding the posted limit. In all cases, the speed display resulted in a significant reduction in mean speeds, 85th percentile speeds, percent of drivers exceeding the posted limit, and speed variation (standard deviations). While not appearing in Figures 1 and 2 due to equipment problems, data from data point 8 showed the impact of law enforcement on speeds to be almost identical to the impact of the speed display. However, data during the hour following the KHP’s departure from the test site (i.e., those labeled “Post-Law Enforcement” in Figures 1 through 4), showed that speeds not only increased to normal, but exceeded baseline speeds. In Figures 3 and 4, however, it can be seen that during periods of active law enforcement, speeds were above the baseline, and rose yet higher following the KHP’s departure.

Conclusions

From the data collected, it is reasonable to conclude that the radar drones are not effective devices for reducing speed-related traffic characteristics. The radar-triggered speed display was quite effective, reducing mean speeds, 85th percentile speeds, percent of drivers exceeding the posted limit, and standard deviations for both cars and trucks. The effects were less pronounced, but still significant, at data point 4, which is approximately 0.8 km (0.5 mi) downstream of the speed display. The display was easily deployed and very mobile. The setup time was less than 10 minutes once the site was identified. In contrast to the effects of the radar-triggered speed display, law
enforcement appears to cause an increase in speeds downstream from the patrol car. Additionally, speeds continue to increase after the patrol car is no longer in the area. The reason for this phenomenon is unknown.

LIGHTGUARD LIGHTED RPMS

The Lightguard lighted RPMs were tested in an experimental configuration used to delineate a crossover in a rural construction zone. Amber lights were used to delineate the inside edge, placed just beyond the edge line, and white lights were used to delineate the outside edge, also placed just beyond the edge line. The lights operated in a steady-burn mode.

Test Site

The evaluation occurred at the westbound entrance to a rural interstate work zone on I-70 approximately 16 km (10 mi) east of Salina, Kansas. In the work zone, the westbound lanes were closed, and two-way traffic was being carried in the eastbound lanes. The Safety Warning System (SWS) was deployed at the lane taper on the westbound lanes, preceding the crossover where the RPMs were installed. The pavement in the crossover is 5.5 m (18 ft) wide with edge lines inset by 0.3 m (1 ft).

Data Collection

One day of lane distribution data was collected upstream of the taper, then the SWS was activated approximately 0.8 km (0.5 mi) upstream of the crossover. After data was collected for one day, the RPMs were activated and another day of data was collected. Two evaluation measures were used to evaluate the RPMs. The first measure focused on speeds (mean and 85th percentile) and the second measure on lane-keeping. The data was collected on the same schedule described for the lane distributions. To measure lane-keeping (and speeds, in the process), a configuration of hoses was set out as shown in Figure 5. Each of the short hoses detected a vehicle’s encroachment into the area near the edge line, and the distance was determined by the distance the hose extended inside the edge line. The hoses were configured to count vehicles that tracked within 0.9 m (3 ft), 0.6 m (2 ft), and 0.3 m (1 ft) of the edge line, as well as those that crossed the edge line. Both edge lines were observed, requiring a total of 16 hoses and 4 counters for the full array. The A and B inputs for each counter were used to measure speeds, while the C and D inputs were used to track vehicle positions. Software was developed to use the speeds and times to match individual vehicles in the data sets produced by the four counters.

Results

During the first and third days of data collection, 16,856 vehicles were recorded. The lane-keeping data shows that only 11 vehicles actually crossed the edge line on the inside, and none crossed the outside edge line. Only 7 vehicles tracked within 0.9 m (3 ft) of the outside edge line. The only change that was significant at a 95% confidence level was that the percent of passenger cars tracking within 0.3 m (1 ft) of the inside edge line decreased from 8.9% to 5.2% after the activation of the RPMs.

The speed data revealed more dramatic effects. Figure 6 shows the speed distributions (in the crossover) for the three days of data collection (passenger cars). The distributions for trucks were very similar. For both passenger cars and trucks, the nighttime mean speed dropped by more than 10 kph (6 mph). The percent of drivers exceeding the posted limit decreased from 29% to 22% with the activation of the SWS, compared to 7% with the activation of the RPMs. The percentages for trucks were 25%, 23%, and 6%, respectively.

Conclusions

The RPMs resulted in a statistically significant improvement in lane-keeping among passenger cars. Other changes were not statistically significant. A much more dramatic effect was observed in the speed-related parameters, whose values decreased sharply with the activation of the RPMs. Because the installation was experimental, no conclusions can be made about either the required effort for installation, maintenance, or removal. The RPM units themselves were easily installed and removed, but the cabling necessary to power the lights could be an obstacle in some locations.

SUMMARY AND FUTURE RESEARCH

Speed reductions resulting from the deployment of the radar-triggered speed display were comparable to those occurring during active law enforcement. However, the speed reduction resulting from
the activation of the speed display propagated downstream to the last operational data collection point, while speeds actually increased at the same location during the periods of active law enforcement. The portability of the device, the ease of setup, and the sturdy construction are significant advantages. Ongoing tests in Texas and a planned test in Kansas during 2001 will further evaluate the effectiveness of this device, focusing on aspects such as the distance over which the speed reductions deteriorate and potential enhancements to the display such as complimentary signing.

The Lightguard RPMs resulted in substantial reductions in speeds and improvements in lane placement. Several enhancements might further improve the effectiveness of the RPMs, including a random flash mode for daytime operation, and a sequenced flash (chasing) mode for nighttime operation. Alternate colors are available, though colors other than the amber and white used in this evaluation might be considered a departure from Manual on Uniform Traffic Control Devices (MUTCD) guidelines. The system may be applicable to other situations, such as lane tapers.

Both evaluations were relatively short in duration. In the rural context in which they were conducted (i.e., very little repeat, or commuter, traffic), the results are probably typical of what could be expected in long-term deployments. However, further investigation is needed to examine the effectiveness of the devices in an urban context over longer periods of time.

In general, both evaluations evidenced strong potential for improvements in work zone safety through the deployment of the respective devices. Further research will likely be able to increase the benefits by improving both the effectiveness and the ease of deployment.

REFERENCES

The Mobility and Safety Impacts of Winter Storm Events in a Freeway Environment

KEITH K. KNAPP, LELAND D. SMITHSON, AND AEMAL J. KHATTAK

This paper describes how data from several Iowa information management systems were used to analyze the mobility and safety impacts of winter storm events. Roadway and weather data were acquired from the roadway weather information system (RWIS), hourly traffic volumes from automatic traffic recorders (ATRs), and crash information from the accident location and analysis system (ALAS). Daily snowfalls were acquired from state and national agencies. Storm and non-storm data for seven interstate roadway segments were considered. Only winter storm events with a duration of four or more hours and a snowfall of 0.51 centimeters per hour (0.20 inches per hour) or more were evaluated. Analysis of the data revealed the impacts of winter weather on freeway traffic. Winter storm events decrease traffic volumes, but the impact is highly variable. The average winter storm volume reduction was approximately 29 percent, but ranged from approximately 16 to 47 percent. A positive relationship was found between percent volume reduction, total snowfall, and the square of maximum gust wind speed. Crash rates also significantly increase during winter storm events, possibly the result of a large decrease in traffic volumes and higher crash reporting rates during winter weather. After controlling for exposure, an increase in snowfall intensity and snowstorm duration also increased winter storm event crash frequency. The results of this research can help determine the potential impacts of winter weather, support the eventual development of a dynamic winter weather driveability level of service system, and assist with planning preventive and emergency operations. Key words: winter weather, mobility, safety, volume.

INTRODUCTION

Traffic volume and safety along a roadway segment is a function of a number of factors (e.g., heavy vehicle percentages, lane widths, etc.). One of these factors is weather. Engineering designs and maintenance attempt to minimize the impacts of weather on traffic, but each year winter storm events impact mobility and safety. This research used data from several Iowa information management systems to evaluate winter weather impacts on traffic volume and safety.

LITERATURE REVIEW

Weather and Volume/Travel Decisions

Hanbali and Kuemmel have investigated winter storm volume reductions (1), using traffic volume and weather data from at least the first three months of 1991 at 11 locations in four states. Traffic volume reductions were calculated for different ranges of total snowfall, average daily traffic, roadway type, time of day, and day of the week (1). Overall, the reductions ranged from 7 to 56 percent (1). The researchers concluded that volume reductions increased with total snowfall, but that the reductions were smaller during peak travel hours and on weekdays (1). A 1977 Federal Highway Administration (FHWA) study had similar findings (2).

Weather and Safety

Several researchers have explored the relationship between adverse weather and safety (3, 4, 5, 6, 7, 8). For example, Hanbali found a significant decrease in crash rates before and after deicing maintenance activity (3), and the results of several Swedish studies have supported these findings (4, 5, 6). The Swedish studies also indicate that severe injury rates on roads with snow and ice can be several times greater than non-winter roadways (4, 5, 6). Perry and Symons also found that total injuries and fatalities increased by 25 percent on snowy days, and the rate of injuries and fatalities increased by 100 percent (7). A Canadian study, on the other hand, reported that winter months (December to March, inclusive), when compared to summer months, had higher minor and material damage accident rates but lower severe and fatal crash rates (4). A 1977 FHWA study had similar findings but found increased severe injury crash rates in snowbelt states when compared to the non-snowbelt states during winter months (8).

DATA COLLECTION

This project used data from the roadway weather information system (RWIS), automatic traffic recorders (ATRs), the accident location and analysis system (ALAS), and the Iowa Department of Agriculture and Land Stewardship (IDALS)/National Weather Service (NWS). Roadway and/or weather data from Iowa RWIS stations and the IDALS/NWS, crash data from ALAS, and hourly traffic volumes from Iowa interstate ATRs were linked. The data were acquired for winter storm event and comparable non-storm event time periods.

Seven RWIS sites along the interstate in Iowa were analyzed. All the RWIS stations had a nearby ATR, and the hourly volumes collected at these ATRs were used to approximate storm and non-storm event traffic volumes adjacent to the RWIS station. The location of the seven RWIS/ATR pairs are shown in Figure 1. Bi-directional ATR hourly traffic volumes were acquired for 1995, 1996, 1997, and 1998. The volume data was not used in this research if it was...
estimates (due to an ATR malfunction) or was measured on a day near a holiday (i.e., a non-typical travel day).

Weather and roadway data from the RWIS stations (See Figure 1) and daily snowfall information from IDALS/NWS observer sites were used to define, identify, and determine the time periods when winter storm events most likely occurred. RWIS and IDALS/NWS data from all or part of the 1995/1996, 1996/1997, and 1997/1998 winter seasons were acquired. In general, winter storm event time periods were defined by those hours when the RWIS stations recorded all the following: 1) precipitation occurring, 2) air temperature below freezing, 3) wet pavement surface (indicated at any of the pavement sensors at the site), and 4) a pavement temperature below freezing (indicated at all of the pavement sensors at the site). Any two winter storm events separated by only one "non-storm" hour were combined. In addition, this research only considered those winter storm event time periods that had a duration of at least four hours and an estimated snowfall intensity (from nearby IDALS/NWS information) of 0.51 centimeters per hour (0.20 inches per hour). The goal was to limit the research analysis to relatively significant winter storm events.

This research compared and statistically analyzed volume and crash data from winter storm and non-storm event time periods. For example, Figure 2 shows the hourly traffic volumes observed at the Jewell, Iowa ATR during a winter storm event on Saturday, April 12, 1997. Figure 2 also shows the average Saturday daily traffic flow profile for April 1997. As expected, the average volume during the winter storm event is at or below the average volume of the non-storm traffic flow profile. If possible, this type of comparison was completed, along with a similar storm/non-storm crash comparison, for each of the winter storm events defined.

WINTER STORM EVENT IMPACT ANALYSIS

Volume Analysis

Overall, 64 winter storm events, encompassing 618 hours, were defined for the traffic volume analysis. Some descriptive statistics of the winter storm event percent volume reductions are shown in Table 1.

Table 1 shows large variability in winter storm event traffic volume impacts. The average storm event volume reduction (by location) ranges from approximately 16 (n = 10) to 47 percent (n=6), and the overall average volume reduction is approximately 29 percent. The variability is shown by the fact that the standard deviation of the percent volume reduction at each RWIS location is close to the average percent volume reduction. The 95 percent confidence interval for the overall average percent volume reduction is 22.3 to 35.8 percent.

Regression analysis (assuming a normal distribution of the data) was used to investigate the relationships between percent volume reduction (the dependant variable) and storm event duration, snowfall intensity and total snowfall, minimum and maximum average (during a one-minute period) wind speed, and maximum gust wind speed (maximum four-second wind speed during a one-minute time period). The regression analysis indicated that percent volume reduction has a statistically significant relationship with total snowfall and the square of maximum gust wind speed. The other variables were either correlated with these two variables or were not found to have a statistically significant relationship with percent volume reduction. The results of this regression analysis are shown in Table 2. The model coefficients indicate that percent volume reduction in-

FIGURE 1 Data collection sites selected
FIGURE 2  Average Saturday traffic flow profile (April 1997) and winter storm event (April 12, 1997) volumes

TABLE 1  Winter Storm Event Traffic Volume Summary

<table>
<thead>
<tr>
<th>Interstate RWIS Location</th>
<th>Number of Storm Events</th>
<th>Storm Event Hours</th>
<th>Average Storm Event Volume Reduction (Percent)</th>
<th>Std. Dev. Storm Event Volume Reduction (Percent)</th>
<th>Min. Storm Event Volume Reduction (Percent)</th>
<th>Max. Storm Event Volume Reduction (Percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#133 – I-235, Des Moines</td>
<td>8</td>
<td>83</td>
<td>36.4</td>
<td>30.5</td>
<td>13.0</td>
<td>86.5</td>
</tr>
<tr>
<td>#512 – I-35, Ames</td>
<td>10</td>
<td>82</td>
<td>15.5</td>
<td>13.7</td>
<td>1.4</td>
<td>46.9</td>
</tr>
<tr>
<td>#606 – I-380, Cedar Rapids</td>
<td>4</td>
<td>70</td>
<td>23.7</td>
<td>18.9</td>
<td>0.8</td>
<td>40.0</td>
</tr>
<tr>
<td>#615 – I-80, Grinnell</td>
<td>6</td>
<td>71</td>
<td>46.9</td>
<td>46.2</td>
<td>-42.1</td>
<td>84.3</td>
</tr>
<tr>
<td>#619 – I-35, Mason City</td>
<td>12</td>
<td>79</td>
<td>19.1</td>
<td>20.1</td>
<td>-1.9</td>
<td>71.6</td>
</tr>
<tr>
<td>#620 – I-80, Adair</td>
<td>10</td>
<td>107</td>
<td>35.3</td>
<td>30.8</td>
<td>-8.0</td>
<td>91.5</td>
</tr>
<tr>
<td>#624 – I-35, Leon</td>
<td>14</td>
<td>126</td>
<td>32.5</td>
<td>23.1</td>
<td>5.5</td>
<td>80.8</td>
</tr>
<tr>
<td>Overall</td>
<td>64</td>
<td>618</td>
<td>29.1</td>
<td>26.7</td>
<td>-42.1</td>
<td>91.5</td>
</tr>
</tbody>
</table>

1 Negative volume reductions indicate an increase in volumes. Overall, three of the storm events defined had negative volume reductions.

Table 2 Regression Analysis Results

(Dependant Variable: Percent Winter Storm Event Volume Reduction)

<table>
<thead>
<tr>
<th>Explanatory Variable</th>
<th>Coefficient</th>
<th>T-Statistic</th>
<th>P-Value</th>
<th>Mean of Variable</th>
<th>Std. Dev. of Variable</th>
<th>Range of Variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Snowfall (centimeters)</td>
<td>0.9010</td>
<td>2.16</td>
<td>0.035</td>
<td>9.562</td>
<td>6.038</td>
<td>2.67 to 27.51</td>
</tr>
<tr>
<td>Max. Gust Wind Speed² (kph³)</td>
<td>0.01143</td>
<td>6.87</td>
<td>0.000</td>
<td>1925.08</td>
<td>1513.93</td>
<td>93.32 to 7558.56</td>
</tr>
<tr>
<td>Constant</td>
<td>-1.582</td>
<td>-1.34</td>
<td>0.730</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

²kph = kilometers per hour, 1 centimeter = 0.39 inches, 1 kilometer = 0.62 miles

Model Summary Statistics: Number of Observations = 64  Mean Square Error = 332  F-Value = 38.65  Coefficient of Multiple Determination = R-Squared = 0.559  P-Value = 0.000  R-Square (Adjusted) = 0.544
Table 3 Summary of Snowstorm Data

<table>
<thead>
<tr>
<th>Sample Statistic</th>
<th>Crash Frequency (crashes/storm)</th>
<th>Storm Duration (hrs)</th>
<th>Traffic Volume (veh)</th>
<th>Snow Intensity (cms/hr)</th>
<th>Max Wind Speed (kmph)</th>
<th>Min Avg. Wind Speed (kmph)</th>
<th>Max Avg. Wind Speed (kmph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>2.00</td>
<td>9.09</td>
<td>7063.70</td>
<td>1.07</td>
<td>37.54</td>
<td>12.52</td>
<td>28.92</td>
</tr>
<tr>
<td>Std. Error</td>
<td>0.47</td>
<td>0.53</td>
<td>1502.06</td>
<td>0.07</td>
<td>1.98</td>
<td>1.27</td>
<td>1.56</td>
</tr>
<tr>
<td>Std. Deviation</td>
<td>2.43</td>
<td>0.89</td>
<td>11037.86</td>
<td>0.53</td>
<td>14.58</td>
<td>9.36</td>
<td>11.43</td>
</tr>
<tr>
<td>Variance</td>
<td>11.74</td>
<td>13.45</td>
<td>121834416.51</td>
<td>0.28</td>
<td>212.53</td>
<td>87.61</td>
<td>130.68</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.00</td>
<td>0.00</td>
<td>231.00</td>
<td>0.51</td>
<td>0.00</td>
<td>0.00</td>
<td>2.54</td>
</tr>
<tr>
<td>Maximum</td>
<td>17.00</td>
<td>19.00</td>
<td>61910.00</td>
<td>2.54</td>
<td>66.01</td>
<td>33.81</td>
<td>54.74</td>
</tr>
<tr>
<td>Sum</td>
<td>108.00</td>
<td>491.00</td>
<td>381440.00</td>
<td>57.68</td>
<td>2026.99</td>
<td>676.20</td>
<td>1561.70</td>
</tr>
<tr>
<td>Count</td>
<td>54.00</td>
<td>54.00</td>
<td>54.00</td>
<td>54.00</td>
<td>54.00</td>
<td>54.00</td>
<td>54.00</td>
</tr>
</tbody>
</table>

1 Conversions: 1 centimeter = 0.39 inches and 1 kilometer = 0.62 miles.

The overall winter storm event crash rate (n = 54) was calculated to be 5.86 crashes per million-vehicle-kilometers (mvkm). Note, however, that the traffic volumes recorded at the nearby ATR station do not represent actual traffic volumes along the entire 48 km-long (30 mile) highway section under investigation. Therefore, the crash rates reported in this paper do not represent the actual crash rate for the interstate sections of interest and should only be used for comparison purposes. The overall non-storm crash rate was calculated to be 0.41 crashes per mvkm (based on the same assumption as stated above). The difference in crash rates between storm and non-storm event time periods was approximately 1,300 percent, indicating a very significant change.

A Poisson regression modeling approach was used to analyze the reported number of crashes (9). The winter storm event crash frequency was the dependent variable, and the independent variables included exposure (the product of section length (km) and traffic volume during the winter storm events) in million-vehicle-kilometers, snowfall intensity, maximum wind gust speed, maximum average wind speed during the snowstorm, and minimum average wind speed during the snowstorm. Table 5 shows the Poisson modeling results. The model indicates significantly positive coefficients for exposure and snowfall intensity. In other words, an increased exposure and snowfall intensity during winter storm events increases crash frequency, but the model also indicates that snowstorm duration has an additional effect besides that captured by the exposure term.

Table 5 Poisson Model Results

<table>
<thead>
<tr>
<th>Explanatory Variable</th>
<th>Coefficient</th>
<th>T-statistic</th>
<th>Marginal Values</th>
<th>Mean of Explanatory Variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure (mvkm)</td>
<td>0.682</td>
<td>6.148</td>
<td>0.832</td>
<td>0.341</td>
</tr>
<tr>
<td>Snowstorm duration (hrs)</td>
<td>0.156</td>
<td>5.826</td>
<td>0.190</td>
<td>9.093</td>
</tr>
<tr>
<td>Snowfall intensity (cms/hr)</td>
<td>0.494</td>
<td>2.226</td>
<td>0.603</td>
<td>1.068</td>
</tr>
<tr>
<td>Max wind gust speed (kmph)</td>
<td>0.009</td>
<td>1.311</td>
<td>0.010</td>
<td>37.540</td>
</tr>
<tr>
<td>Constant</td>
<td>-2.315</td>
<td>-5.142</td>
<td>-2.826</td>
<td>-</td>
</tr>
</tbody>
</table>

1 Conversions: 1 centimeter = 0.39 inches and 1 kilometer = 0.62 miles.

Model Summary Statistics: Number of observations = 54. Log likelihood function \( L(\beta) = -84.314 \), Restricted Log likelihood \( L(0) = -151.546 \), \( \rho^2 = 1 - \frac{L(\beta)}{L(0)} = 0.443 \).
SUMMARY OF FINDINGS

- The 64 winter storm events used in the traffic volume analysis reduced volumes by an average of approximately 29 percent, but the reduction was relatively variable. The 54 winter storm events used in the crash analysis had an overall crash rate of 5.86 mvkm compared to a non-storm crash rate of 0.41 mvkm. A difference of approximately 1,300 percent.

- The traffic volume regression analysis indicates a significant relationship between percent winter storm event volume reduction, total snowfall, and the square of maximum gust wind speed. The crash regression analysis found a significant relationship between winter storm event crash frequency, exposure (the product of section length and volume), and snowfall intensity.

- Several factors could be responsible for the difference between the non-storm and snowstorm crash rates. First, the winter storm event definition used in this study represents relatively severe weather conditions under which the likelihood of crashes could be very high. Second, under such severe weather conditions and extended snowstorm durations traffic volumes tend to reduce appreciably. With substantially reduced traffic volumes, the occurrence of only a few crashes can result in substantial crash rates. Third, there could be a bias in crash reporting during snowstorms compared to non-storm conditions. Crashes are more likely to be reported during snowstorms compared to non-storm conditions because adverse weather conditions may necessitate a call for help by crash victims.

- A combination of the results found in this research and comparable winter weather vehicle speeds could eventually be used to determine a winter weather level of service. Relationships between volume, speed, and weather/roadway conditions would need to be defined and/or established. The speeds for specific roadway and/or weather conditions might be acquired from past research, ATRs, and/or possibly the application of video-based data collection equipment. Speed and volume data would need to be collected, archived, and weather/roadway conditions defined and correlated with these traffic flow characteristics.

ACKNOWLEDGMENT

The authors wish to thank the Iowa Department of Transportation and the Iowa Highway Research Board for financial support. The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Iowa Department of Transportation or the Iowa Highway Research Board (TR-426). The discussion in this paper represents a partial summary of the work completed for a project entitled the Safety and Mobility Impacts of Winter Storm Events in a Freeway Environment.

REFERENCES

Operational Analysis of Terminating Freeway Auxiliary Lanes with One-Lane and Two-Lane Exit Ramps: A Case Study

RALPH A. BATENHORST AND JEFF G. GERKEN

This paper summarizes the findings of a case study on the operational analysis of weaving areas created by auxiliary lanes between two successive interchanges. For auxiliary lanes less than 1,500 feet in length, AASHTO lane balance principles permit the termination of the auxiliary lane with a one-lane exit ramp. For auxiliary lanes greater than 1,500 feet in length, the lane balance principles require that the auxiliary lane be dropped with a two-lane exit ramp or tapered into the through roadway downstream of a one-lane exit ramp. The operational analyses of the case study were conducted as part of a Major Investment Study (MIS) in Dallas, Texas. As part of the study, auxiliary lanes were recommended at various locations along two major freeway corridors. At twenty of these locations, additional analyses were conducted to compare the quality-of-service provided by a one-lane exit ramp versus a two-lane exit ramp. The range of traffic and geometric conditions among the twenty sites varied. The analyses were conducted using three software packages: the Highway Capacity Software (HCS), CORSIM and Synchro/Simtraffic. The findings of the case study suggest that a one-lane exit ramp may provide the best traffic operations regardless of weaving length. The experience gained from the case study is presented to aid practitioners in the design of safe and efficient freeway facilities and to aid researchers in current and future efforts to define and understand the operational effects of geometric design. Key words: traffic operations, simulation, lane balance, auxiliary lanes, weaving.

INTRODUCTION

Few would argue that the urban freeway corridors throughout the United States are becoming increasingly congested. Inadequate capacity, substantial traffic volume growth, aging infrastructure, and the presence of nonstandard design features are all contributing factors to this growing problem.

The challenge ahead for today’s engineers and planners is amplified by lessons learned that we cannot always build our way out of a problem. Right-of-way constraints, funding limitations and the public’s growing sensitivity to the environmental impacts of roadway projects have forced the transportation industry to do more with less. For freeway systems, this means less dependence on expensive widening projects and a greater emphasis on managing demand and implementing cost-effective improvements to eliminate bottlenecks. Auxiliary lanes and the principles of lane balance are excellent examples of the latter. They play an important role in the ability of a freeway system to efficiently and safely accommodate higher traffic volumes without the addition of basic freeway lanes.

GUIDELINES FOR AUXILIARY LANES

The American Association of State Highway and Transportation Officials (AASHTO) (1) defines an auxiliary lane as the portion of the roadway adjoining the traveled way for parking, speed change, turning, storage for turning weaving, truck climbing, and other purposes supplementary to through-traffic movements. In a freeway environment, auxiliary lanes may be provided downstream of an entrance ramp to accommodate merging traffic, upstream of an exit ramp to accommodate diverging traffic, or between two closely spaced interchanges to accommodate weaving traffic. In addition, auxiliary lanes may be carried through one or more interchanges to serve one or more of the listed purposes.

This paper focuses on auxiliary lanes between two successive interchanges. Under these conditions, the auxiliary lane serves both as an acceleration lane for entering traffic and as a deceleration lane for exiting traffic. The auxiliary lane is typically added with a single entrance ramp lane while the termination of the auxiliary lane is subject to the principles of lane balance.

PRINCIPLES OF LANE BALANCE

To realize efficient traffic operation through and beyond an interchange, AASHTO recommends that there be a balance in the number of traffic lanes on the freeway and ramps. For auxiliary lanes between two successive interchanges, two conditions are possible:

- Condition 1: For auxiliary lanes less than 1,500 feet in length (e.g., between closely spaced interchanges or between the loop ramp entrance and the loop ramp exit of a cloverleaf interchange), the lane balance principles permit the termination of the auxiliary lane with a one-lane exit ramp as shown in Figure 1.
- Condition 2: For auxiliary lanes greater than 1,500 feet in length, the lane balance principles state that the number of approach lanes on the freeway must be equal to the number of lanes on the exit, less one.

Under Condition 2, the auxiliary lane may be terminated by one of two methods. The first method, shown in Figure 2, drops the auxiliary lane with a two-lane exit. In this configuration, traffic in the auxiliary lane must exit. Traffic in the basic lane to the left of the auxiliary lane may exit or may proceed along the mainline. The second method, shown in Figure 3, provides a one-lane exit ramp, but carries the auxiliary lane through the exit before it is tapered into the through roadway. This design provides a recovery lane for drivers who inadvertently remain in the discontinued lane.
From personal observations, the application of AASHTO guidelines regarding auxiliary lanes and lane balance seems to vary from state-to-state. This variation is due, in part, to interpretation of the guidelines by agency/consultant staff, differences in the driver characteristics and driving environments that each state must provide for, and the lessons learned from past experiences. Even in cases where AASHTO guidelines are applied consistently, such decisions are oftentimes made during final design activities when it is too late for the appropriate consideration of the operational effects of design decisions.

**OPERATIONAL ANALYSIS – PAST, CURRENT AND FUTURE RESEARCH**

The methodologies for analyzing freeway weaving sections that are contained in Chapter 4 of the 1997 Highway Capacity Manual (HCM) (2) are based on research conducted in the early 1960s through the early 1980s (3-7). Recent research on basic freeway sections (8) and ramp junctions (9) has been incorporated but has not resulted in significant changes to the methodology.

In the HCM procedures, the configuration of the weaving section is the critical geometric condition affecting the quality of weaving operations. Three types of configuration (A, B and C) are defined based on the minimum number of lane changes that must be made by weaving vehicles as they travel through the weaving section. A freeway auxiliary lane added with a one-lane entrance ramp and terminated with a one-lane exit ramp is defined as a Type A weave. A freeway auxiliary lane added with a one-lane entrance ramp and terminated with a two-lane exit ramp or tapered into the through roadway downstream of a one-lane exit ramp is defined as a Type B weave. A freeway auxiliary lane added with a one-lane entrance ramp and terminated with a two-lane exit ramp or tapered into the through roadway downstream of a one-lane exit ramp is defined as a Type C weave.

On-going research being conducted under the National Cooperative Highway Research Program (NCHRP) will result in improved methods for capacity and quality-of-service analyses of weaving areas (10). In addition to updating the freeway methodologies of the HCM, the findings of the research will address analysis of weaving areas on arterials, collector-distributor roads, and frontage roads. Another key element of the research is the assessment of the applicability and validity of traffic simulation models for analysis of weaving areas. Based on the scheduled completion of the research and, in part, on the controversial nature of the proposed methodologies, the
The findings of the research are not expected to be included in the 2000 Highway Capacity Manual, scheduled for completion in early 2000. As such, it is unknown when the new methodology will be available to practitioners.

AN OPERATIONAL ANALYSIS CASE STUDY

This paper summarizes the findings of a case study on the operational analysis of the weaving areas created by auxiliary lanes between two successive interchanges. The operational analyses of the case study were conducted as part of a Major Investment Study (MIS) in Dallas, Texas.

Study Background

The Northwest Corridor Major Investment Study was initiated by Dallas Area Rapid Transit (DART) in the Spring of 1998. The Northwest Corridor extends in a northwesterly direction from downtown Dallas and includes portions of the I-35E (15.7 miles) and SH 114 (8.9 miles) freeway corridors. Mobility elements evaluated as part of the MIS include rail transit, HOV lanes, general freeway improvements, bus service improvements, Transportation System Management (TSM), Travel Demand Management (TDM), and Advanced Transportation Management Systems (ATMS)/Intelligent Transportation Systems (ITS).

A key element of the study was the identification of freeway bottleneck improvements in the I-35E and SH 114 corridors to be included in the TSM/TDM alternative. Bottleneck locations were identified as those areas with poor level of service (LOS E or F) and/or poor accident ratings. In each of the bottleneck areas, improvement alternatives that were considered to be within the context of a TSM alternative were identified and assessed. Additional basic lanes or other major capacity-adding measures were not considered. Based on operational analyses (which are not addressed in this paper), auxiliary lanes were recommended at various locations within the study area to improve the level of service between two successive interchanges and to assist in accommodating high entering and/or exiting traffic volumes.

Auxiliary Lanes Alternatives

For auxiliary lanes added to existing freeways, tapering the auxiliary lane downstream of a one-lane exit ramp of a diamond interchange configuration can be cost-prohibitive. For elevated freeways (e.g., cross-street under the freeway), this taper would likely occur on structure. For depressed freeways (i.e., cross-street over the freeway), the taper is oftentimes constrained by bridge piers or abutments. Since the purpose of the bottleneck analysis was to identify low-cost improvements for inclusion in a TDM/TSM alternative, tapering the auxiliary lane downstream of a one-lane exit ramp was eliminated from further consideration.

On the surface, eliminating the option of tapering the auxiliary lane downstream of the exit ramp would appear to simplify the decision-making process to the following:
- For auxiliary lane length < 1,500 feet – Terminate auxiliary lane with one-lane exit ramp.
- For auxiliary lane length > 1,500 feet – Terminate auxiliary lane with two-lane exit ramp.

The AASHTO guidelines (1) imply that the provision of lane balance is necessary for efficient traffic operations. Are we sure? Is it possible that terminating auxiliary lanes of less than 1,500 feet with a two-lane exit ramp provides the best traffic operations? Similarly, is it possible that terminating auxiliary lanes of greater than 1,500 feet with a one-lane exit ramp provides the best traffic operations? To provide insight to this question, further operational analyses were conducted at 20 locations where auxiliary lanes were recommended. At each location, the analyses compared the quality-of-service provided by a one-lane exit ramp versus a two-lane exit ramp.

Site Characteristics

Ten sites from the I-35E corridor (five southbound sites and five northbound sites) and ten sites from the SH 114 corridor (five eastbound site and five westbound sites) were evaluated. The analyses were conducted using existing peak hour traffic volumes. The range of traffic and geometric conditions among the 20 sites varied as follows:
- Number of directional freeway lanes (upstream of the weaving section): 2-3
- Number of directional freeway lanes in the weaving section: 3-4
- Weaving section length (feet): 1,100-3,600
- Freeway volume (per lane) upstream of the weaving section (vph): 1,590-2,295
- Entrance ramp volume (vph): 330-1,420
- Exit ramp volume (vph): 150-1,200

Other key assumptions included:
- Mainline free flow speed: 60 mph on I-35E; 70 mph on SH 114
- Ramp speed: 45 mph
- No ramp-to-ramp traffic
- All other conditions ideal

Methodology

Three software packages were utilized to assess the quality-of-service provided in the weaving section for one-lane exit ramps versus two-lane exit ramps.

1. Highway Capacity Software (HCS) Version 3.1a - HCS is a macroscopic, deterministic model which replicates the procedures of Chapter 4 of the 1997 HCM. In these procedures, the quality-of-service within a weaving section is based on the average density of all vehicles in the section. With the exception of weaving section length, all of the analysis locations of the case study reflect geometric and operational conditions within the limitations of the HCM procedures. The weaving length limitations of the HCM represent the range of the data used in the calibration of the HCM methodology. However, input values beyond the limitations do not necessarily result in erroneous findings. Although the HCM recommends the application of Chapter 5 (Ramp Junction) procedures for these cases, the weaving procedures were applied for comparison purposes. For the purposes of the case study, the configuration providing the lowest average density in the weaving section was assumed to provide the best traffic operations.

2. CORSIM (TSIS Version 4.2) - Sponsored by the Federal Highway Administration, CORSIM is a microscopic, stochastic simulation model for analyzing urban networks. In microsimulation each vehicle is individually tracked through the model, and com-
Comprehensive operational measures of effectiveness are collected on every vehicle in the model for every second of model simulation. A wide variety of link and system-wide operational measurement statistics can be generated. For each analysis location, a simple model network representing the weaving section was created. In the comparison of one-lane exit ramps and two-lane exit ramps, all input parameters except the exit ramp configuration were held constant. For the purposes of the case study, the configuration providing the lowest overall delay for the entire system was assumed to provide the best traffic operations.

### 3. Synchro/Simtraffic Version 4.0

Synchro/Simtraffic is also a microsimulation package. Originally developed for intersection and arterial traffic flow, Synchro/Simtraffic has been updated to now model freeways including high speed merges and weaving sections. Similar to the CORSIM methodology, a simple model network representing the weaving section was created for each analysis location. In the comparison of one-lane exits and two-lane exits, all input parameters except the exit ramp configuration were held constant. For the purposes of the case study, the configuration providing the lowest overall delay for the entire system was assumed to provide the best traffic operations.

### Findings

The results of the assessment are summarized in Table 1. The key input data and the configuration providing the best traffic operations for the three methodologies are summarized in Table 1 for each methodology. Key observations from the findings include:

- Based on results of the HCS analyses, a one-lane exit ramp provided better traffic operations than a two-lane exit ramp at all twenty locations. In each case, the two-lane exit ramp (analyzed as a Type B weave configuration) resulted in higher average density within the weaving section than did a one-lane exit ramp (analyzed as a Type A weave configuration). The increase in density ranged from 12.8% to 29.5% and averaged 21.6%.

### Table 1: Operational Analysis Summary

<table>
<thead>
<tr>
<th>Facility</th>
<th>Direction</th>
<th>Location</th>
<th>Number of Upstream Basic Lanes</th>
<th>Number of Lanes in Weaving Section</th>
<th>Length of Upstream Weaving Section (ft)</th>
<th>Peak Hour Volume (vph)</th>
<th>Exit Ramp Configuration Providing Best Operations</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-35E</td>
<td>Northbound</td>
<td>Mockingbird to Empire Central Regal Row to Raceway Northside to Whitlock Hebron Pkwy to Corporate Drive Corporate Drive to SH 121</td>
<td>3</td>
<td>4</td>
<td>1,400</td>
<td>4,760</td>
<td>510</td>
</tr>
<tr>
<td>I-35E</td>
<td>Southbound</td>
<td>SH 121 to Corporate Drive Corporate Drive to Hebron Pkwy Vista Ridge to Frankford Regal Row to Empire Central Empire Central to Mockingbird</td>
<td>3</td>
<td>4</td>
<td>1,900</td>
<td>6,000</td>
<td>940</td>
</tr>
<tr>
<td>SH 114</td>
<td>Eastbound</td>
<td>Freeport to Esters Esters to Belt Line Belt Line to Valley View Valley View to Walnut Hill O’Connor to Rochelle</td>
<td>3</td>
<td>4</td>
<td>1,400</td>
<td>6,170</td>
<td>330</td>
</tr>
<tr>
<td>SH 114</td>
<td>Westbound</td>
<td>Rochelle to O’Connor O’Connor to Hidden Ridge Valley View to Beltline Belt Line to Esters Esters to Freeport</td>
<td>2</td>
<td>3</td>
<td>1,100</td>
<td>3,880</td>
<td>870</td>
</tr>
<tr>
<td>SH 114</td>
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<td>O’Connor to Hidden Ridge Valley View to Beltline Belt Line to Esters Esters to Freeport</td>
<td>3</td>
<td>4</td>
<td>2,200</td>
<td>6,850</td>
<td>900</td>
</tr>
<tr>
<td>SH 114</td>
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<td>4</td>
<td>2,600</td>
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<tr>
<td>SH 114</td>
<td>Westbound</td>
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<td>4</td>
<td>1,600</td>
<td>6,880</td>
<td>400</td>
</tr>
</tbody>
</table>
Based on the results of the CORSIM analyses, a one-lane exit ramp provided better traffic operations at thirteen of the twenty locations. However, at all twenty locations the change in total system delay between the one-lane exit ramp configuration and the two-lane exit ramp configuration was not substantial, ranging from –2.4% (where the two-lane exit ramp provided the best traffic operations) to 3.7% (where the one-lane exit ramp provided the best operations). Of the five locations with weaving length of 1,500 feet or less, three locations operated best with a one-lane exit ramp while two locations operated best with a two-lane exit ramp. Of the fifteen locations with weaving length greater than 1,500 feet, ten locations operated best with a one-lane exit ramp while five locations operated best with a two-lane exit ramp.

Based on results of the Synchro/Simtraffic analyses, a one-lane exit ramp provided better traffic operations than a two-lane exit ramp at all twenty locations. In each case, the two-lane exit ramp resulted in higher total system delay than did a one-lane exit ramp. The increase in total system delay ranged from 0.4% to 309.9% and averaged 33.7%.

CONCLUSIONS

For auxiliary lanes less than 1,500 feet in length, AASHTO lane balance principles permit the termination of the auxiliary lane with a one-lane exit ramp. For auxiliary lanes greater than 1,500 feet in length, the lane balance principles require that the auxiliary lane be dropped with a two-lane exit or tapered into the through roadway downstream of a one-lane exit. The findings of the case study suggest that a one-lane exit ramp may provide the best traffic operations, regardless of weave length. Observations using the animation feature of the simulation models provided a possible explanation for these findings. With a one-lane exit ramp, all of the exiting traffic must utilize the auxiliary lane. With a two-lane exit ramp, a portion of the exiting traffic remains in the basic lane to the left of the auxiliary lane. This was observed to result in a higher density for the basic lane and additional delay for through traffic.

This paper presents the findings of a case study. No field data to collaborate or refute the findings of the case study were collected. Although all of the analysis sites are located in the Dallas metropolitan area, it is important to note that the analyses were conducted based on assumed rather than measured driver characteristics. Thus, the relevance of the findings is not restricted to a specific geographical area.

The experience gained from the case study is presented to aid practitioners in the design of safe and efficient freeway facilities and to aid researchers in current and future efforts to define and understand the operational effects of geometric design. To the latter, the findings of the case study support the need for additional research on the operational effects of auxiliary lanes and lane balance.

ACKNOWLEDGEMENT

The Northwest Corridor Major Investment Study was conducted under contract to the Dallas Area Rapid Transit (DART). Special thanks to the staff of the Texas Department of Transportation, the Nebraska Department of Roads, and the Iowa Department of Transportation for their insights during the preparation of this paper.

REFERENCES

A Practical Approach to Managing Intersection Traffic Data for Large Scale Studies

JEFF GERKEN AND DAN MEYERS

As an increasing number of metropolitan areas study the possibility of enhancing the modal options of their transportation systems, the number of Environmental Impact Statements (EIS) that need to be prepared has increased significantly. A recent EIS was conducted that required peak hour intersection Level of Service (LOS) calculations for over 60 intersections for two base-year and seven future-year scenarios (nearly 1,100 intersection data records). Tight time constraints and the need for efficient stewardship of this large data set lent itself to employing a data management tool. The proper tool would encompass electronic data management as well as provide signal optimization and LOS calculations. The traffic engineering software package Synchro was chosen for this task. Existing turning movement counts (TMC), geometric conditions, and signal timing were entered into peak period Synchro files. The Synchro base year TMC were exported in comma delimited (CSV) file format and converted to approach turn percentages using a spreadsheet program. The regional transportation planning model output provided daily link volumes for each scenario. Intersection approach volumes were then determined using historical K and D factors. Incorporating the approach volumes into the TMC spreadsheet provided horizon year TMC. The TMC were then imported back into the Synchro file and optimized to provide future year intersection LOS. This innovative procedure saved considerable time in both data error checking and traffic analysis. Once the data set was entered into Synchro, all further data management and analysis was electronically handled, therefore reducing data entry time and the potential for data handling errors. Key words: turn movement counts, level of service, universal traffic data format (UTDF), Synchro.

BACKGROUND

Current Practice

A widely accepted standard for signalized intersection LOS analysis is the Highway Capacity Software (HCS). The most recent version of HCS is 3.1, which was recently converted to a windows-based software program. HCS 3.1 is a direct application of the 1997 Highway Capacity Manual. Signalized intersection analysis requires data inputs such as lane geometry, peak-hour turning volumes, and signal phasing and timing as a minimum. Additional inputs are numerous and can range from ideal saturation flow rate to right turn treatment. Figure 1 illustrates a typical methodology for HCS analysis.

INTRODUCTION

As an increasing number of metropolitan areas study the possibility of enhancing the modal options of their transportation systems, the number of Environmental Impact Statements (EIS) that need to be prepared has increased significantly. A typical EIS contains numerous sections addressing impacts such as noise, air quality, and traffic. This paper summarizes an innovative methodology for conducting traffic analysis as required by an EIS or any other large-scale traffic analysis effort.

A recent EIS for Tampa, Florida was conducted that required AM and PM peak-hour intersection Level of Service (LOS) calculations for over 60 intersections for two base year and seven future year scenarios (nearly 1,100 intersection data records). Tight time constraints and the need for efficient stewardship of this large data set lent itself to employing a data management tool. Innovations in standardized traffic data formatting have made electronic data management a practical tool for all practitioners. The Tampa EIS serves as the case study for this paper.

It is not the intention of this paper to endorse or discredit any traffic engineering software package. This paper simply identifies a case study and the methodology the authors used to conduct the traffic analysis portion of an EIS. It is hoped that practitioners will benefit from the methodology and other information presented in this paper.

FIGURE 1 Typical HCS analysis process
Tampa Case Study

The Tampa case study required nearly 1,100 signalized intersection LOS analyses (60 intersections, 2 peak periods, and 9 scenarios). The traditional approach would require one file for each intersection for each scenario. It is easy to see using the standard HCS methodology would require significant resources, both manpower and time, to complete the analysis task. The tight time constraint of the project deadline required the traffic analysis team to seek out a LOS analysis package that incorporated electronic data management. The appropriate package had to encompass electronic data management as well as provide signal optimization and LOS calculations. The ideal analysis tool required entering intersection characteristics (geometry, volume, and signal data) only once and then allowed manipulation of the electronic data by scenarios to complete the required analysis. Additional project needs included a turning movement count (TMC) review mechanism for local experts as well as a project quality review mechanism for the traffic analysis team.

LOS Analysis Package

The traffic engineering software package Synchro (version 3.2) was chosen as the analysis tool. Synchro is a macroscopic, deterministic, signalized intersection LOS analysis software program utilizing the 1994 HCM methodologies. Synchro has a companion model called SimTraffic, which is a microscopic, stochastic simulation package. Additionally, Synchro has the ability to interface with database files using a universal traffic data format (UTDF). UTDF is explained in greater detail later in this section.

To begin the process, the traffic analysis team developed a link-node system corresponding directly to the desired traffic network under study. The network development was streamlined by using a “dxf” formatted graphics file as a background base map. The ability to develop a network of signalized intersections allowed for system analysis—i.e. use of offsets for progression, queueing and blocking problem identification, etc.—rather than analyzing each individual intersection independently. A more subtle advantage to a system analysis—i.e. PM peak hour with existing geometric conditions—compared to individual intersection analyses—i.e. Elm and Main for PM peak with existing geometric conditions—would simply be minimization of errors due to confusion. Additionally, individual files would not be required for each intersection greatly reducing the number of files to manage. For example, in this case study the 1,080 files that would have been required if individual intersection analysis were used (60 intersections; 2 peak periods; 9 scenarios) was reduced to 18 files (2 peak periods; 9 scenarios) with the system approach. Practitioners dealing with stacks of signal timing sheets, peak hour volume reports, and geometric drawings would most likely agree that handling all intersections in a system is less confusing than the alternative and more importantly results in fewer errors.

Initial intersection data input was accomplished through successive input screens for traffic volume, geometry, and signal timing. Once each intersection data was input, a calibration effort was initiated. Calibration of the macroscopic analysis model entailed adjustment of the saturation flow rates, startup loss times, and right turn on red treatment to name a few. Further, the SimTraffic option of Synchro was used for visual quality review. No additional programming is required to use the features of SimTraffic since Synchro serves as the input editor for SimTraffic.

Traffic analysis of this scale has a high potential for input or analysis errors due in most part to the size of the project and the enormous amount of data involved. Common errors that were observed using SimTraffic for quality review were mainly geometric problems such as turn lane assignments and turn bay lengths. The traffic analysis team, being knowledgeable on the study corridor, easily reviewed the input data for errors within a short time using the SimTraffic program. Figure 2 illustrates the typical methodology for using Synchro.

FIGURE 2 Typical Synchro analysis process

UTDF

UTDF was developed by Trafficware—the developer of Synchro—in an effort to spur a standard format for traffic engineering variables. UTDF enables data access through open-ended or nonproprietary software programs such as spreadsheets, text editors, or database programs. UTDF simply provides a means to electronically manipulate standard traffic data, in this case traffic volumes. For instance, imagine reentering the turning movement volumes for an intersection 17 times, then moving onto the next intersection as in this case study. Wouldn’t you rather spend your project budget analyzing the traffic conditions, testing alternatives, or developing mitigation concepts rather than entering data?

UTDF uses text files to store and share data. Both comma delimited (CSV) and column aligned text files are supported. The column aligned files can easily be manipulated with a text editor. For this study, the CSV format was used since the data can be modified with a spreadsheet program. Figure 3 illustrates traffic volume data for several intersections in the case study. The first two rows are header information with specific details to the UTDF file. In Figure 3, the header indicates that these are turning movement counts, and the counts are in vehicles per hour (60 minute counts). Each possible turn movement (NBL, NBT, etc.) is identified in the third row, with

Data Collection
volume/geometry/timing/phasing/etc.

Synchro Software
manual entry into baseline network scenario file

SimTraffic Software
for visual quality check or microsimulation analysis

LOS Output

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er
the intersection identification (INTID) number in the third column. The intersection identification number corresponds to the node number assigned in Synchro when the network was initially built. For example, node #1 is a “T” intersection with 275 vehicles approaching the intersection from the south and turning left. Other data are provided (date and time) which allows multiple counts by date and time to be stored in a single database file. For this case study, the turning movement counts for all 60 intersections for each peak period were stored in a single UTDF file.

FIGURE 3 Base year multiple intersection volumes, UTDF format

![Image of the UTDF format for base year intersection volumes](image.png)

The intersection identification number in the third column. The intersection identification number corresponds to the node number assigned in Synchro when the network was initially built. For example, node #1 is a “T” intersection with 275 vehicles approaching the intersection from the south and turning left. Other data are provided (date and time) which allows multiple counts by date and time to be stored in a single database file. For this case study, the turning movement counts for all 60 intersections for each peak period were stored in a single UTDF file.

FIGURE 5 Graphical representation of turning movement data

![Image of the graphical representation of turning movement data](image.png)

The intersection identification number in the third column.

**METHODOLOGY/APPLICATION**

**Case Study**

A Light-Rail Transit (LRT) EIS was conducted in Tampa, Florida that required nearly 1,100 intersection LOS analyses. The large number of intersection analyses was desired to document the impacts of LRT right-of-way takes and traffic volume changes on the study corridor intersections. The use of UTDF incorporated with a standard spreadsheet program provided a very powerful methodology to accomplish the required task. Figure 4 illustrates the methodology developed for this project.

The first step was to enter the traffic data into base year, or existing conditions, Synchro files for both peak periods. This process is discussed in the LOS Analysis Package section of this paper. These files served as the platform for all subsequent analyses. Existing peak period LOS was then determined as a direct output from the existing conditions files.

The next step was to develop future year traffic conditions based on planning model growth and existing traffic turning movement percentages. The Synchro base year TMC were exported in CSV file format (Figure 3) and converted to approach turn percentages using a spreadsheet program (Figure 5). This established the basic traffic flow patterns for the study corridor. The intersection identification number, originally assigned as the node number in Synchro, serves as the key to connecting intersection data. The regional transportation planning model output provided daily link volumes for each of the nine scenarios. These data were provided in plot format and were manually entered into a separate spreadsheet corresponding to the assigned intersection identification number from the Synchro file. Intersection approach volumes were then determined using historical K and D factors. Linking the approach volumes to the TMC spreadsheet provided horizon year TMC (future year turning volumes). Figure 5 illustrates this process. The project further required providing base year TMC as a review mechanism for local applications.
See Figure 3

"Write" volumes 'csv' file from Synchro

See Figure 5

"Read" Future turn movement volumes

QC Review

See Figure 6

(Optional) Alter geometric, signal, volume conditions

(Optional) SimTraffic visual check

Optimize signal timing for Future Conditions

Intersection LOS Future Conditions

Manual Entry

Synchro Existing Conditions File

Intersection LOS Existing Conditions

Forecasted Segment ADT

Intersection approach volume scenarios

Existing Segment ADT

Spreadsheet (Calculates TM %, Future TM volumes)

TMC Graphic Base Year and Future Volumes as well as turn %

Data Collection

Formatted Data

Spreadsheet Process

Synchro Process

Analysis Output/ Deliverable

Legend

Notes: Processes electronic unless otherwise noted

FIGURE 4 Case study methodology application
Gerken and Meyers

Turning Movement Count

60 Minute Counts

<table>
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<tr>
<th>DATE</th>
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<td>249</td>
<td>2996</td>
<td>95</td>
<td>75</td>
<td>85</td>
<td>50</td>
<td>27</td>
<td>15</td>
<td>25</td>
</tr>
<tr>
<td>8/19/20</td>
<td>1700</td>
<td>6</td>
<td>187</td>
<td>844</td>
<td>381</td>
<td>1020</td>
<td>1614</td>
<td>705</td>
<td>925</td>
<td>2946</td>
<td>347</td>
<td>432</td>
<td>2034</td>
<td>927</td>
</tr>
<tr>
<td>8/19/20</td>
<td>1700</td>
<td>27</td>
<td>157</td>
<td>917</td>
<td>710</td>
<td>105</td>
<td>963</td>
<td>460</td>
<td>189</td>
<td>1748</td>
<td>121</td>
<td>346</td>
<td>2491</td>
<td>75</td>
</tr>
<tr>
<td>8/19/20</td>
<td>1700</td>
<td>32</td>
<td>168</td>
<td>46</td>
<td>233</td>
<td>4</td>
<td>12</td>
<td>3</td>
<td>22</td>
<td>4287</td>
<td>193</td>
<td>78</td>
<td>3408</td>
<td>53</td>
</tr>
<tr>
<td>8/19/20</td>
<td>1700</td>
<td>34</td>
<td>166</td>
<td>439</td>
<td>75</td>
<td>132</td>
<td>1378</td>
<td>132</td>
<td>247</td>
<td>121</td>
<td>346</td>
<td>2491</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td>8/19/20</td>
<td>1700</td>
<td>37</td>
<td>232</td>
<td>361</td>
<td>200</td>
<td>174</td>
<td>1313</td>
<td>70</td>
<td>229</td>
<td>1912</td>
<td>107</td>
<td>159</td>
<td>2353</td>
<td>95</td>
</tr>
</tbody>
</table>

FIGURE 6 Future year volumes, UTDF format

CONCLUDING REMARKS

A methodology was presented for a practical approach to managing intersection traffic data for large-scale traffic analysis efforts. An EIS in Tampa, Florida served as the case study in which nearly 1,100 intersection LOS analyses were required. The methodology presented allowed for manual intersection data entry to occur once, reducing time and data handling errors. The catalyst for the electronic data handling was the use of a universal traffic data format.

ACKNOWLEDGEMENTS

The Environmental Impact Statement used as the case study in this paper was prepared for Hillsborough Area Rapid Transit (HART) in Tampa, Florida. Special thanks to the technical review committee consisting of representatives from the City of Tampa, Florida Department of Transportation and HART for their input.
Pavement maintenance is traditionally the most expensive function of a highway-operating agency. However, the long-term benefits of maintenance strategies are typically not quantified. More is known about the initial, relative improvements associated with a given maintenance strategy than is known about its impact on a pavement’s performance and life. This is one of the main reasons given for not integrating maintenance activities into pavement management systems. This paper discusses procedures used to quantify the improvements and costs associated with typical maintenance strategies and activities and demonstrates the benefit of utilizing these relationships in pavement management, specifically the tradeoffs between pavement rehabilitation and maintenance. Iowa Department of Transportation contract maintenance and in-house maintenance records are used to quantify improvements and costs associated with individual maintenance strategies. The authors intend to use the results of this study in a multi-year prioritization program used by the Iowa Department of Transportation as its pavement management system software. The paper demonstrates the benefits of using maintenance data in the pavement management process. Key words: pavement, maintenance, and management systems.

INTRODUCTION

Pavement maintenance is a very crucial component of any highway agency’s operation. Highway agencies try to maintain their existing highway network in a state that provides a safe and smooth ride to the travelling public through the application of maintenance strategies. Research has been conducted at the national level to determine the value of maintenance to encourage highway agencies to adopt sound maintenance practices (preventive maintenance) and understand the benefits of maintaining the highway network. Individual highway agencies need to carry this research further to determine the impact of their own maintenance practices on the condition of their highway network. The results can be incorporated into a pavement management system to conduct resource allocation and project selection.

This paper discusses procedures used to determine the impact of several maintenance strategies on pavement condition for the Iowa Department of Transportation (Iowa DOT). The research described goes through the process of obtaining the data, assessing the impact, and finally determining the benefits of each individual treatment strategy or a combination of strategies. The paper is divided into three major sections. The first section describes the methodology used to acquire the necessary data, maintenance activity identification, data review and validation, and then data integration using a dynamically segmented GIS database. The second section describes the impact assessment of specific maintenance strategies as they were used by the Iowa DOT. This process is completed by determining the change in pavement condition (considering the before and after condition data) as a result of the application of a specific maintenance strategy. Finally, conclusions are made to the value and benefit of maintenance treatments and how that information is incorporated into the Iowa DOT pavement management system.

Another component of the research was to look at the cost of maintenance treatments and determine the feasibility of individual maintenance alternatives. At this point, not enough data were available to allow the researchers to conduct a benefit cost analysis. Instead, cost estimates will be used in the Iowa DOT pavement management system to integrate both pavement management and pavement maintenance in one system.

METHODOLOGY

As noted previously, a primary objective of this research is to quantify the relative improvements to pavement condition resulting from different maintenance activities and the cost of these improvements. The objective of this portion of the research is to perform data integration necessary to accomplish this objective, specifically integrate historic programmed and routine maintenance activities and pavement condition data. This is accomplished through four tasks: 1) data acquisition, 2) maintenance activity identification, 3) data review, and 4) data integration and summary. The following section documents each of these tasks as well as discusses issues encountered during the integration process.

Pavement Condition Data

Pavement condition data were obtained from the Iowa DOT pavement management information system (PMIS). Condition data, which are typically collected biennially, were provided by pavement management section for the years 1992 through 1998. Condition measures include a composite index, ride, friction, cracking, patching, and faulting.

Programmed Maintenance Data

The Iowa DOT Maintenance Division provided records of programmed (contracted) maintenance activities (MP Projects) for the years 1991 through 1998. Of particular interest to this study were the dates during which work was performed, the type(s) and cost(s) of
work performed, and project location. In many cases, several types of work, designated by work codes, were performed on the same project.

**Routine Maintenance Data**

Information about routine maintenance activities (in-house maintenance work) was obtained from the Iowa DOT Maintenance Management System (MMS) (Office of Maintenance) for the years 1994 through 1997. Once again, key elements of interest were the dates during which work was performed, the type(s) and cost(s) of work performed, and work location. For the purposes of this research, maintenance activities of interest were those that directly improved the roadway surface. Furthermore, activities that performed or served the same basic function were grouped together and considered a single activity. All other maintenance activities, although present in the programmed and routine maintenance records, were not considered. Tables 1 and 2 present the programmed and routine maintenance activities of interest.

**TABLE 1 Programmed Maintenance Activities**

<table>
<thead>
<tr>
<th>Programmed Maintenance Activities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint and crack filling with emulsion (ACC pavement 2-lane)</td>
</tr>
<tr>
<td>Joint and crack sealing (ACC pavement 2-lane)</td>
</tr>
<tr>
<td>Joint and crack filling with emulsion (ACC pavement 4-lane)</td>
</tr>
<tr>
<td>Joint and crack sealing (ACC pavement 4-lane)</td>
</tr>
<tr>
<td>Joint and crack sealing (PCC pavement 2-lane)</td>
</tr>
<tr>
<td>Joint and crack sealing (PCC pavement 4-lane)</td>
</tr>
<tr>
<td>Full-depth patching ACC/PCC ACC partial-depth patching</td>
</tr>
<tr>
<td>PCC partial-depth patching</td>
</tr>
<tr>
<td>Longitudinal joint repair</td>
</tr>
<tr>
<td>Transverse joint repair (per 2-lane width)</td>
</tr>
<tr>
<td>Microsurfacing (2-lane)</td>
</tr>
<tr>
<td>Microsurfacing (4-lane)</td>
</tr>
<tr>
<td>Pavement seal coat (CRS-2P 2-lane)</td>
</tr>
<tr>
<td>Pavement slurry seal (ACC pavement 2-lane)</td>
</tr>
<tr>
<td>Pavement seal coat (CRS-2P 4-lane)</td>
</tr>
<tr>
<td>Pavement slurry seal (ACC pavement 4-lane)</td>
</tr>
<tr>
<td>Pavement double slurry seal (ACC pavement 2-lane)</td>
</tr>
<tr>
<td>Pavement double slurry seal (ACC pavement 4-lane)</td>
</tr>
<tr>
<td>ACC resurfacing (2-lane) 1 inch deep</td>
</tr>
<tr>
<td>ACC resurfacing (2-lane) 2 inch deep</td>
</tr>
<tr>
<td>ACC resurfacing (2-lane) 3 inch deep</td>
</tr>
<tr>
<td>ACC resurfacing (2-lane) 4-5.5 inch deep</td>
</tr>
<tr>
<td>ACC resurfacing (2-lane divided) 4-5.5 inch deep</td>
</tr>
<tr>
<td>ACC resurfacing (2-lane) 6 inch deep</td>
</tr>
<tr>
<td>Adds milling to ACC resurfacing projects (1.5 inch depth)</td>
</tr>
<tr>
<td>Intermittent AC resurfacing (spot leveling)</td>
</tr>
</tbody>
</table>

**TABLE 2 Routine Maintenance Activities**

<table>
<thead>
<tr>
<th>Routine Maintenance Activities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spall Patching and Hand Leveling</td>
</tr>
<tr>
<td>Machine Surface Restoration and Leveling</td>
</tr>
<tr>
<td>Permanent Surface Repair</td>
</tr>
<tr>
<td>Joint and Crack Filling</td>
</tr>
<tr>
<td>Joint and Crack Routing and Sealing</td>
</tr>
<tr>
<td>Strip Sealing – Edge Sealing</td>
</tr>
<tr>
<td>Pavement Replacement</td>
</tr>
<tr>
<td>Seal Coating – Slurry Sealing – Fog Sealing</td>
</tr>
<tr>
<td>Pavement Expansion Relief Joints</td>
</tr>
<tr>
<td>Burn/Plane or Mill Surface</td>
</tr>
<tr>
<td>Underseal/Raise Pavement</td>
</tr>
</tbody>
</table>

The data review process entailed a more detailed analysis of the component data sets. Data sets were analyzed for consistency, completeness, and potential data integration issues.

**Pavement Condition**

The pavement condition data are both consistent and complete. Locations of pavement management sections are clearly defined by county, route, milepost, and direction. Modifications to section definitions over time have also been appropriately addressed within the data set. Field collected condition data are provided (for most sections) on a biennial basis. A composite index is often estimated for years in which field data are not collected.

The complete data set, with the exception of a few urban sections, is appropriate for further analysis. Effective use of the aforementioned urban sections is prohibited because of the complex relationship among existing and historic section definitions.

**Programmed Maintenance**

The majority of the programmed maintenance data is also consistent and complete. Locations of pavement management sections are clearly defined by route and milepost. However, in some instances the milepost information is missing, incomplete, or inaccurate. As a result, these records (projects) can not be used in analysis.

To accurately attribute condition improvements to the appropriate maintenance activity, the data set was initially limited to projects with a single surface improvement work code (i.e., a single maintenance activity was undertaken on a particular section in a given year). This significantly decreased the number of observations for the maintenance activities. Therefore, logical work code (activity) pairs were identified, based on the data provided, for inclusion in analysis. Activity pairs are those surface improvement activities regularly performed in conjunction with each other or one preceding the other, such as patching and resurfacing. The activity pairs are presented in Table 3. Projects are not considered if more than two activities occur on a section.
Routine Maintenance

Several aspects of the routine maintenance data from the Maintenance Management System prohibit its use in the analysis. Milepost information is missing, incomplete, or inaccurate (to a much greater extent than programmed maintenance activities.) In addition, the milepost location description is not specific enough to accurately assign activities to the appropriate pavement management section(s). Furthermore, cost information is missing or incomplete in many records, and “miscellaneous” is often assigned to surface maintenance activities.

As noted in the previous section, only the historic programmed maintenance activities and pavement condition data are used to assess the pavement condition improvement associated with specific maintenance activities. Several different tools are utilized to integrate and summarize these data. Primary tools include a geographic information system (GIS) with dynamic segmentation capabilities, Intergraph Corp. MGE, and a relational database management system, Oracle. This section briefly outlines the manner in which the maintenance activity and condition data are integrated as well as how the aforementioned tools are utilized. The procedures necessary to create the GIS environment for dynamic segmentation are not discussed within this context.

Data Validation

The first step of data integration is data validation through use of GIS dynamic segmentation capabilities. Dynamic segmentation provides the ability to create representations of roadway features or events; in this case pavement management sections and maintenance projects, along a GIS representation of the roadway network. Database attributes (county, route, begin and end milepost) describe the location and extent of the pavement management sections and maintenance projects along the network. The objective of the validation is to ensure that the graphic representation of these features accurately represents the feature, specifically the section or project extent, in the field. The length of the graphic representation of the maintenance projects and pavement management sections is calculated using dynamic segmentation and compared to the their field lengths, calculated using other database attributes. If the difference between the graphic distance and field distance is not within a specified tolerance, the project or section is eliminated from further analysis. This difference may be defined as an absolute difference, such as one-kilometer, or a proportional difference, e.g., the graphic distance is 90 percent of the field distance. An absolute difference of one-kilometer was used for all sections in this research; however, use of a proportional difference may be more appropriate for short sections.

Maintenance Activity Identification

The next step of data integration is identifying how many maintenance projects (activities) occurred along a pavement management section during each year of analysis. To accurately attribute condition improvements to the appropriate maintenance activity, the only sections considered are those along which a single project (maintenance activity code) occurred during a given year. If a
single maintenance project occurred on the section, the type of activity, or activity pair, is assigned to the section. Dynamic segmentation is utilized to overlay pavement condition data and condition data, on a year by year basis, and update the condition data accordingly. Since condition data are not available prior to 1992 or after 1998, this process is repeated for the years of 1993 through 1997 only.

**New Record Creation**

The previous process yields a year by year account of the number, and type, of programmed maintenance activities occurring on each pavement management section. Type is provided only if a single activity occurred during a given year. The pavement management sections along which this occurs are of primary interest. The third step of the data integration process uses these sections and the maintenance data to generate a new subset of data. Dynamic segmentation is utilized to overlay the aforementioned data sets and create the new record set. This record set represents a union of the pavement management sections and maintenance activities for each year between 1993 and 1997. Data associated with the new record set are its graphic length, the total graphic length and project cost of the maintenance project from which it originated, and the section identifier of the pavement management section from which it originated. The data are used in the subsequent steps to assign a proportion of maintenance activity and cost to each pavement management section.

**Maintenance Activity Summary**

The next steps in the data integration process involve modifying and aggregating the records contained in the data set created in the previous step. First, the proportional, or weighted, cost of maintenance for each record in the new data set is calculated. This is accomplished by multiplying the total project cost by the graphic length of the new record and then dividing by the graphic length of the entire maintenance project. The new cost represents the cost of the maintenance activity along that portion of the pavement management section. Upon calculating the maintenance activity cost of each new record, all records are summarized by pavement management section identifier and year of maintenance activity. The graphic length and cost (weighted) of all records are summed for each management section and year. This yields the total lane miles and cost of maintenance performed on a pavement management section in a given year.

**Combined Maintenance-Condition Summary**

The last step in the data integration process is associating the summarized maintenance data with the appropriate pavement management section. The result is a database table containing pavement condition and programmed maintenance activities, by type, cost, and lane mile, on a year by year basis. Another important attribute, derived from this data set, is the proportion of the pavement management section along which the maintenance activity occurred. The proportion is calculated by dividing the total lane miles of maintenance by the length of the pavement management section. This proportion is a primary factor in the analysis that follows, as is the timing of the condition data with respect to a maintenance activity. For example, if a maintenance activity occurred along only 20 percent of a pavement management section, the activity will likely have little impact on the condition of the section as a whole. This will be discussed in more detail in the next section.

**IMPACT ASSESSMENT**

To assess the impacts of different maintenance activities on pavement condition, two indices were taken into consideration. These were the Pavement Condition Index (PCI) and the International Roughness Index (IRI). PCI is a composite index with a range of 0–100 (zero representing the worst possible condition and 100 representing the best possible condition). The IRI, the International Roughness Index, is a profile index describing road profile roughness that causes vehicle vibrations. Expressed in units of slope (inches per mile and/or meters per kilometer), it can take values of zero or greater (zero representing an absolutely flat profile).

The combined maintenance-condition summary file was reviewed and, following a step-wise procedure, an assessment of the impact of maintenance activities on PCI and IRI values was conducted. The authors identified highway sections in the combined maintenance-condition summary file where:

1. Only one type of maintenance activity was carried out in a year.
2. The activity covered more than 50 percent of the section length.
3. Field measured distress data (PCI and/or IRI) were available before and after the maintenance activity.
4. The PCI and IRI values indicated some improvement in pavement condition due to the maintenance activity.

For example, if a section of highway was resurfaced (ACC resurfacing 2-lane 2-inch deep–WC 42) in 1995 and distress data were measured in 1994 and 1996, then the difference in the before and after distress data indicate the improvement in the section condition due to the application of that maintenance strategy. The process was completed manually by checking each pavement section over the entire analysis period.

Observed improvements to PCI and IRI were noted. However, those improvements did not reflect the actual improvements because they did not take into account the deterioration of pavement condition between the treatment time and the time that it was re-observed. A correction was applied to the observed improvements in PCI and IRI values by taking into consideration the deterioration of the pavement between the treatment time and the after observation period. Age-based PCI & IRI equations developed for the Iowa DOT pavement management system were used for this purpose.

Tables 4 and 5 present a summary of the results for improvements in PCI and IRI due to different types of maintenance activities. A brief description of the two tables follows.

- The first two columns in both tables list the work code and its description.
- The third column lists the observed mean difference in PCI and IRI.
- The fourth column lists the mean change in PCI and IRI when deterioration effects between the before and after time period were taken into consideration.
- The last column provides the number of observations.

Some of the results are based on relatively few observations and thus the confidence in the PCI and/or IRI improvements is low. However, in the absence of other information on the improvements.
TABLE 4  Impact of Maintenance Activities on PCI Values

<table>
<thead>
<tr>
<th>Work Code</th>
<th>Description</th>
<th>Mean Observed Change in PCI</th>
<th>Mean Change in PCI After Correction</th>
<th>No. of Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>WC1</td>
<td>Joint and crack filling with emulsion ACC 2-lane</td>
<td>1.63</td>
<td>6.33</td>
<td>11</td>
</tr>
<tr>
<td>WC2</td>
<td>Joint and crack sealing ACC 2-Lane private</td>
<td>2.14</td>
<td>6.31</td>
<td>21</td>
</tr>
<tr>
<td>WC3</td>
<td>Joint and crack sealing PCC 2-lane private</td>
<td>0.50</td>
<td>1.66</td>
<td>2</td>
</tr>
<tr>
<td>WC6</td>
<td>Joint and crack sealing ACC 4-Lane private</td>
<td>-1.25</td>
<td>1.48</td>
<td>4</td>
</tr>
<tr>
<td>WC10</td>
<td>Full depth patching ACC/PCC</td>
<td>1.08</td>
<td>3.50*</td>
<td>23</td>
</tr>
<tr>
<td>WC11</td>
<td>ACC Partial depth patching</td>
<td>1.00</td>
<td>5.72</td>
<td>1</td>
</tr>
<tr>
<td>WC20</td>
<td>Microsurfacing 2-Lanes</td>
<td>2.10</td>
<td>4.76</td>
<td>10</td>
</tr>
<tr>
<td>WC21</td>
<td>Pvt. Fog seal ACC 2-Lane</td>
<td>1.00</td>
<td>6.47</td>
<td>1</td>
</tr>
<tr>
<td>WC22</td>
<td>Pvt. Seal coat CRS-2P 2-Lane</td>
<td>3.33</td>
<td>5.08</td>
<td>3</td>
</tr>
<tr>
<td>WC40</td>
<td>Intermittent AC resurfacing (spot leveling)</td>
<td>5.28</td>
<td>8.60</td>
<td>7</td>
</tr>
<tr>
<td>WC42</td>
<td>ACC resurfacing 2-Lane 2” deep</td>
<td>5.67</td>
<td>11.41</td>
<td>3</td>
</tr>
<tr>
<td>WC43</td>
<td>ACC resurfacing 2-Lane 3” deep</td>
<td>8.16</td>
<td>11.03</td>
<td>6</td>
</tr>
<tr>
<td>WC44</td>
<td>ACC resurfacing 2-Lane 4'-4.5’ deep</td>
<td>-2.260</td>
<td>-2.320</td>
<td>1</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>92</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* This conservative result is based on PCC PCI equation

TABLE 5  Impact of Maintenance Activities on IRI Values

<table>
<thead>
<tr>
<th>Work Code</th>
<th>Description</th>
<th>Mean Observed Change in IRI</th>
<th>Mean Change in IRI After Correction</th>
<th>No. of Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>WC1</td>
<td>Joint and crack filling with emulsion ACC 2-lane</td>
<td>-0.294</td>
<td>-0.432</td>
<td>14</td>
</tr>
<tr>
<td>WC2</td>
<td>Joint and crack sealing ACC 2-Lane private</td>
<td>-0.225</td>
<td>-0.340</td>
<td>16</td>
</tr>
<tr>
<td>WC10</td>
<td>Full depth patching ACC/PCC</td>
<td>-0.515</td>
<td>-0.570*</td>
<td>8</td>
</tr>
<tr>
<td>WC20</td>
<td>Microsurfacing 2-Lane</td>
<td>-0.292</td>
<td>-0.324</td>
<td>15</td>
</tr>
<tr>
<td>WC40</td>
<td>Intermittent AC resurfacing (spot leveling)</td>
<td>-0.450</td>
<td>-0.600</td>
<td>3</td>
</tr>
<tr>
<td>WC42</td>
<td>ACC resurfacing 2-Lane 2” deep</td>
<td>-1.210</td>
<td>-1.323</td>
<td>4</td>
</tr>
<tr>
<td>WC43</td>
<td>ACC resurfacing 2-Lane 3” deep</td>
<td>-0.580</td>
<td>-0.725</td>
<td>4</td>
</tr>
<tr>
<td>WC44</td>
<td>ACC resurfacing 2-Lane 4'-4.5’ deep</td>
<td>-2.260</td>
<td>-2.320</td>
<td>1</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>65</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* This conservative result is based on PCC IRI equation

due to those maintenance activities on distress indices, these are the best estimates. More data are needed to refine the estimates based on relatively few observations.

CONCLUSIONS

The results from the impact assessment section clearly show the benefits of specific maintenance strategies using either the PCI or the IRI condition indicators. As discussed earlier, however, more data are needed so that a more refined benefit analysis can be conducted. As more data are collected, more pavement sections and maintenance strategies can be considered. This will allow the researchers to refine the improvements in PCI and IRI values as a result of applying a specific treatment strategy.

The next step in this process is to integrate the data from this research into the Iowa DOT pavement management system. The Iowa DOT uses a multi-year prioritization program for resource allocation and project selection. So far, the pavement management system only considers rehabilitation and reconstruction strategies. During the current phase of the project (2000), the maintenance data will be integrated into the pavement management system and the impact of doing maintenance will be assessed. This will allow the Iowa DOT staff to better represent their maintenance program and provide with the ability to justify additional budget request for preventive maintenance work.

ACKNOWLEDGMENTS

The research presented in this paper was sponsored by the Iowa Department of Transportation. The opinions, findings, and conclusions expressed are those of the authors and not necessarily those of the Iowa Department of Transportation.
Interim Guidelines for Thin Maintenance Surfaces in Iowa

CHARLES T. JAHREN, KENNETH L. BERGESON, AHMED AL-HAMMADI, SERHAN CELIK, GABRIEL LAU, AND HERNANDO QUINTERO

The first phase of a two-phase research project was conducted to develop guidelines for Iowa transportation officials on the use of thin maintenance surfaces (TMS) for asphaltic concrete and bituminous roads. Thin maintenance surfaces are seal coats (chip seals), slurry seals, and micro-surfacing. Interim guidelines were developed to provide guidance on which roads are good candidates for TMS, when TMS should be placed, and what type of thin maintenance surface should be selected. The guidelines were developed specifically for Iowa weather, traffic conditions, road-user expectations, and transportation official expectations.

INTRODUCTION

The use of thin maintenance surfaces (TMS) can be a cost-effective approach for maintaining the quality of pavement. These surfaces are usually applied to flexible pavements. They include seal coats, slurry seals, and micro-surfacing.

Seal Coats are constructed by spraying binder on the road (usually emulsified asphalt) and then spreading aggregate before the binder sets. The aggregate is rolled into the binder to ensure that it remains in place. The aggregate may be rock chips, pea gravel, or sand. A seal coat made with rock chip aggregate is sometimes called a chip seal, while a seal coat made with sand is sometimes called a sand seal. Double seal coats are constructed by successively placing two layers of aggregate.

Slurry Seal is produced by mixing the aggregate and binder in a mobile mixing machine. The binder is usually asphalt emulsion, and the aggregate varies in size and type, depending on the application. Portland cement, hydrated lime, or aluminum sulfate is often added to aid in setting the slurry. The slurry is applied with a spreader box that is pulled behind the mixing truck and distributes and finishes the slurry.

Micro-surfacing is similar to slurry seal technology. Polymer-modified binder and one-hundred percent crushed aggregate is used. Micro-surfacing cures faster and may be applied in a thicker layer than slurry seal.

Studies have shown that transportation agencies can maintain a road network with better pavement condition at a lower cost by properly using TMS. In planning TMS programs, project selection, treatment selection, and timing are extremely important.

Projects must be selected for maintenance when TMS are still effective. In most cases, the proper time to apply the TMS is before the need is apparent to the casual observer. This is because once pavements start to deteriorate, they deteriorate rapidly beyond the point where TMS is effective. Maintenance planners may be reluctant to order treatments for roads that appear to be structurally sound, when nearby roads appear to be in greater need of repair. However, when TMS applications are properly timed, road networks will show improvements in service life over the long term (1).

TMS do not increase the structural rating of the road and will fail quickly if applied to a road that is experiencing a structural failure. Cracks reflect quickly through slurry seals and micro-surfacing. Therefore, these treatments should be applied before cracks form or in conjunction with crack maintenance programs.

It is important to select the right maintenance treatment for each situation. Pavement condition, traffic volumes, road type (urban, rural, interstate, primary, secondary), materials availability, and local preference must be considered in making this decision. For maximum benefit, TMS must be applied before pavement distress is apparent. Research has shown that pavement deteriorates slowly when it is new and then deteriorates more rapidly after it reaches a certain age (Figure 1). If the pavement is allowed to deteriorate rapidly, it will soon be necessary to rebuild the pavement, an expensive proposition. Alternatively, a thin maintenance surface may be applied that will improve the pavement condition to the point where pavement performance deteriorates slowly. Several maintenance treatments may be applied periodically to maintain the pavement above the point of quick deterioration. For each dollar spent on maintenance before the age of rapid deterioration, four dollars can be saved in rebuilding costs (1). Preventive maintenance provides benefits in addition to cost savings: on the average, road users enjoy better pavement conditions when compared to a strategy of allowing the pavement to deteriorate to the point that rebuilding is necessary.

In the past, many applications of TMS have been unsuccessful because they have been applied too late and failed rapidly. After such experiences, transportation personnel tend to hesitate to use TMS. Also, it is difficult to institute a program of preventive maintenance with properly timed treatments because the public often perceives that money is being wasted on good roads while other projects are being neglected. It would be desirable to develop an assessment procedure that would allow planners to accurately determine the optimum timing of TMS and to include this assessment procedure in an overall pavement management system. Also, it would be desirable to clearly explain the need for prompt treatment of pavements before distress is apparent.
The Iowa DOT is considering the development of preventive maintenance programs. First, however, it would be desirable to develop a system for planning TMS maintenance programs that are tailored to Iowa’s climate, materials, and contracting practices.

**PROBLEM STATEMENT**

By properly using TMS as a preventive maintenance technique, agencies can cost-efficiently maintain the surface condition of Iowa highways and streets at a high level. For this strategy to be successful, proper selection, timing, and application are critical. Previous international, national, and state research has provided a basic framework for implementing such maintenance programs in Iowa. However, it is necessary to customize this framework to address Iowa’s specific needs with regard to aggregates, climate, construction practices, traffic, and fund management.

**METHODOLOGY**

Several steps were required to execute the study to this point. These steps are described in detail elsewhere; they are summarized below (2, 3, 4). The project commenced in May 1997 with a literature review to find previously developed guidelines for TMS. Researchers also found information on materials and mix designs for TMS, as well as assessment techniques for roads that are candidates for TMS treatments. The results of the literature review are discussed in connection with specific topics throughout this report.

A survey was also conducted to determine current uses of TMS by Iowa counties and municipalities. Questions were also asked regarding contracting strategy, future plans, and needs for information.

Plans were made for observing construction and assessing the performance of TMS research test sections. Before the start of this research project, the Iowa DOT had contracted to construct a set of test sections in Linn County on US 151 northeast of Cedar Rapids and Benton County on US 30 just west of the intersection of US 218. The test sections included several types of seal coats using local aggregates, micro-surfacing, slurry seal, cape seal (seal coat with slurry top), and a thin lift hot mix overlay.

Researchers conducted a pre-construction condition survey, observed construction, and conducted three post-construction condition surveys. Due to the contractor’s schedule, construction of these test sections occurred late in the construction season (September and October 1997) when cold temperatures did not allow the emulsion to cure properly. In addition, many of the application rates varied considerably from the target rates. This compromised the research value of these test sections.

Plans were then made for constructing a set of test sections during the 1998 construction season between Huxley and Alleman on US 69. These test sections were designed by the researchers in cooperation with the Iowa DOT and with Koch Materials Inc., which supplied emulsion and asphalt cement for the project. The sections included several types of seal coat using local and imported aggregate, micro-surfacing, micro-surfacing with a seal coat interlayer, a hot sand mix overlay, and a Nova Chip ultra thin hot mix seal. Minnesota DOT provided considerable assistance in designing the chip seal application rates. Researchers conducted a pre-construction condition survey, observed construction, and conducted a post-construction condition survey.

**INTERIM GUIDELINES FOR TMS**

After reviewing the literature and the results of the 1997 test sections, a set of preliminary guidelines was developed to assist transportation officials with TMS timing and selection. It is expected that these guidelines will be refined as additional performance information is obtained from the test sections.

**Three-Step Decision Procedure**

A three-step decision procedure is recommended (see flowchart, Figure 2). A detailed explanation of the tables is provided elsewhere (4).

**Step 1 - Collect information on candidate roads for thin maintenance surfaces.** A performance survey should be conducted to assess the amount and type of distress that the road is suffering. The survey could be a detailed distress survey to provide input for PCI calculations. If a pavement management system is in place, the PCI has been calculated and tracked for a number of years. Thus additional helpful information regarding the rate of deterioration is available. At least a visual assessment should be made and rut depths should be noted. The traffic count should also be obtained and areas that must withstand many turning and stopping movements should be noted.

**Step 2 - Identify feasible treatments.** Using Table 1, identify feasible treatments. Table 2 provides additional guidance for selecting treatments for roads where rutting is the primary distress.
Step 3 - Consider other factors before making a final selection.
Table 3 provides a list of other factors that should be considered before making a final selection.

Timing

Properly timing the construction of TMS is extremely important. If the treatment is applied too soon, funds are being expended on roads that do not require treatment. If the treatment is applied too late, the road may have deteriorated to the point that TMS are ineffective. Most experts suggest that TMS be first applied to a road seven to 10 years after it is first constructed.

---

**TABLE 1** Recommended Maintenance Strategies for Various Distress Types and Usage (2, 4)

<table>
<thead>
<tr>
<th>Distress Survey</th>
<th>Traffic</th>
<th>Seal Coat</th>
<th>Slurry Seal</th>
<th>Micro-Surfacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>ADT &lt; 2000</td>
<td>R</td>
<td>R</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>2000 &gt; ADT &lt; 5000</td>
<td>M&lt;sup&gt;1&lt;/sup&gt;</td>
<td>M&lt;sup&gt;1&lt;/sup&gt;</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>ADT &gt; 5000</td>
<td>NR</td>
<td>NR</td>
<td>R</td>
<td></td>
</tr>
</tbody>
</table>

1. Traffic
   - ADT < 2000
   - 2000 > ADT < 5000
   - ADT > 5000

2. Bleeding
   - R
   - NR
   - R

3. Rutting
   - NR
   - R
   - R

4. Raveling
   - R
   - R
   - R

5. Cracking
   - Few tight cracks
   - Extensive cracking
   - R
   - NR
   - NR

6. Improving friction
   - Yes
   - Yes
   - Yes<sup>2</sup>

7. Snow plow damage
   - Most susceptible
   - Moderately susceptible
   - Least susceptible

R = Recommended
NR = Not Recommended
M = Marginal
<sup>1</sup>There is a greater likelihood of success when used in lower speed traffic
<sup>2</sup>Micro-surfacing reportedly retains high friction for a longer period of time

---

**TABLE 2** Selection Process for Medium and High Traffic Based on Rutting and Cracking (3, 4)

<table>
<thead>
<tr>
<th>Rut Depth</th>
<th>Treatment</th>
<th>Less than</th>
<th>¼ to ½ in.</th>
<th>½ to 1 in.</th>
<th>Greater than</th>
<th>1 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Micro-Surfacing&lt;sup&gt;1&lt;/sup&gt;</td>
<td>One course</td>
<td>Scratch course and final surface</td>
<td>Rut box and final surface</td>
<td>Multiple placement with rut box</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slurry Seal&lt;sup&gt;2&lt;/sup&gt;</td>
<td>One course</td>
<td>One course</td>
<td>Micro-surfacing Scratch course and final surface</td>
<td>See note 3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<sup>1</sup>As recommended by International Slurry Seal Association
<sup>2</sup>Current practice in Iowa
<sup>3</sup>Sometimes successful (anecdotal evidence)

---

**CONCLUSIONS**

The following conclusions were made:
- When properly selected, timed, and constructed, TMS can economically maintain the condition and extend the life of pavement surfaces.
- Good construction techniques and attention to detail are critical to the success of TMS.
- Warm weather is required for several days after application to properly cure emulsion products.
- If TMS are applied to roads that are in poor condition, they are likely to have a limited life.
- Roads should be first considered for TMS seven to 12 years.
after construction (when fine aggregate first starts to ravel).

• According to the literature, the expected life of a thin maintenance surface is between five and 10 years.

**Table 3: Factors That Affect Maintenance Planners' Decisions**

<table>
<thead>
<tr>
<th></th>
<th>Seal Coat (SC)</th>
<th>Slurry Seal (SS)</th>
<th>Micro-Surfacing (MS)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Past Practices</strong></td>
<td>Most officials prefer not to change successful past practice unless there is a definite reason for a change. These reasons could be positive or negative changes in funding, neighbor complaints, user complaints, or an opportunity to use better product.</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Funding and Cost</strong></td>
<td>Least expensive option—less funding is required.</td>
<td>More expensive than SC and less expensive than MS.</td>
<td>Most expensive option—more funding is required.</td>
</tr>
<tr>
<td><strong>Durability</strong></td>
<td>Dependent on aggregate type, binder type, and application technique.</td>
<td>Less durable than micro-surfacing.</td>
<td>More durable than slurry seal.</td>
</tr>
<tr>
<td><strong>Turning and Stopping Traffic</strong></td>
<td>Can be flushed by turning and stopping traffic.</td>
<td>Can hold turning and stopping traffic.</td>
<td>Best wear in turning and stopping traffic.</td>
</tr>
<tr>
<td><strong>Dust During Construction</strong></td>
<td>Considerable dust possible.</td>
<td>Little dust possible.</td>
<td>Little dust possible.</td>
</tr>
<tr>
<td><strong>Curing Time</strong></td>
<td>Road can be opened after rolling is completed and speed should be limited to about 20 mph for 2 hours.</td>
<td>Road can be opened after 2 hours in warm weather, and 6-12 hours in cold weather.</td>
<td>Road can be opened after 1 hour.</td>
</tr>
<tr>
<td><strong>Noise and Surface Texture</strong></td>
<td>Fairly noisy surface, open surface texture, and many loose rocks immediately after construction.</td>
<td>Less noise, dense surface texture (close to hot mix surface).</td>
<td>Less noise, dense surface texture (close to hot mix surface).</td>
</tr>
<tr>
<td><strong>Availability of Contractors</strong></td>
<td>13 contractors in Iowa.</td>
<td>3 contractors in Iowa.</td>
<td>2 contractors in Iowa.</td>
</tr>
<tr>
<td><strong>Use of local Aggregates</strong></td>
<td>Maximum flexibility - Can use somewhat dusty aggregates with cutback binder. - Can use emulsion or cutbacks. - Rock chips, pea gravel, and sand may be used.</td>
<td>Less flexibility.</td>
<td>Least flexible. The binder is highly reactive (break time is affected by clay content).</td>
</tr>
</tbody>
</table>

1 Dust is mitigated by using washed, hard, or precoated aggregate.
2 U.S. Department of Transportation, Federal Highway Administration.

**Acknowledgements**

This research project was sponsored by Iowa DOT and CTRE (TR 435). A steering committee was appointed to assist the researchers. Members included John Selmer, P.E., Iowa DOT Maintenance Operations; Francis Todey, P.E., Iowa DOT Maintenance Operations; John Hagen, P.E., formerly Iowa DOT Bituminous Engineer, currently Ames Resident Construction Engineer; Neal Guess, P.E., Newton City Engineer; Randy Kraul, P.E., Carroll City Engineer; Richard Scheik, P.E. and L.S., Kossuth County Engineer; Dave Paulson, P.E., Carroll County Engineer; William Dunshee, Fort Dodge Asphalt, Fort Dodge, Iowa; Richard Burchett, Stabilt Construction Co., Harlan, Iowa; Bill Ballou, Koch Materials, Inc., Salina, Kansas. This support is gratefully acknowledged.

**References**

Selection of Milling Depth for Asphalt Pavement Rehabilitation

ZHONG WU, MUSTAQUE HOSSAIN, AND ANDREW J. GISI

Milling of an asphalt concrete (AC) pavement surface refers to the mechanical removal of a part of the pavement surface. The Kansas Department of Transportation (KDOT) and the Kansas Turnpike Authority (KTA) routinely mill some AC pavements before inlaying. KTA selects the milling depth based on engineering judgement. On some major projects where deep milling is involved, KDOT currently selects a mill-and-inlay depth so that the ratio of this depth to the remaining milled pavement thickness would be higher than 1.0. This somewhat insures that much of the full pavement thickness would consist of newer materials. These practices of KDOT and KTA were analyzed in a recent study and the findings are reported in this paper. Nine mill-and-inlaid pavement sections, selected on six different routes in Kansas, were studied. Falling Weight Deflectometer (FWD) tests were done on those sections at 15 or 30-m intervals before milling, after milling, and after inlaying. Laboratory fatigue tests were also done on the AC beams sawn from four test sections. The results show that in order to achieve higher fatigue life, the mill-and-inlay thickness should be at least 1.25 times of the thickness of the remaining milled pavement layer. Based on the mechanistic-empirical analysis procedure, it was found that the mill-and-inlaid pavement fatigue life also does not linearly increase with the inlay thickness. Key words: asphalt pavement, milling-and-inlay, fatigue life, rutting, serviceability, FWD.

INTRODUCTION

Milling of an asphalt concrete (AC) pavement surface refers to the mechanical removal of a part of the pavement surface. The Kansas Department of Transportation (KDOT) and the Kansas Turnpike Authority (KTA) routinely mill some AC pavements before inlaying. KTA selects the milling depth based on engineering judgement. On some major projects where deep milling is involved, KDOT currently selects a mill-and-inlay depth so that the ratio of this depth to the remaining milled pavement thickness would be higher than 1.0. This somewhat insures that much of the full pavement thickness would consist of newer materials. The assumption is that the existing AC and base layers are in good structural condition to provide extended service lives. However, little is known about the damage of these materials due to this rehabilitation action (mill and inlay) as these layers age and undergo repetitive loading while the top of the surface is being replaced. In most cases, the milling depth is selected based on the rule-of-thumb or experience of the agency for a specific surface distress, such as rutting, rather than on any engineering analysis. In fact, no design procedure is available for this rehabilitation since the mill and inlay pavements are not exclusively covered in the 1993 AASHTO Pavement Design Guide (1). Current practices of KDOT and KTA for mill-and-inlay strategy were analyzed in a recent study by Kansas State University and will be reported in this paper.

TEST SECTIONS AND DATA COLLECTION

Test Section

Nine 300-m long mill-and-inlaid pavement sections, on six different routes in Kansas, were studied in this project. Table 1 shows the location and general features of these projects. The projects on the I-35 and I-335 are under the jurisdiction of the Kansas Turnpike Authority (KTA), and the rest belong to the KDOT network. Falling Weight Deflectometer (FWD) tests were done on those sections at 50 or 100-feet intervals before milling, after milling, and after inlaying.

Field Sampling and Fatigue Testing

Laboratory fatigue tests were done on the AC beams sawn from four test sections on I-35, US-59, and K-16. Tests were done in a three-point flexural loading fashion using a sinusoidal load of 10 Hz frequency. Constant stress mode was chosen in the flexural tests, and test temperature was controlled at 20°C. The initial tensile strain was measured at the bottom fiber of the beams at the 200th repetition of the applied load, and the beams were then loaded to failure. The applied load was varied so that a range of tensile strains and corresponding varying cycles to failure would be obtained. Detailed discussions on the fatigue tests may be found elsewhere (2).

Existing Pavement Condition

Most of the sites were visited before rehabilitation and quantitative distress data were obtained from the KDOT’s pavement management system database. Common distresses were longitudinal or fatigue cracking, transverse cracking, and rutting. In general, rutting and transverse cracking were found on all sites. On average, each site had about 6.25 to 12.5 mm rutting. Fatigue cracking was observed on the sites on I-35, I-335, US-59, and K-92. Severe fatigue cracking was found on the US-59 and K-92 sites. K-16 and K-177

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had a few longitudinal cracks. The former also had severe transverse cracking. These distresses contributed to the roughness shown in the last column of Table 1. Only K-177 had lower severity of all distresses mentioned, and that is well reflected in its lower International Roughness Index (IRI) value when compared with other sites.

**FWD Data Analysis and Existing Pavement Damage**

As mentioned earlier, three different sets of FWD data were collected on each test section—before milling, after milling, and after inlaying. A linear elastic backcalculation program EVERCALC, developed by the Washington Department of Transportation, was used to analyze the deflection data. The pavements were modeled as a four-layer structure (before milling and after milling) and a five-layer structure (after inlaying). A “stiff” layer was used whenever necessary for convergence. The backcalculated AC layer moduli were adjusted to a pavement temperature of 20°C. In general, the backcalculation results were judged to be good (root-mean-square error less than 2%). Details can be found elsewhere (2).

The existing pavement damage was assessed by comparing two critical pavement responses corresponding to an 80 kN single-axle, dual tire load with a tire pressure of 690 kPa: 1) the horizontal tensile strain at the bottom of the AC layer, and 2) the vertical compressive strain on the top of the subgrade before milling, after milling, and after inlaying. The layered elastic pavement analysis program, ELSYM5, was used in this analysis. It was found that both tensile strains and vertical strains increased significantly after milling. However, after inlaying, both strain values decreased at least to the pre-milling level, thereby indicating that the critical pavement responses remain unaffected due to the mill-and-inlay action (2).

### Table 1 Test Section Features

<table>
<thead>
<tr>
<th>Route</th>
<th>County</th>
<th>Last Rehab. Year</th>
<th>Present Mill and Inlay Thickness (mm)</th>
<th>Existing AC Base Thick. (mm)</th>
<th>Current Annual ESAL's (on design lane)</th>
<th>IRI (before milling)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-35 S</td>
<td>Sumner</td>
<td>1991</td>
<td>64</td>
<td>190</td>
<td>388,816</td>
<td>1.72</td>
</tr>
<tr>
<td>I-35 N</td>
<td>Chase</td>
<td>1988</td>
<td>50</td>
<td>178</td>
<td>268,841</td>
<td>1.51</td>
</tr>
<tr>
<td>I-335 N</td>
<td>Osage</td>
<td>1992</td>
<td>50</td>
<td>178</td>
<td>146,000</td>
<td>1.67</td>
</tr>
<tr>
<td>US-59-1</td>
<td>Anderson</td>
<td>1988</td>
<td>25</td>
<td>127</td>
<td>12,775</td>
<td>1.67</td>
</tr>
<tr>
<td>US-59-2</td>
<td>Anderson</td>
<td>1988</td>
<td>50</td>
<td>100</td>
<td>22,630</td>
<td>1.81</td>
</tr>
<tr>
<td>K-16</td>
<td>Pottawatomie</td>
<td>1985</td>
<td>25</td>
<td>152</td>
<td>4,380</td>
<td>1.85</td>
</tr>
<tr>
<td>K-92-1</td>
<td>Jefferson</td>
<td>1990</td>
<td>50</td>
<td>89</td>
<td>6,205</td>
<td>1.17</td>
</tr>
<tr>
<td>K-92-2</td>
<td>Jefferson</td>
<td>1990</td>
<td>25</td>
<td>114</td>
<td>6,205</td>
<td>1.17</td>
</tr>
<tr>
<td>K-177</td>
<td>Riley</td>
<td>1992</td>
<td>150</td>
<td>114</td>
<td>39,055</td>
<td>1.15</td>
</tr>
</tbody>
</table>

**DETERMINATION OF THE MILL-AND-INLAY THICKNESS**

### Equivalent Thickness Method (ETM)

The first methodology for determining mill-and-inlay thickness used in this study is ETM, which follows the “AASHTO Overlay Method Using NDT” as described in the Volume II, Appendix NN of the 1986 AASHTO Guide for Design of Pavement Structures (3). This overlay design methodology uses nondestructive testing to evaluate the in-situ deflection basin characteristics of a pavement. A backcalculation technique is used to estimate the elastic moduli ($E_i$) of the pavement layers. The effective structural number ($S_{neff}$) of the existing pavement from the following equation:

$$S_{neff} = 0.0045 \sum h_i \left( E_i \right)^{1/3}$$

Where:

- $h_i$ is the thickness of the $i$-th layer above the subgrade; and
- $E_i$ is the elastic modulus of the $i$-th layer above the subgrade.

The overlay/inlay requirements can then be estimated by using the effective thickness concept. In this study, the structural numbers (SNs) of the pavements since last rehabilitation, available in KDOT and KTA data bases, have been taken as design SNs. The backcalculated moduli of the milled pavement and other layers (if any) were used to calculate the $S_{neff}$. The deficiency in structural number because of milling was calculated by subtracting $S_{neff}$ from the design/historical SN. Finally, the inlay thickness was calculated by dividing this difference with 0.42, which is assumed to be the structural layer coefficient of the new inlay AC material.
Table 2 tabulates the required inlay thicknesses for all pavement test sections in this study. The results obtained were mixed. Apparently, there were no thickness requirements for I-35 South and I-355 for the given traffic whereas I-35 North sections required about 18 mm overlay. The results on US-59, K-16, and K-92-1 were very misleading presumably due to the very high existing AC moduli obtained in the backcalculation process. On K-177, 100 mm of inlay was required. Thus, this procedure does not appear to be applicable to the mill-and-inlaid pavements.

Regression Equation Method

In order to study the effect of the mill-and-inlay thickness on pavement structural life, multiple regression analysis was done where the tensile strain at the bottom of the AC layer (before milling or after inlaying) was taken as a dependent variable and several independent variables were investigated. The independent variables were ratio of the inlay thickness to the remaining pavement thickness, total pavement thickness above subgrade, ratio of the inlay modulus to the existing AC layer modulus, subgrade modulus, and effective pavement modulus above subgrade. Data from all sections except K-177 was not included in the analysis since it had a much higher milling depth compared to other sections.

Excellent $R^2$ value was obtained by including all five variables. However, the students' t-values of the moduli ratio and total thickness above subgrade were too low, implying that those two variables were not highly correlated with the independent variable. After dropping those two variables, the following equation was obtained:

$$
\varepsilon_r = 21.55 \times r_0 - 0.03813 E_p + 4.203 M_r \quad (R^2 = 0.91)
$$

where

- $\varepsilon_r$ = horizontal tensile strain at bottom of the AC layer after inlaying/before milling (microstrains);
- $r_0$ = thickness ratio of the inlay layer to the remaining milled AC layer;
- $E_p$ = effective pavement modulus above subgrade, ksi (backcalculated using the AASHTO algorithm);
- $M_r$ = subgrade modulus, ksi (backcalculated using the AASHTO algorithm).

The limitations of Equation (2) include a smaller database and possible hidden auto-correlation of $r_0$, $E_p$, and $M_r$ with the horizontal tensile strain taken as a dependent variable. Equation (2) was used to compute the horizontal tensile strains at the bottom of the AC layer for the I-35 South and US-59-1 sections for different mill-and-inlay thickness ratios. Figure 1 shows the relationships obtained. It appears that the fatigue lives of these pavements could be doubled by choosing thickness ratios of 1.25 for US-59 and 1.50 for I-35, respectively. Thus, it is apparent that in order to achieve a very high fatigue life, the mill-and-inlay thickness should be at least 1.25 times the thickness of the remaining AC layer after milling. However, in some cases, the increase in thickness ratio may not significantly decrease the tensile strain at the bottom of the AC layer (or in other words, may not increase the fatigue life significantly) since it also depends upon the existing pavement layer moduli.

Mechanistic-Empirical (M-E) Method

The M-E design method for asphalt pavements commonly include estimation of pavement damages due to traffic and environmental loading by using distress prediction models or transfer functions that relate a critical structural response to specific distress damage (e.g., fatigue cracking or rutting). In this study, both fatigue lives and lives corresponding to limiting rutting of the test sections were estimated.

The functional lives were predicted from the AASHTO serviceability equations were compared to the fatigue lives predicted from the laboratory fatigue equations for the sites on I-35, US-59, and K-16. The results show that the fatigue lives on most sections are usually equal to or more than the functional lives. This indicates that the pavement sections in this study will most likely be rehabilitated due to excessive roughness. However, in some cases, the fatigue lives will be exhausted before the functional lives. Thus, it is important to safeguard against premature fatigue cracking, the mill-and-inlay strategy selection must also consider the fatigue life.

The rutting potential of the mill-and-inlaid pavements was studied. Based on the results from the rut depth equation developed by LTPP (5), no significant change of rutting potential of the mill- and-inlaid AC pavements was evident no matter what mill-and-inlay thicknesses. The rutting potential of the mill-and-inlaid pavements was evident no matter what mill-and-inlay thicknesses.

TABLE 2 Computed Inlay Thicknesses by the Equivalent Thickness Method

<table>
<thead>
<tr>
<th>No.</th>
<th>Section</th>
<th>Design SN</th>
<th>Existing AC $E_{ac}$ (MPa)</th>
<th>Thickness (mm)</th>
<th>BASE $E_{ac}$ (MPa)</th>
<th>Thickness (mm)</th>
<th>$SN_{eff}$</th>
<th>Inlay Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>I-35 S</td>
<td>4.2</td>
<td>1,090</td>
<td>191</td>
<td>200</td>
<td>457</td>
<td>4.31</td>
<td>-8</td>
</tr>
<tr>
<td>2</td>
<td>I-35 N</td>
<td>5.1</td>
<td>2,310</td>
<td>178</td>
<td>234</td>
<td>457</td>
<td>4.81</td>
<td>18</td>
</tr>
<tr>
<td>3</td>
<td>I-355</td>
<td>5.0</td>
<td>1,545</td>
<td>178</td>
<td>138</td>
<td>457</td>
<td>5.20</td>
<td>-13</td>
</tr>
<tr>
<td>4</td>
<td>US-59-1</td>
<td>3.3</td>
<td>2,351</td>
<td>127</td>
<td>483</td>
<td>152</td>
<td>4.11</td>
<td>-46</td>
</tr>
<tr>
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ness is selected. Since this thickness is usually smaller compared to the total AC thickness, and the overall pavement structure is not being changed, the rutting potential of this type of pavements can be considered insignificant. However, this analysis assumes that the mixture and construction process would not contribute to the rutting on the inlays (2).

Figure 2 shows the relationships between the mill-and-inlay thickness and the allowable ESALs for I-35 South and US-59-1 using the laboratory fatigue models developed in this study. The figure shows that on I-35 South, with an increase in mill-and-inlay thickness, the allowable ESALs first increased to a maximum value, then decreased. The allowable ESALs to fatigue failure did not vary much for 25 to 75 mm of milling thickness. This means that the milling thickness has a critical range of values on I-35 South, outside which the fatigue life will decrease. The above analysis implies that since rutting potential does change on mill-and-inlaid pavements before and after inlaying, the pavement fatigue life should be selected in such a way that it may safeguard the pavement functional life. Thus the critical mill depth range obtained from the fatigue life analysis should be used to estimate the optimum mill-and-inlay depth.

ACKNOWLEDGMENTS

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REFERENCES

The objective of the Dulles Corridor Rapid Transit Project was to identify and define transit Intelligent Transportation System (ITS) technologies, or concepts, recommended for implementation in the Dulles Corridor. The Corridor is located in the Washington, D.C. region in northern Virginia. In the project, a comprehensive set of ITS concepts was prioritized and grouped into three categories for deployment — coordinate, implement, and monitor. The “coordinate” concepts are currently proposed or deployed in the Corridor or are primarily functions of a traffic agency or information service provider. The “implement” concepts are recommended for implementation as a part of the project and provide the greatest potential for payoff. The “monitor” concepts are immature technologies that can be implemented beyond the period of the project or may be implemented sooner if the technology matures rapidly. A technology implementation plan was then developed for the Corridor. The plan provides implementation phasing, and capital, operations, and maintenance cost estimates for each of the implementation concepts. The plan also provides qualitative benefits of the ITS concepts. The paper discusses the project background and process used in prioritizing and grouping the ITS concepts. It presents the ITS concepts with respect to the three categories (groupings), and discusses those recommended for implementation as a part of the project (the implement concepts). Key words: transit ITS, bus rapid transit (BRT), market packages, corridor, northern Virginia.

PURPOSE

The purpose of this paper is to present the process and results of the Dulles Corridor Rapid Transit Project. The project identified and prioritized a comprehensive set of Intelligent Transportation System (ITS) technologies, or concepts, applicable to the Dulles Corridor. The prioritized ITS concepts were reviewed further and grouped into three categories for implementation — coordinate, implement, and monitor. A discussion of the project background, process, and recommended ITS concepts is provided below. The implement concepts are discussed in greater detail because they are recommended for deployment specifically as a part of the Dulles Corridor Rapid Transit Project.

BACKGROUND

The Dulles Corridor is located in the Washington, D.C. region in northern Virginia. The corridor has become one of the most recognized places in the country for technology and internet-based companies to locate. The corridor runs approximately 30 miles from West Falls Church to Leesburg, with major activity centers along the way including Tysons Corner, Reston/Herdon, Dulles International Airport, and eastern Loudoun County.

Growth along the Dulles Corridor has increased beyond the pace of transportation improvements, and in all likelihood, it will continue. With this in mind, transportation agencies in this region have embarked on a multimodal transit program. Providing oversight and direction to this effort is the Dulles Corridor Task Force. The Task Force is made up of executives from stakeholder transportation and planning agencies in the Dulles Corridor. The Task Force has subcommittees to recommend funding, service delivery, technology, and management of the project. The Technology Task Group is responsible for developing recommendations for the application of ITS technologies in the Dulles Corridor project. Representation in the group includes the following: Northern Virginia Transportation Commission, Virginia Department of Rail and Public Transportation, Virginia Department of Transportation, Washington Metropolitan Area Transit Authority, Washington Airports Task Force, Fairfax County, Loudoun County, Dulles Area Transportation Association, and Metropolitan Washington Airports Authority.

To provide rapid transit service in the Corridor, the Dulles Corridor Task Force developed an implementation program, which consists of the following four phases:

- Phase I: Express Bus – Starting in 1999, this phase provides express bus service and new bus routes within Fairfax County serving Herndon/Monrovia and Wiehle Avenue to Tysons Corner and the West Falls Church Metro station.
- Phase II: Enhanced Express Bus – Starting in 2001, this phase provides additional bus routes and buses serving eastern Loudoun County and Fairfax County to Tysons Corner and the West Falls Church Metro station.
- Phase III: Bus Rapid Transit (BRT) – Starting in 2003, this phase provides new BRT routes and buses serving eastern Loudoun County, Dulles Airport, Reston/Herdon, Tysons Corner, and the West Falls Church Metro station.
- Phase IV: Rail – Starting in 2006, this phase provides rail from Metrorail’s orange line East Falls Church station through Tysons Corner. Starting in 2010, it extends rail from Tysons Corner to Reston/Herdon, Dulles Airport, and Routes 606 and 772 in Loudoun County.

PROCESS

A straightforward process was used to derive viable, comprehensive, and integrated ITS concepts for the Corridor. The process used a step-by-step approach to pare down concepts. Approved evalu-
tion criteria, costs, and other coordinated efforts were used to select concepts for implementation in the Corridor. The process was very effective in keeping the Technology Task Group informed and involved. The process was comprehensive and based upon the U.S. DOT National ITS Architecture, Version 2.0.

The U.S. DOT National ITS Architecture is a framework for the integration of ITS into the transportation system. A comprehensive list of ITS concepts, applicable to the Dulles Corridor, was developed from the National ITS Architecture market packages. Market packages provide an accessible, deployment oriented perspective defining specific technology application concepts. The market package approach helps to map ITS projects, developed from the recommended ITS concepts, to the various subsystems and components of the National ITS Architecture. The use of market packages also helps to identify recommended information flows between subsystems.

To gain a greater level of specificity for this project, some market packages were broken out in further detail or were tailored to this project. For example, the concepts of wayside/in-station traveler information (e.g., dynamic message signs at transit stops) and in-vehicle traveler information (automated next-stop annunciation) were broken out in greater detail from the market package, Transit Traveler Information. Other concepts, such as transit vehicle tracking and broadcast traveler information, remained at the level of detail as defined in the National ITS Architecture market packages. Some of the concepts, such as platform screen doors and precision docking, are unique and are not associated with a particular, existing market package.

Once the list of applicable ITS concepts was developed, each was evaluated against weighted criteria and ranked. The Technology Task Group developed the criteria, assigned weights to each, and performed the evaluation and ranking. The following criteria were used to evaluate and rank the ITS concepts: consistency with the Technology Task Group’s application criteria and policy, technical feasibility, customer benefits, operator benefits, integration and compatibility with existing and planned systems, cost effectiveness (qualitative), and community and agency opportunity.

Benefits were assessed qualitatively because of the lack of available quantitative benefits information. The ITS concepts were then screened further using a logic and expert panel check. As a result of the prioritization and screening process, the list of concepts was broken into the following three categories:

- **Coordinate**: Those already proposed or implemented by candidate transit operators in the Corridor, or those that are primarily functions of a traffic agency or other agency.
- **Implement**: Those recommended for implementation as a part of the Dulles Corridor Rapid Transit Project with the greatest potential for payoff.
- **Monitor**: Those that could be implemented beyond the time period of the project or may be implemented sooner if the technology moves rapidly from an immature technology to a mature one.
After the concepts were grouped into the above three deployment categories, the implement concepts were organized into the following four functional areas: 1) electronic payment, 2) safety and security, 3) traveler information, and 4) operations.

Figure 1 presents the ITS concepts recommended for deployment in the Dulles Corridor. The concepts are grouped into the three deployment categories (coordinate, implement, monitor). The bold concepts in the coordinate ring are currently employed in the region.

A technology implementation plan was then developed for the Corridor and presented to the Dulles Corridor Task Force for adoption. The plan provides implementation phasing and capital, operations, and maintenance cost estimates for each of the implementation concepts.

The above process gave the study team a picture of the Dulles Corridor implementation recommendations in the context of regional efforts to implement ITS. A discussion of the three deployment categories is provided below. Implementation concepts are highlighted.

**COORDINATION CONCEPTS**

Some of the ITS concepts should be deployed in the Dulles Corridor by an agency or organization other than the Corridor’s designated transit operator. These ITS concepts are often intermodal in nature and require coordination with the designated transit operator. In general, the coordination ITS concepts did not rate as high a priority as those recommended for implementation by this project. However, the concepts are applicable to the Corridor and provide many benefits.

The importance of the coordination concepts is that many of them are currently deployed in the region. Thus, an ITS infrastructure is already available to build upon and expand ITS in the Dulles Corridor. This provides a cost-efficient approach for ITS deployment in the Corridor. The coordination concepts are represented in the middle ring of Figure 1. Concepts in bold are currently employed in the region.

**IMPLEMENTATION CONCEPTS**

The ITS concepts recommended for implementation in the Dulles Corridor provide the greatest benefit to the Corridor’s transit passengers and designated transit operator. These concepts ranked highest in the prioritization and screening process discussed earlier in the “Process” section. The implementation concepts are grouped into four packages: 1) traveler information, 2) electronic payment, 3) safety and security, and 4) operations. The concepts are represented in the center circle of Figure 1.

**Traveler Information Concepts**

The traveler information concepts provide real-time transit vehicle schedule information at transit stops and real-time occupancy information at parking facilities. It also provides next-stop location information to passengers onboard transit vehicles. Transit vehicle tracking provides real-time location input to the transit information technologies. The traveler information concepts increase customer convenience, save passengers time, relieve uncertainty and anxiety, help travelers make smart decisions, and build customer loyalty and confidence. Each of the traveler information concepts is discussed below.

**Transit Vehicle Tracking**

Transit vehicle tracking, which includes automatic vehicle location (AVL) and computer-aided dispatching (CAD) functions, provides real-time location information for schedule adherence, dispatch, and traveler information. Often, other ITS applications interface or are integrated with the transit vehicle tracking system. These applications include a silent alarm for alerting dispatchers of emergencies, vehicle engine probes to alert drivers and dispatchers of potential engine problems, automatic passenger counters (APC), and in-vehicle traveler information systems (automated next-stop annunciation). For this project, however, the system provides vehicle location, schedule adherence, and dispatching.

**Parking Facility Information**

This concept provides real-time parking availability information and navigational guidance for parking lots and garages. Information is typically provided via dynamic message signs (DMS) in the vicinity of the parking facility and at the parking facility. Signage may specify the number of parking spaces or whether or not the facility is full. Signage may also direct drivers to the parking facility and to vacant sections of the facility. Signage may be located adjacent to arterials and freeways near the parking facility, at facility entrances, and inside the facility.

**Wayside/In-Station Traveler Information**

This concept provides real-time arrival/departure information at transit stops and station platforms. Information can be displayed on monitors or DMS signs. Information displayed on signs can also be announced simultaneously over integrated speakers or a station’s public address system. Advertising and general public service announcements can also be provided over the system.

**In-Vehicle Traveler Information**

An in-vehicle traveler information system provides visual and audio announcements inside the transit vehicle automatically. Typically, announcements include next stop, major cross street, transfer point, and landmark information. Additional information, such as public service announcements and advertisements, may be provided at other times. An in-vehicle traveler information system meets Americans With Disabilities Act (ADA) requirements.
Electronic Payment Concepts

The electronic payment concepts recommended for implementation in the Corridor allow travelers to pay transit fares, parking fees, and tolls by electronic means (i.e., magnetic stripe card, smart card, and transponder). The concepts can be integrated into one universal system, and customers can be issued one account. The electronic payment concepts increase customer convenience (e.g., exact change not required, simplification through a single account), allow for cost savings to customers and transportation agencies, and save customers’ time. Coordinated electronic toll collection, electronic fare payment, and parking facility electronic payment in the Corridor will provide the nation’s first multimodal, integrated electronic payment system. The electronic payment concepts are discussed below.

Electronic Fare Payment (EFP)

This concept provides an electronic means of collecting and processing fares. Customers use a smart card or magnetic stripe card instead of tokens or cash to pay for transit rides. The electronic fare payment system may be linked to the transit vehicle tracking system for distance-based fare collection.

Parking Facility Electronic Payment

This concept collects parking fees electronically and detects and processes violators. Payment may be made using a credit/debit card, smart card, or vehicle-mounted transponder.

Safety & Security Concepts

The safety and security concepts provide surveillance in transit vehicles, in transit stations, at transit stops, and in parking facilities. Surveillance components consist of video, silent alarms, and two-way intercoms. The safety and security package deters vandalism and other criminal activities. This creates a safer environment for transit patrons and reduces maintenance costs due to vandalism. The safety and security concepts are discussed below.

Onboard Transit Security

Onboard transit security provides video monitoring of the passenger safety environment onboard the transit vehicle. Video images may be recorded and later reviewed, or they may be transmitted in real time over the bus’s communications system to a central location. A silent alarm feature allows transit drivers to request assistance from dispatching in case of an emergency. Often there is a direct link of this feature to the authorities. The onboard transit security system is usually linked to the transit vehicle tracking system. Therefore, a vehicle can be quickly located during emergencies.

Transit Facility Security

Transit facility security provides remote, real-time video monitoring and recording of the passenger safety environment at transit stops and in stations. It allows passengers to request assistance via a two-way intercom system in case of an emergency. A direct link is often provided to the authorities.

Parking Facility Security

Parking facility security provides remote, real-time video monitoring and recording of the safety environment in parking lots and garages. It allows patrons to request assistance via a two-way intercom system in case of an emergency. A direct link may also be provided to the authorities.

Operations Concepts

The operations concepts improve the operations and maintenance functions of the transit system. They control access to and automate docking at BRT stations, monitor vehicle mechanics and manage maintenance, provide priority to buses at traffic signals, and assist in the dispatching of transit police. In short, the operation concepts improve transit travel times by reducing dwell times at stations and traffic signal delays, improve equipment reliability and reduce the number of delays due to equipment failure, and improve response time in emergencies. Each of the operations concepts is discussed below.

BRT Station Lane Access Control

This concept limits access at BRT stations to BRT buses. It prevents passenger vehicles and trucks from accidentally traveling on the entrance ramp to a BRT station. A gate, located at the front of the entrance ramp, is used to control BRT station access. The gate opens as a BRT bus, equipped with a transponder, passes a transponder reader upstream from the BRT station entrance ramp.

BRT Precision Docking System

A precision docking system assists drivers in correctly placing a transit vehicle at a stop or station. For example, the system would automatically position a bus, or assist a bus driver in positioning a bus, at a BRT station during a stop. The system designates where the bus must stop along the station platform.

Transit Vehicle Mechanical Safety Monitoring and Maintenance

This concept automatically monitors the condition of transit vehicle engine components, via engine sensors, and provides warnings if failures occur. The system may be linked to the transit vehicle track-
ing system to log the location and time of an incident, and to transmit real-time data to the transit management center or depot. This concept also provides vehicle diagnostics and manages the maintenance records of transit vehicles. It may simply consist of a computer spreadsheet to record and monitor maintenance activity or be a sophisticated computerized diagnostic system. Engine data, stored in the vehicle’s processor, may be downloaded onto the central system for analysis.

Traffic Signal Priority (TSP) Study

Traffic signal priority holds a traffic signal green, or turns it green earlier than scheduled, to provide right-of-way to transit vehicles. Signal priority is typically granted to transit vehicles running behind schedule. The number of passengers onboard the transit vehicle may also be used as a criterion in determining whether or not to grant the transit vehicle priority. This system can be linked to the transit vehicle tracking system to determine if the vehicle is running behind schedule. It could also be linked to an APC system to determine the number of passengers onboard the vehicle.

Emergency Response

The emergency response concept provides automatic location of transit police vehicles and computerized dispatching to assist dispatchers in deploying appropriate resources to an emergency quickly and efficiently. This concept may be coordinated with transit vehicle tracking/onboard transit security, transit facility security, and parking facility security. The emergency response concept included here is for the designated transit operator. The emergency response concept included in the coordination concepts ties the emergency response systems of several agencies (e.g., VDOT, Virginia State Police, and local police, fire, and emergency management services) together for inter-agency coordination.

MONITOR CONCEPTS

Some ITS concepts are applicable to the Dulles Corridor but are technologically immature. However, these technologies are expected to become more reliable and proven over time. A few other concepts are technologically mature but ranked lower than the implementation concepts during the prioritization process (i.e., APCs and platform screen doors). However, their deployment would still be beneficial for the Corridor and their ranking may improve in the future, depending on needs. These concepts should be monitored over time and should be implemented if it is determined that they are reliable and address the needs of and goals for the Corridor. The monitor concepts are shown in the outer ring of Figure 1.

Technology is developing at a rapid pace these days. For ITS, what may currently be the state of the art may be obsolete in five to ten years. Also, what is not reliable or proven today may be commonplace in the next decade. The market is a major driving force for this. A technology that can provide greater benefits at a reasonable price will tend to be more popular. Immature technologies tend to be costly. Research and development costs are recouped during the initial years of implementation. Over time, the cost tends to go down as the market for the technology increases and its use becomes widespread. In addition, cutting edge technologies may be more costly to operate and maintain than proven technologies.

Therefore, the above ITS concepts should be monitored during the life of the project and beyond. They have great potential and can provide significant benefits as costs eventually come down.

ACKNOWLEDGMENTS

The authors would like to thank the members of the Technology Task Group for their contribution to the project. Their input, insight, and guidance were very helpful.

REFERENCES

Mayday: Concept or Reality?

Hau L. To, Janelle L. Monette, and Farideh Amiri

Oftentimes intelligent transportation systems (ITS) is perceived as cutting-edge technology that will become fully realized in the near future. One aspect of ITS, mayday technologies, has already arrived. From commercial products backed by large automobile manufacturers (such as GM OnStar™ and Ford Rescu™) to aftermarket products (such as CERES™ and AutoGuard™), these technologies are being equipped in consumer vehicles across the nation. In an effort to determine the impact of emergency calls from such systems, the Minnesota Department of Transportation (Mn/DOT), Minnesota State Patrol 2100 (MSP), and Mayo Emergency Communications Center (MEC) have teamed up with Veridian Engineering to develop an emergency communications infrastructure capable of directly accepting mayday calls and intelligent enough to accurately route calls to the proper authority depending on the geographic location and the nature of the incident. The Mayday Plus system consists of the in-vehicle module (IVM), dispatcher interface and communications gateway. The IVM consists of cellular and global positioning systems (GPS) technologies and is capable of transmitting voice, location, and crash severity data. An example of crash severity data includes the change in velocity of the vehicle upon impact. The dispatcher interface is a personal computer that displays data received from the vehicle or other dispatch centers interconnected to the Mayday Plus system. The communications gateway is the key to the Mayday Plus system. The gateway automatically and logically routes data and voice based on the geographic location and the nature of the incident. The Mayday Plus equipment has been installed in test vehicles in the Rochester, Minnesota area. This paper reports the findings of the six-month Mayday Plus operational test.

OVERVIEW OF MAYDAY TECHNOLOGIES

The emergence of wireless technologies has revolutionized emergency response. The combination of cellular and global positioning systems (GPS) has allowed for access to valuable incident location information. This is commonly known as Mayday technologies. Several initiatives from around the country in recent years have focused on testing the operational feasibility of such technologies. States that have provided test beds for Mayday include: Washington, Colorado, New York, and Minnesota. The New York and Minnesota initiatives remain the only ongoing tests. While Minnesota’s operational test (Mayday Plus) shares similarities with prior initiatives, the project centers more on creating an emergency infrastructure capable of receiving calls not only from Mayday Plus devices, but from a variety of commercial products as well.

THE MAYDAY PLUS PROJECT AND COMPONENTS

The Mayday Plus project is spearheaded by the Minnesota Department of Transportation (Mn/DOT) in partnership with the Minnesota State Patrol (MSP) District 2100 and the Mayo Emergency Communications Center (MEC). Veridian Engineering and their proposed team were retained for system development and Castle Rock Consultants was selected as the independent evaluator.

The three primary components of the Mayday Plus system include the in-vehicle module (IVM), dispatcher interface stations, and Gateway.

In-Vehicle Module (IVM)

The IVM is composed of a cellular handset and antennae, GPS receiver and antennae, and Veridian’s “black box”. It automatically transmits crash data when preset thresholds are exceeded. The black box collects and transmits valuable crash severity data, such as indication of rollover, change in velocity upon impact, principle direction of force, and heading direction of the vehicle. Additionally, the data stream associated with each IVM includes telephone call-back number, driver, and vehicle information. MSP and MEC desired the additional information, since it was perceived as valuable in enhancing emergency response. In addition to automatic crash notification, the IVM allows users to manually send three types of distress signals: emergency, roadside assistance, and Good Samaritan. The intent of the Good Samaritan feature is to allow third-party witnesses to report a roadside incident.

Dispatcher Interface

The dispatcher interface allows emergency dispatchers to view and manipulate data sent from IVMs. Dispatcher interfaces have been installed at MSP 2100, MEC, and Rural Metro (a nationwide, private, third-party response center.) In addition to the data mentioned earlier, the location of the vehicle is also sent in the message string. The interface provides the dispatcher with a map of the vehicle location and allows forwarding of calls and faxing of data. It was developed in a Windows-based environment to increase usability as a result of most dispatchers’ familiarity with using Windows-based systems.

Mayday Plus Gateway

The Mayday Plus Gateway is a unique feature of the system. Cellular 911 call handling protocols for the state of Minnesota are quite
specific. While the State Patrol initially handles all cellular 911 calls, incidents occurring on county or city roads are forwarded to the appropriate local law enforcement center (LEC) according to the location of the event. Additionally, MEC provides different medical response services according to predetermined boundaries. MEC emergency services are not always dispatched to the scene of an incident. They operate according to primary, secondary, and air flight boundaries.

In order to develop a system that would consider these jurisdictional issues, Mayday Plus call routing procedures required in-depth scrutiny. The Gateway serves as the brains behind the automatic routing of Mayday calls according to the location of an incident. For example, an emergency call data transmitted from an IVM located on a county road within the MSP 2100 district and MEC primary service area will be forwarded to both MSP 2100 and MEC call centers. MSP 2100 will initially receive the voice and forward it onto the LEC. MSP 2100 can also send any relevant data to the LEC via fax. The following diagram (Figure 1) summarizes the call routing procedures set up by the Mayday Plus system.

**FIGURE 1 Routing of calls (diagram provided by Veridian Engineering)**

Additionally, the Gateway automatically routes calls to the proper authority depending on the type of call (emergency, automatic collision, roadside assistance, or Good Samaritan). For example, if a roadside assistance call is transmitted from a location within MSP 2100, the data will be sent to MSP, but Rural Metro will receive the voice connection and will transfer the “voice” to the appropriate response center. Within the operational test, Rural Metro transfers the voice and sends a fax to AAA, a local roadside assistance company.

**THE MAYDAY PLUS OPERATIONAL TEST**

The operational test, scheduled for six months (24 weeks), began August 16, 1999 and runs through January 30, 2000. During this period, the independent evaluation will be performed. The evaluation consists of seven detailed test plans covering several areas of interest. The test plans pertain to:
- evaluating the perceptions of Mayday Plus; and
- market feasibility of Mayday Plus.

During the project definition stages, there were only a few commercial Mayday products available on the market. However, after an in-depth analysis of the market situation, it was determined that there were over a dozen different devices available. A decision to eliminate the last detailed test plan was made based upon these findings. The following sections outline some of the findings resulting from the test plans.

**System Functionality**

Multiple functions of the system were evaluated during the operational test. When the operational test began in August, initial interaction with the system showed that not all of the desired and previously agreed functions were fully operational. While this did not prevent actual operation of the system, it detracted from the perceived usefulness of the system by dispatchers. The Mayday Plus system received upgrades during October and November to resolve the majority of the preliminary technical issues identified by dispatch supervisors at MSP 2100 and MEC, Mn/DOT and the independent evaluator. For example, at the initiation of the operational test, incoming Mayday Plus calls appeared at the bottom of the call list, making it difficult to discern the most recently received call. This was resolved in the November upgrades. As of one month prior to the end of the operational test, the following issues remain outstanding:
- the data for archived calls cannot be fully retrieved. For example, a user cannot retrieve the map or address of the location associated with an individual call. Archived data can only be viewed in a tabular format and cannot be viewed individually. A fully operational system would need this functionality.
- the location of an incident is only archived when a fax of the call is sent. Archived information is accessed by dispatchers for reports and follow up to incidents. Location information is necessary for these reports.
- the date of information-only calls is not recorded in the archived data. This limits the users’ ability to easily retrieve call information from the system. Calls are difficult to distinguish from one another without this critical piece of information.

In addition to the issues noted above, a fully operational Mayday Plus infrastructure will need improved safeguards. For example, during a scheduled testing period on November 30, the system was “down” at Rural Metro. The driver performing test calls as well as MSP 2100 and MEC dispatch centers had no indication of the problem. When the driver activated the IVM during the period the system was experiencing technical difficulties, the IVM did not alert the driver of any complications. Furthermore, the IVM did not direct the driver to manually dial 911 or present the driver with alternatives. The problem was discovered when the driver performing the test calls manually contacted MSP and MEC about difficulties connecting to the dispatch centers. During this incident, it was difficult to contact technical help.

**Usability**

The Mayday Plus system was analyzed for usability of the IVM and dispatcher interface. The IVM consists of a handset similar to a
cellular telephone equipped with six additional buttons dedicated to various functions of the Mayday Plus system. The system is set up so that a user presses a button for a desired service. For example, if a driver ran out of gas on the freeway, the “ROAD” button could be pressed to connect the driver with a roadside assistance provider. The participants using the system did not experience any difficulties operating the IVM during the operational test.

The dispatcher interface went through an evolutionary process during the operational testing of the Mayday Plus system. One month prior to the end on the operational test the interface is still undergoing enhancements. Dispatchers have commented on the ease of use and simplicity of navigating through the system. Additionally, they have expressed that the system was easy to learn.

**Impacts on Public Agencies**

An issue that originally spurred the interest of the Minnesota State Patrol to participate in the Mayday Plus operational test was the proliferation of cellular 911 calls. As more and more consumers purchase cellular telephones, the number of cellular 911 calls handled by MSP escalates. What is perceived as a safety feature by the general public has proven to be a nuisance by Public Safety Answering Points (PSAPs). For any one vehicular emergency (according to a preliminary analysis of 250 cellular 911 calls) there may be up to a dozen Good Samaritan cellular 911 calls. Even after emergency personnel have appeared on the scene, drivers will continue to report the incident. This detracts from MSP’s ability to respond efficiently to other emergencies that may arise immediately thereafter. While public education is required to teach citizens when to responsibly make cellular 911 calls, a Mayday emergency infrastructure may quell some of the other concerns.

In addition to the cellular 911 concern, emergency response agencies have discussed the importance of the “golden hour” which is defined as the first 60 minutes after trauma occurs, in which the lives of a majority of critically injured patients can be saved if appropriate emergency response is provided. When dealing with rural traffic incidents, identifying accurate location information can consume precious golden hour minutes, which limits the amount of time trauma physicians have to save a victim’s life. One concern is the lack of accurate location information sent with cellular 911 calls. Preliminary results have indicated that the Mayday Plus system decreases response time to severe crashes. Additionally, dispatchers have stated that the system is especially useful in situations where the vehicle driver is unconscious or unsure of their location.

From a transportation agency standpoint, the safety of the traveling public is of utmost concern. In a study performed by Mn/DOT in 1994 (where rural travelers were asked to prioritize travel-related concerns with ITS solutions), Mayday technologies ranked high. The results of that study initiated the Mayday Plus Operational Test. Furthermore, interagency cooperation, particularly between MSP and Mn/DOT, is significant. Not only are these agencies typically located in the same building, MSP provides dispatching of maintenance vehicles. In addition, MSP dispatchers also input pavement condition reports into a Mn/DOT system for use by patrol and DOT personnel. An additional benefit of the Mayday Plus system to Mn/DOT is the ability to promptly handle incidents alleviating traffic flow problems associated with highway incidents.

**User Perceptions**

Castle Rock surveyed all of the public participants as well as the dispatchers involved in the operational test, and the overall perception was that Mayday technologies are beneficial. Generally public participants perceive the benefit of Mayday technologies as the ability to provide “peace of mind” while traveling. Also, 100% of the individuals surveyed stated that they would purchase a system like Mayday Plus if it were affordable. Although not every participant surveyed worried about getting assistance in case of an incident, 100% of the participants believed that the Mayday Plus system will allow faster emergency response. In addition, an in-vehicle Mayday device will make them feel safer and be simple to use.

**Institutional Issues**

The Mayday Plus project attempted to closely match actual emergency protocols and procedures. As a result, interagency cooperation between the Mn/DOT, MSP, emergency responders, and PSAPs were required. Institutional issues that were faced during the operational tested included: resolving varying agency concerns, the effect of reduced management involvement and staff turnover, procedural impacts, and improved coordination among interagency departments

**MAYDAY PLUS: CONCEPT OR REALITY?**

Mayday Plus is a concept progressing rapidly towards reality. Early operational test results have conveyed widespread user acceptance. The location information of successful test calls has proven to be accurate to a degree that is acceptable to its system users. While the system has proven to be a fully-operational stand-alone system, widespread deployment regionally and nationwide will require consideration and resolution of several key issues including:

- successful interoperability of commercial Mayday products. The Mayday Plus project has proven the successfullness of sending important crash data to emergency dispatch centers. Commercial Mayday products, while they function in a similar manner as IVMs, they do not provide a direct data link to emergency dispatch centers.
- cooperation from private national message centers and commercial product providers. National message centers and commercial product providers currently use databases that provide non-priority numbers into PSAPs. In the case of a life-threatening vehicular emergency, key emergency providers need to be notified immediately to provide optimal care.
facilitating the integration of technology at smaller PSAPs. Technology comes at a price. A comprehensive, nationwide emergency infrastructure would need to integrate with PSAPs. While full integration of Mayday dispatching technologies may not be feasible in the short-term, the inclusion of PSAPs nationwide should be included in the vision of a comprehensive emergency infrastructure.

Although the project evaluated functionality, usability, impacts on public agencies, and user perceptions of the Mayday Plus system, it was unable to test the integration of the system with commercial Mayday devices. This is, in fact, the key to a fully operational and functional Mayday emergency infrastructure. Without the ability to interface with various types of Mayday devices, the comprehensive benefits of a Mayday emergency infrastructure cannot be reaped.
A Proposed Modification of the Bridge Gross Weight Formula

CARL E. KURT

A study was conducted using 201 different truck configurations and the entire bridge system of one state to develop a proposed modification of the current Bridge Gross Weight Formula. A new proposed formula is presented that minimizes the concerns for long trucks with a large number of axles. The proposed formula is less sensitive to axle group length and the number of axles than the current bridge formula.

STATEMENT OF PROBLEM

The FHWA (1) developed a bridge formula to estimate the “equivalent” load of a generic truck. This formula has worked very well for truck configurations using the nation’s highways at that time. The generic truck was broken into groups of axles and the allowable weight for each group is calculated by the formula:

\[
\text{Group(N)} = 500 \left[ \frac{\text{L}}{\text{N} - 1} + 12\text{N} + 36 \right]
\]

Where

- \( L \) = Length of group
- \( N \) = Number of axles in group.

After the allowable weight is calculated for each axle group, the allowable weight for the vehicle is the sum of the allowable weights for each axle group. This equation requires numerous calculations depending upon the configuration of the vehicle.

Numerous researchers (2,3,4,5) observed the overall length of many trucks had significantly increased since the original bridge formula was developed. They also observed that the number of axles has also increased significantly. Both factors allow the trucking industry to significantly increase the allowable weight carried by long trucks. For states that use the original bridge formula to make permit decisions, there has been considerable concern on the impact these long trucks have on existing bridges.

The objective of this paper is to present the results of a study considering 201 different truck configurations on all bridges of a given state highway system. A total of 1,178 bridges were considered in the study. To aid in the study, an object oriented computer program was developed. A brief description of this algorithm will also be presented.

STUDY TRUCK CONFIGURATIONS

The 201 truck configurations considered in this study can be broken down into three classifications. The first classification, straight trucks, contains eight straight trucks with a single front axle and a combination of drive axles. The total number of axles ranges from 2 to 6 axles and the overall length of the trucks ranges from 12 to 45 feet. The second classification of trucks, combination trucks, consists of a straight truck plus a trailer, or “pup,” with a towbar. Fifty-six (56) different combination truck configurations were considered. Their length varies from 36 to 74 feet and each has between 4 and 16 axles. The final classification of trucks, triples, consists of a semi-tractor that tows two trailers in series. A total of 135 triple configurations were considered. Their length varies from 86 to 100.5 feet, and they have between 6 and 17 axles. A detailed description of each truck configuration can be found in Reference 6.

The distribution of weights between axles was determined by limiting single and tandem axle weights to 20,000 and 34,000 pounds respectively. The maximum axle weight for each axle group with three or more axles was calculated using the existing bridge formula. The percentage of total overall vehicle weight on each axle was calculated based on the allowable weight assigned to each axle of an axle group divided by the sum of axle group weights. In reality, the sum of the axle group weights is generally not the allowable weight for the vehicle but was used here to calculate a percentage of total weight for each axle.

BRIDGE DATA

While the author has received BARS data from two different states, the data used to modify the existing bridge formula came from only one state. In each case, the number of spans, total bridge length, length of each span and connectivity between adjacent spans was found in the data. A bridge operating rating, usually for an HS truck, was also available. With the operating rating of the rating HS vehicle and bridge geometry known, the envelope of allowable live load moments and shears was calculated at critical points along the bridge.

For the development of the modified bridge formula study, data for 1,178 bridges was considered. Posted bridges were not considered in the analysis because most states do not allow permitted trucks to travel over posted bridges.

BRIDGE LOAD RATING SOFTWARE

A bridge load rating program, PBRat, was developed, in Visual C++, using object-oriented programming techniques. With this technique, three objects were developed. One describes the bridge, another describes the truck, and the last one describes the influence line coefficients. Thus, the number of spans, length of each span, etc. became properties of the bridge object.

This approach to software development has proved to be very successful. On 200 MHz microcomputers, approximately 1,000 bridges per second per rating vehicle axle were analyzed. Thus, all
bridges on a system with 4,000 bridges could be load rated in less than a minute for a 17 axle permit truck.

RESULTS OF ANALYSIS USING THE CURRENT BRIDGE FORMULA

As PBRat analyzed the 1,178 bridges for the 201 truck configurations, the allowable truck weight, based on PBRat, was compared to the allowable load, based on the current bridge gross weight formula. The level of overload, the ratio of the actual allowable load from the current bridge formula, and the allowable load from PBRat were calculated as a percentage. In Figure 1, the maximum, minimum, and average level of overloaded was shown to increase as the length of the truck increased and as the number of truck axles increased. The level of overloading approached 100% (double the operating capacity of the bridge) for the very long trucks with large number of axles.

The number of bridges overloaded using the current bridge gross weight formula was also plotted as function of truck length in Figure 2. Again, the number of bridges overloaded was found to increase as the length of the truck increased. When the number of bridges overloaded as a function of the number of axles, similar behavior was observed. When the current bridge gross weight formula is used to evaluate today’s longer vehicles with many axles, it tends to...
allow more weight, therefore more bridges are loaded beyond their operating capacity.

**Proposed Modified Bridge Gross Weight Formula**

Several forms for a modified bridge formula were tested, including the Texas Transportation Institute (TTI) formula (7). However, the generic form of the one eventually selected is:

\[
W = 1000 \left[ \frac{C_1 L N}{(N + C_2)} + C_3 N + C_4 \right]
\]  

(2)

Where

- \(L\) = Length in feet
- \(N\) = Number of axles

\(C_1, C_2, C_3, C_4\) = Constants

\(W\) = Gross weight in pounds.

A study was conducted to determine the optimum value for the constants \(C_1\) through \(C_4\). After careful study, the best values for \(C_1\) through \(C_4\) were 0.5, -1, and 3, respectively. In each case, the number of overloaded bridges was nearly constant when plotted against the number of axles. Thus, the form of the proposed modified bridge formula becomes

\[
W = 1000 \left[ 0.5L N/(N - 1) + 3N + C_4 \right]
\]  

(3)

Where

- \(L\) = Length in feet
- \(N\) = Number of axles
- \(C_4\) = Constant for overloading
- \(W\) = Gross weight in pounds.
To help agencies assign the proper value of $C_4$ to the formula, the chart in Figure 3 was developed. In Figure 4, the average percentage of overloaded bridges is plotted as a function of $C_4$. As noted in the figure, if the agency decided that it was appropriate to overload approximately 5% of its bridges, the value of $C_4$ would be set to 33.

Figure 3 allows one to predict the number of bridges for the entire system overloaded for various values of $C_4$, it does not measure the number of bridges for each truck configuration with the same number of axles. The results of this study are shown in Figure 4. As seen in Figure 4, the average degree of overload is approximately constant with the number of truck axles.

While Figures 3 and 4 are helpful in setting the value for $C_4$, additional information is useful. For example, in Figures 5 and 6, the number of truck/bridge combinations or bridges that were overloaded by a known percentage for all truck/bridge combinations or the worst truck are shown as a function of the value $C_4$.

If one assumes that it is acceptable to overload 5 percent ($C_4 = 33$) of the bridges in a system, the level of overload is presented in Figure 7 for all bridges in the system. When compared to Figure 1 using the current bridge formula, one makes several interesting observations. For example, the impact of truck length is greatly reduced and the impact of axle length is also reduced. Also, the maximum level of overload was reduced from nearly 100 percent to only 64 percent. In Figure 8, the number of bridges overloaded using a $C_4 = 33$ is also presented. Again, in this figure, the impact of truck length is minimized and the number of bridges overloaded is reduced for all truck lengths.
IMPACT OF MODIFIED BRIDGE FORMULA

With the development of the proposed modified bridge formula, the user can control the conservatism used in adopting the variable $C_4$. In general, the modified formula reduces the allowable loads of trucks with large overall lengths and a large number of axles. At a value of 33, the modified formula reduces the long group lengths with large number of axles approximately 30,000 pounds. However, for the shorter trucks, the allowable gross weight is increased using the proposed modified formula.

SUMMARY AND CONCLUSIONS

Over 231,000 truck/bridge combinations were considered in this study to develop a proposed modified bridge gross weight formula to estimate the allowable load for a generic truck configuration. A formula is proposed that minimizes the impact of truck length and number of axles on the overall allowable gross vehicle weight. The degree of overloading is set by a new constant, $C_4$, which may be set by the agency.

ACKNOWLEDGEMENTS

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Evaluation and Repair of Damaged Prestressed Concrete Girder Bridges

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Every year, numerous prestressed concrete (P/C) girder bridges are damaged by overheight vehicles. When this happens, bridge engineers are faced with numerous questions relative to the behavior and strength of the bridge. These questions must be answered so decisions can be made concerning traffic restrictions and future maintenance actions. Results of an investigation of damaged P/C bridge behavior and damaged P/C beam strength are briefly presented in this paper. In this project, two P/C bridges carrying I-680 near Beebeetown, Iowa were tested. The westbound (WB) bridge was accidentally damaged and tested in the damaged state and following replacement of the damaged beams. The eastbound (EB) bridge was not damaged and was used as a reference. Following testing of the bridges and removal of the damaged beams, one of the beams was tested in an “as-removed” condition while the other was strengthened with carbon fiber reinforced polymer (CFRP) longitudinal plates and external CFRP stirrups. Both beams were tested to failure. Additionally, a three beam laboratory P/C beam bridge model was tested. The model had a total length of 40 ft-4 in. (12.29 m) and a width of 18 ft (5.49 m). The bridge model was tested 180 times to study the effects of incremental damage and load placement on the behavior of a controlled specimen. Following testing of the model, two of the beams were removed, the first was tested in an undamaged condition, the other following intentional damage and CFRP repair. Both beams were tested to failure. An additional component of the work was the development of several analytical models of the damaged and repaired bridges. Both three-dimensional grillage (downstand grillage) and stiffened plate models were created. The models were calibrated using the experimental deflections recorded during both bridge tests. The analytical models were used to describe the live load distribution patterns in both the damaged and undamaged bridges. Significant redistribution of moment away from the damaged beams to the adjacent undamaged beams and curb/rail section was observed. Key words: prestressed concrete, FRP, repair, finite element, load testing.

INTRODUCTION

In 1996, an unknown overheight vehicle struck the center span of a 3-span prestressed concrete (P/C) bridge carrying I-680 over County Road L34 near Beebeetown, Iowa. Due to concerns about the remaining strength of the two most severely damaged beams, unknown effect of the damage on the load distribution patterns in the remaining structure, and concerns regarding the durability and effectiveness of any proposed repair, it was decided that the beams would be replaced. Frequently the decision to replace a damaged prestressed beam is made because of a lack of knowledge about the reserve strength of the bridge rather than from calculations that definitively indicate that the bridge has been compromised. The damaged bridge provided an opportunity to perform an in-place assessment of load distribution in damaged and undamaged bridges. The damaged beams were eventually tested following their removal from the bridge to determine the effect of damage on their remaining strength and to determine the effectiveness of carbon fiber reinforced polymer (CFRP) strengthening techniques. An additional aspect of this research was analytical modeling of the I-680 bridges in the damaged and repaired conditions so that the effect of damage on load distribution could be quantified. The models were calibrated using the experimental results.

In addition to the tests conducted on the I-680 bridges, a 40 ft-4 in. (12.3 m) long and 18 ft (5.5 m) wide P/C bridge model was also tested. The model was damaged in small increments to record the relative changes in bridge behavior due to incrementally applied damage. A total of 180 tests were conducted with various load placements and levels of damage. Two beams from the bridge model were then tested as isolated specimens, one undamaged and the other following intentional damage and CFRP repair. Due to space limitations, results of the bridge model testing and tests conducted on the isolated beams removed from the model are not presented in this paper. For additional details concerning all aspects of this research, refer to Klaiber, Wipf, Russo, Paradis and Mateega (1).

The objectives for this project were as follows:

- Determine the load distribution patterns in undamaged and damaged bridges.
- Ascertain whether the live load distribution in damaged bridges can be predicted with reasonable accuracy.
- Establish the effect of damage on the remaining strength of P/C beams.
- Determine whether damaged beams be economically and effectively repaired with strength as the controlling factor.

BEEBEETOWN BRIDGE TESTS

The I-680 bridges cross county road L34 near Beebeetown, IA (see Figure 1). The bridges are asymmetric three-span bridges designed by the Iowa DOT in August 1965. The spans are 43 ft-1 ½ in., 56 ft-3 in., and 47 ft-3 ½ in. (13.14 m, 17.14 m, and 14.41 m) long from east to west between the substructure centerlines. There are eleven beam lines in each structure. The first seven beam lines adjacent to the median are on 5 ft (1.52 m) centers, typical of Iowa DOT practice at the time these bridges were designed and constructed. To account for a ramp taper on both the eastbound (EB) and westbound (WB) bridges, there are also flared beam lines. These beams are spaced at 3 ft-6 in. (1.07 m) centers as a minimum and flare out to 5 ft (1.52 m)
on center at their widest point. The measured thickness of the slab-in-place including overlay exceeded 9 in. (230 mm) when the damaged beams were removed from the WB bridge; the overlay was approximately 3 in. (75 mm) thick.

Experimental Results

A large number of static load tests were conducted on the dual bridges in question. As previously noted, the WB bridge was damaged by overheight vehicle impact while the EB bridge was undamaged. An initial series of 43 static tests were conducted on each bridge to characterize the response under load and to determine the effect of various load placements on the relative distribution of load in the bridge. An additional 35 tests were conducted on the WB bridge following replacement of the damaged beams. Four test lanes were established. Lane 1W/1E was located so that the truck was directly over the flared beams (damaged beams of the WB bridge) and as close to the rail as possible (see Figure 3). The second lane, Lane 2W/2E is parallel to Lane 1W/1E but offset laterally by 12 ft (3.6 m). The third lane, Lane 3W/3E is located adjacent to the centerline of the through traffic lanes while the fourth lane, Lane 4W/4E, is adjacent to the median railing. In addition to the various load placements, a variety of data was collected from the bridges including quarter point and midspan deflections, as well as strains on the exposed strands of the damaged beams, on the diaphragms, and at the ends of the beams.

The impetus for this research project was the collision of an unknown vehicle with the north three beams of the overhead WB structure in July 1996 (see Figure 2). Damage was centered ±5 ft (1.5 m) west of the midspan diaphragm of the center span. The damage to the WB bridge was such that approximately 6 ft (1.8 m) of the bottom flange was spalled or fractured from the north fascia beam, Beam 1W, exposing numerous prestressing strands. Several of the strands in Beam 1W seemed to be lax; however, no strands were severed during this collision. There was a preexisting severed strand from a 1993 collision. There was less damage on the first interior beam, Beam 2W. Significant cracking of the bottom flange as well as fracturing of the core concrete was present in both beams, but to a lesser extent on Beam 2W. Web cracking spread over the west half of Beams 1W and 2W. Cracking seems to have been arrested by the midspan cast-in-place concrete diaphragm. The second interior beam, Beam 3W, was also damaged, but not as severely, with the damage consisting of the spalling of a patch installed following prior collisions with the bridge.

For the tests conducted in Lane 1W/1E, the position where the test truck is closest to the edge beams on the flared side of the bridge, the center span data indicate a different deflected shape in the WB and EB bridges with the damaged WB bridge deflecting more over a number of beam lines including those known to be undamaged. The deflections indicate that load is "shed" from the damaged beam lines in the WB bridge; this was later confirmed analytically. The maximum center span midspan deflection in the WB and EB bridges was measured to be 0.064 in. (1.6 mm) and 0.053 in. (1.3 mm), respectively. Diaphragm strains in both bridges were small during the Lane 1W/E loading, the maximum being approximately +15µε in both bridges, and agree with the findings of others that the diaphragm plays an insignificant role in live load distribution. Replacement of the damaged beams results in the WB repaired bridge behaving essentially the same as the undamaged EB bridge, thus, the change in behavior of the original WB and EB bridge data can be directly attributed to the presence of isolated main member damage.

The correlation between the effects of two trucks placed in adjacent lanes or in the same lane (i.e., multiple trucks on the bridge) is excellent compared to the effects of linear superposition of individual truck test results. The most significant deflection for Beams 1 and 2 in both bridges occurred when the two trucks were placed end-to-end in Lane 1, L1W/E-P4&P6 (see Figure 4). The deflection of the
The center span of the WB bridge was 0.085 in. (2.16 mm), approximately L/7,900 and was 0.083 in. (2.11 mm), L/8,100 in the EB bridge. Other beams in the WB bridge deflected more than their counterparts in the EB bridge by a small amount, i.e., 0.01 in. (0.25 mm) for most of the beams. The beams in both bridges typically deflect as if they are simply supported and have uniform stiffness or only slightly non-uniform as for the WB bridge. The maximum exposed strand strains were also recorded during the test with two trucks in Lane 1. The maximum strain recorded in Beam 1W was +186 µε and in Beam 2W, +169 µε. These strains correspond to a stress range of approximately 5,300 psi (36.5 MPa) and 4,800 psi (33.1 MPa) in the two strands, respectively. These stress ranges are small and represent a stress range of less than 2% of the ultimate strength of the strand.

Analytical Results

In order to further understand the experimental results and quantify the effect of damaged beams on the load distribution pattern of the WB bridge, analytical models were created and calibrated using the experimental data. First, a series of undamaged models were created to predict the response of the repaired WB bridge and the undamaged EB bridge, which as previously mentioned, behaved essentially the same. Once comfortable with the correlation, damage was introduced into the model so that the analytical and experimental behaviors were in agreement. The results of these analyses indicate the difference in response of the damaged and repaired bridges.

Figure 5 depicts the analytical model and an example load placement for the repaired WB bridge. The figure depicts a stiffened plate model created using the STAAD-III software program. The deck was modeled using a shell element while the beams were modeled using eccentrically linked beam elements having the properties of the P/C beams. Loads were applied as point loads on the surface of the deck. In addition to the stiffened plate model, three-dimensional grillage models were also created in order to test the ability of different modeling techniques to capture the experimental response. Both types of models were able to reasonably simulate the experimental behavior. For the load case of two test trucks placed side-by-side in adjacent lanes as close to the damaged beams as possible, the analytical model predicts a live load moment in the most heavily loaded beam, Beam 2W, of 191 ft-kips (259 kN-m) as opposed to 252 ft-kips (342 kN-m) using the AASHTO distribution factor of S/5.5 for multi-lane loading. This demonstrates the significant conservatism of the AASHTO formulas for this bridge. A significant amount of moment is carried in the curb and rail section adjacent the loaded edge of the bridge.

Following the creation of the undamaged models, a damaged model was created using the “best” undamaged model as the starting point. Simulation of the amount and extent of damage is crude at best in that it is difficult to truly describe the extent and effect of the properties of the remaining beam. A simple procedure of ignoring a portion of the bottom flange based on visual inspection of the extent and severity damage was able to reasonably simulate the effects of the damage on the bridge performance. A comparison of the transverse deflected shape of the bridge as recorded in the field and predicted analytically is presented in Figure 6. The figure presents the measured deflection at midspan of Beams 11, 9, 7, 5, 3, 2, and 1W of the damaged WB bridge with two test trucks side-by-side with their rear tandems centered at midspan. The excellent agreement of the experimental and analytical results is apparent. Further examination of the
analytical results indicates that for the isolated main members in this bridge, that is to Beams 1W and 2W, for loads placed over the damaged beam lines, a significant redistribution of moment occurs, much of it being taken up by the adjacent curb and rail as well as the nearest undamaged beam lines. The amount of load carried by beams remote from the damage is insignificant. The primary means of load redistribution is via transverse flexure of the slab.

FIGURE 6 Damaged WB bridge experimental vs. analytical deflections; L1 and L2-P5

ISOLATED BEAM TESTING

The second important aspect of this project was to determine the feasibility of using CFRP laminates to restore the strength and to a lesser extent stiffness properties to damaged P/C beams. Although the experimental and analytical work conducted on this bridge indicated that in all likelihood the beams did not need to be removed for strength considerations, there may be instances in which structural strengthening is needed. The use of CFRP materials was seen as an attractive solution to the repair problem due to their high strength/weight ratio and their ease of installation. Long-term environmental performance of the materials was not studied.

Beam 1W and 2W were removed from the I-680 bridge in Beebeetown in October 1997. Following their removal, they were transported to the structural testing facilities of the University of Nebraska at Omaha located at Wilson Concrete in LaPlatte, Nebraska. Beam 1W was tested “as-is” as a baseline specimen while Beam 2W was to be intentionally damaged further and then repaired using CFRP materials. A photograph of Beam 1W in the test frame is presented in Figure 7.

FIGURE 7 Beam 1W in test frame; damage under left (west) actuator

Beam 1W (Baseline) Testing

At the time of the test, Beams 1W and 2W were inspected for the first time since the original damage inspection approximately two years prior. Following the initial damage and eventual removal of the beams from service, a protective tarp was installed to prevent loose debris from falling onto the county road under the bridge. It is known that the beams were hit several more times prior to removal due to the presence of several tears in the tarp. The amount of additional damage could not be documented. The notes from one of the researchers written the day of the test indicated that it was possible to see completely through the web. “With a minor amount of effort it would have been fairly easy to create a large void in the web simply by removing the fractured concrete.” The researcher goes on to communicate that “the tension region damage is extreme…one load point is right over the damage. The damage extends over three stirrups and most of the three stirrups are exposed…I think this will be the source of failure in this test.” These comments indicate damage much more severe than apparent from the initial inspection photos and description.

A baseline service load test and an ultimate load test were conducted on this beam. Prior to testing, the beam had one initially severed strand and an additional strand was intentionally severed in an attempt to measure the effective prestressing force in the strands. The service load test applied a maximum constant live load moment between the actuators of approximately 718 ft-kips (973 kN-m), approximately twice the design live load and impact moment for this beam during service. The load-deflection response of the beam was linear during the service test. Upward movement of the neutral axis was noted at midspan during the test, likely due to the influence of cracks. The neutral axis location at the undamaged quarter points was relatively constant throughout the service test. It is notable that when Beam 1W was still part of the I-680 WB bridge, the recorded exposed strand strain was +150 µε under the action of a single truck producing a moment of 655 ft-kips (888 kN-m) distributed to several beams. The corresponding deflection was 0.064 in. (1.63 mm). The strain and deflection in the isolated beam under a similar directly applied moment are +1,920 µε and 0.92 in. (23.37 mm), an increase of 12.8 and 14.4 times, respectively. The in-situ strains and deflections are only a small fraction of those measured in the isolated beam under similar applied moments. This dramatic increase in recorded strains in the isolated beam demonstrates the inherent load distribution and system redundancy in the damaged WB bridge.

Figure 8 depicts Beam 1W at the ultimate applied load. At ultimate load, the live load moment was 2,067 ft-kips (2,802 kN-m) and the midspan deflection was 8.62 in. (219 mm); the response of the beam was ductile. Failure of the beam was ductile. Failure of the beam was through the development of a large shear crack under the actuator placed over the damaged region of the beam. At ultimate load, the overlay partially debonded from the original slab. The flexural strength of this beam was 14% greater than that predicted by standard AASHTO code equations for a beam with three severed strands.

Beam 2W (CFRP Strengthened) Testing

The purpose of testing Beam 2W was to determine the effectiveness of CFRP retrofit techniques on the strength of a damaged and repaired P/C beam. Beam 2W was first further damaged by severing several strands then patched with a cementitious patching material,
FIGURE 8 South face of Beam 1W in the damaged region following the ultimate load test

CFRP longitudinal plates to replace the tensile capacity of the severed strands, and CFRP stirrups to help maintain bond between the longitudinal plates and the concrete (see Figure 9).

FIGURE 9 Beam 2W after repair with CFRP longitudinal plates (not shown) and CFRP stirrups

Following a series of service load tests in which it was determined that the repair had stiffened Beam 2W considerably as compared to its pre-retrofit response, an ultimate load test was conducted. It should be noted that in addition to the difference between Beam 1W and 2W in terms of one beam being damaged and the other repaired, Beam 2W had a considerably narrower composite slab, and the slab was more heavily damaged than in Beam 1W. For these reasons an exact A vs. B comparison is not possible for the two beams.

During the ultimate load test, the load vs. deflection response for repaired Beam 2W was substantially stiffer than for the baseline specimen Beam 1W (see Figure 10). This is due to the repair. At a load per actuator slightly more than 105 kips (467 kN), a midspan/damaged region live load moment of 2,480 ft-kips (3,362 kN-m), the beam failed catastrophically. Beam 2W completely collapsed at ultimate load. Although the failure of the beam was catastrophic, significant inelastic behavior and large deflections preceded the failure. The mode of failure is somewhat analogous to that observed in Beam 1W, failure of the beam directly under the west load point in the damaged region. At the time of failure, the overlay in this beam had partially debonded and spalled. However, this was not until the ultimate load was approached. The failure was sudden so a primary cause of failure was not observed though it appeared from assessment of the beam following the test that a combined shear/compression failure occurred in the damaged region. Up to near collapse, the longitudinal plates appeared to be well-bonded to the beam. The numerous external confinement stirrups were effective at ensuring bond between the plates and beam though numerous sounds were heard throughout the load test as the epoxy bonding the stirrups and plates to the beam cracked. It appears that at maximum load, the plates began to debond. This is evident by the decreasing strain in the plates following maximum load. The strains in the three longitudinal 5 in. x 0.08 in. (127 mm x 2 mm) CFRP plates bonded to the underside of the bottom flange were an average of +7,200 µε, approximately 85% of the failure strain of the cured laminate. The composite stress in the laminate was computed to be 122,400 psi (844 MPa) and the total force in the three plates, close to 147 kips (654 kN), equivalent to the capacity of approximately 3.5, ½ in.φ (12.7 mm), 270 ksi (1860 MPa) prestressing strands. Having only removed 2 strands from the section, this retrofit attained its design goal of replacing the lost tensile capacity of the damaged strands. A lesser amount of CFRP would have also achieved this objective and ensured a greater amount of displacement ductility.

FIGURE 10 Load vs. deflection response; repaired Beam 2W vs. baseline specimen 1W

CONCLUSIONS

A sample of the results from a significant research program conducted at Iowa State University in the past several years has been presented concerning the field testing of damaged bridges and the performance of damaged beams. The results indicate the following:

- The damaged WB bridge behaved differently from the companion EB bridge and from the repaired WB bridge. The response of the repaired WB bridge and undamaged EB bridge is similar with the differences in response between these bridges and the damaged WB bridge attributable to the main member damage. Damage to the first two beam lines resulted in an observed experimental redistribution of load to beams otherwise undamaged.
- Analytical models of the repaired WB bridge were developed that correspond well to the measured response from the field tests. The analysis indicates that the moments in the most heavily loaded beam lines due to a variety of critical load placements are substantially less than those predicted by AASHTO equations. The models were then subjected to analytical representations of damage and the model calibrated to the field test results from the damaged bridge tests. It is observed that a significant amount of moment is redistributed away from the damaged beam lines, part being carried by the adjacent curb and rail and the rest by several other close
beams.

Following removal from service, two of the damaged beams were tested. Beam 1W, a control specimen, was not repaired but tested “as-is” to acquire the response of a damaged beam. The test indicated that the beam had sufficient strength to have remained in service. The failure was eventually in shear through the region of significant web damage, but only after the beam had deflected considerably. Beam 2W was repaired with CFRP longitudinal plates for flexural strengthening and external CFRP stirrups for bond and confinement. This beam was also tested to failure and attained a capacity, considering dead load effects as well, of approximately 12% greater than Beam 1W. The repair was successful in restoring strength and stiffness to the damaged beam. The fact that the failure was catastrophic is not to be misconstrued to imply there was no warning of impending failure. The significant deflections were clearly visible, and it must be remembered that there was no opportunity for redistribution of load as there would be had the beam been part of a complete bridge system.

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An Alternate Shear Connector for Composite Action

F.W. KLAIBER and T.J. WIPF

In Iowa, there are over 20,000 bridges on the secondary system. The majority of these bridges are the responsibility of the local county engineers who with limited budgets, frequently design and construct short span bridges with their own labor forces. The primary objective of the research presented in this paper is to perform laboratory testing on an alternative that counties can design and construct. This project involves testing and modifying a system of steel beams with concrete fill between them. This system has been used by several Iowa counties on low water stream crossings for more than 20 years. There is no reinforcing steel in this system or connection between the steel beams and the concrete. With the proposed modification which uses an alternate shear connector (ASC) for composite action, less materials will be required and longer spans will be possible. Key words: composite action, shear connector, shear strength, bridges, steel beams.

INTRODUCTION

This paper will provide an overview on the use of an alternate shear connector (ASC) for obtaining composite action. The ASC was conceived as a means of using an existing bridge alternative on longer span bridges. The existing alternative is referred to as a beam-in-slab-bridge (BISB).

In Iowa, there are a significant number of BISBs on low volume roads. As shown in Figure 1, the structure uses a series of steel W sections spanning between abutments. The majority of bridges constructed to date have used W10 and W12 sections on 2 ft (0.61 m) centers. Steel straps are welded to the bottom flanges to hold the steel beams in place while the concrete is placed; there is no reinforcing in this system or physical connection between the steel beams and the concrete. Plywood, placed between adjacent beams on the top surface of the bottom flanges, is used for form work. The width of the forms is made a few inches less than the beam spacing so that the concrete is in contact with the bottom flange when placed. Thus, even after the form work deteriorates there will be adequate bearing between the steel and the concrete. These structures have been used for spans varying from 20 ft (6.10 m) to 40 ft (12.19 m) with and without guardrails.

Upon reviewing this system, the authors determined that by making two modifications to the beam-in-slab system (adding composite action and reducing the weight of the structure), it would be possible to increase the strength of the system and use the system on longer spans. Also, if the top flanges of the beams were removed from the riding surface, the skid resistance of the bridge surface could be improved. In the BISB system, there is sufficient concrete to carry compressive forces without the contribution of the top flange of the steel beams. The two modifications investigated were: 1) developing composite action between the concrete and steel and 2) reducing the size of the concrete slab.

Leonhardt, et al. (1) have shown that by punching holes in the web of the steel sections, composite action between the concrete and steel can be developed. Concrete dowels are formed when concrete fills the holes. These concrete dowels resist the horizontal shear at the steel-concrete interface and prevent vertical separation of the two materials. Transverse reinforcement in some of the holes is required to confine the concrete to the steel. This type of shear connector has also been investigated by Roberts and Heywood (2) and is similar to Perfobond strips shown in Figure 2 that were tested by Oguejiofor and Hosain (3). Concrete on the tension side of the BISB (see Figure 1) is obviously providing minimal strength contribution and thus is essentially only adding to the dead load of the system. By replacing the plywood forms with sections of arched formwork (corrugated metal pipe or plastic pipe) as shown in Figure 3, the amount of concrete on the tension side can be significantly reduced. Pipe formwork of the appropriate diameter will provide the desired slab thickness (dimension ‘h’ in Figure 3). Coring holes through the web of the steel beam with steel reinforcement through several of the holes (i.e. the ASC) will make it possible to obtain composite action. Incorporating these modifications to the beam-in-slab system would make it possible to use this system on significantly longer spans with reduced cost. Even with these modifications, the simplicity of the beam-in-slab system permits it to be constructed by county work forces.

RESEARCH RESULTS

This investigation consists of several tasks to address the desired modifications: Static push-out tests for determining the strength of various types of ASC between concrete and steel; cyclic load tests of the ASCs for determining their fatigue characteristics and slip behavior; use of ASCs in composite beam tests for strength and behavior data. In the following sections, each of these tasks will be briefly described.
Static Push-Out Tests

Dimensions of the push-out specimens are shown in Figure 4. As shown, each specimen consisted of a stiffened steel plate 3/8 in. (10 mm) x 20 in. (510 mm) x 15 in. (380 mm) and two concrete slabs: 8 1/4 in. (210 mm) x 21 in. (535 mm) x 20 in. (510 mm). In each specimen, the steel plate and reinforced concrete slabs were positioned so that the contact area was 17 in. (430 mm) x 2 1/2 in. (65 mm) in each slab for a total contact area of 85 in² (54,840 mm²). Load was applied to the steel and transmitted into the concrete slabs through shear; voids at the bottom of the steel plates (see Figure 4) prevented transfer of force by bearing.

Eleven series of push-out tests were completed. Variables investigated include hole diameter, hole spacing, hole alignment, addition of reinforcement through a hole, reinforcement size, and cored vs. torched holes. The configuration of holes in the steel plate and presence of reinforcement, etc. in 10 of the 11 series are illustrated in Figure 5. Series 7 involved plain steel plates and thus has not been included in this figure. In reviewing Figure 5, one notes there are three series without reinforcement (Series 1, 2, and 3) and two series with torched holes (Series 8 and 9). There were 3 specimens in each series except for Series 1 in which there were 6 specimens; thus, a total of 36 push-out tests were completed.

Slip and separation between the steel plates and concrete slabs were measured on all push-out specimens. The instrumentation for all specimens consisted of seven direct current differential transformers (DCDTs). Two of the DCDTs were positioned to measure slip between the concrete slabs and steel plates, four to measure separation at two elevations along the slab, and one for detecting lateral deflection of the stiffened plate. The two slip readings obtained were averaged to produce the average slip per connector. The load per connector is one half of the total load applied to the specimen. Separation between the concrete and steel was very small and thus could be neglected. Out-of-plane movement of the stiffener plate was also determined to be very small.

Typical load-slip data for some of the series tested are presented in Figures 6 and 7. Shown in Figure 6 is the load-slip data from Series 1 and 10; both these series have 1 ¼ in. (30 mm) diameter holes on 3 in. (75 mm) centers. The only difference is Series 10 also has a #4 reinforcing bar. Comparing the two curves in this figure one notes the increase in strength resulting from the added reinforcement. The effect of torching holes vs. coring holes in the steel is illustrated in
As indicated, poor quality torched holes have slightly less strength; however, carefully torched holes have essentially the same strength as cored holes.

From the results of the tests, it was determined that the strength of the ASC was influenced by five variables: concrete compressive strength, the friction between the steel plate and the concrete, the concrete dowel formed by concrete, the reinforcing bar placed through the shear hole, and the transverse slab reinforcing. It was also determined that hole spacing was not an important influence on the strength of the connection if the spacing was at least 1.6 times the hole diameter. Therefore, spacing of the shear holes was not included in the design equation that was developed, however, the design equation is only valid for hole spacings greater than 1.6 times the shear hole diameter.

In reviewing beam sizes that might be used for bridge stringers, it was determined that the smallest web thickness that might be encountered in the field would be approximately 3/8 in. (10 mm), which is the plate thickness used in the push-out specimens. Therefore, steel plate thickness also was not included as a variable; the expression developed will thus result in conservative strength values for web thicknesses greater than 3/8 in. (10 mm).

Using the experimental data from the various push-out tests, an equation (4) was developed for predicting the shear strength of the ASC. The primary factors that influence the shear strength of the ASC are the concrete compressive strength, the number and area of the shear holes, and the amount of transverse reinforcement.

**Fatigue Strength of the ASC**

Push-out specimens (see Figure 4) were also used to determine the fatigue strength and slip behavior of the ASC when subjected to cyclic loading. Three series of specimens were tested: Series FS1, which is the same as Series 10 in Figure 5i; Series FS2, which is the same as Series 5 in Figure 5e except #4 reinforcing bar used instead of a #3; and Series FS3, which is the same as Series 1 in Figure 5a except #4 reinforcing bar added to center hole. A total of 21 specimens were subjected to cyclic loading. Results from these tests indicate the ASC has adequate capacity for use in typical single span bridges. There was minimal strength gained by staggering the shear hole alignment (Series FS1 vs. Series FS2). Also, the reinforcement in full diameter holes (Series FS3) is more effective than reinforcement in half-circle holes (Series FS1).

**Use of ASC in Composite Beams**

To obtain additional strength and behavior information on the ASC, three full-scale composite beam specimens (two static and one fatigue) and two full-scale two-beam specimens representing potential bridge systems were constructed and tested. The ASC was used in all five specimens. As was previously noted, this portion of the investigation is still in progress, thus only information on the two beam specimens (Specimens 4 and 5) is presented in this paper. Both of these specimens were 34 ft (10.36 m) in length.

![Diagram of push-out specimen](image-url)
FIGURE 5 Details of hole arrangements used in push-out tests (1 in.=25.4 mm)
Specimen 4, as shown in Figure 8a, consisted of two W21 x 62’s with their top flanges embedded in an 8 in. (205 mm) concrete slab. As may be seen in this figure, the only reinforcement in the slab was #5 bars placed every 15 in. (380 mm) through the 1 1/4 in. (30 mm) diameter holes which were torched in both beams. The reinforcing system in Specimen 4 is based on the Canadian steel-free deck research (5). A steel-free deck obtains its strength through an arching type behavior of the concrete slab with the beams acting as the supports and the reinforcing steel providing the lateral restraint (tension ties). The transverse reinforcement in Specimen 4 has a dual purpose; it acts as a tension tie between the beams, and it contributes to the strength of the ASC. To obtain proper development of the transverse steel, it was necessary to provide the hooks illustrated.

Specimen 5, shown in Figure 8b, consisted of two W21 x 62’s fully embedded in a concrete arch system which spans between the beams. This specimen is more directly related to the modified BISB (shown in Figure 3) in that it incorporates removing some of the concrete on the tension side of the specimen and using the ASC for composite action between the steel beams and concrete. Transverse reinforcement in this specimen is #4 bars on 15 in. (380 mm) centers.

**SUMMARY AND CONCLUSIONS**

Based on the limited number of specimens tested in this investigation, the following observations and conclusions can be made:

- The strength of the ASC was determined to be primarily a function of shear hole area, amount of transverse reinforcement, number of shear holes, and the concrete compressive strength.
- The ASC was determined to have adequate fatigue strength for use on low volume roads.
- Composite beam specimens with ASC reached their ultimate capacity without any distress in the ASC.
- Modifications, which will not effect the ease of construction of the bridge, can be made to the existing BISB to make it possible to use the system in situations requiring longer span lengths.
ACKNOWLEDGMENTS

The research presented in this paper was funded by the Highway Research Board (TR-410) and the Highway Division, Iowa DOT, Ames, Iowa. The authors wish to thank numerous Iowa county engineers for their support, cooperation, and counsel and the several civil engineering graduate and undergraduate students who worked on this investigation. 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.

REFERENCES

Raised Medians and Economic Impact on Adjacent Businesses

William E. Frawley and William L. Eisele

The use of raised medians in urban areas has increased in recent years. Raised medians restrict access to businesses along a corridor by limiting turning movements to select mid-block locations. Therefore, a very common remark at public hearings related to the construction of raised medians is that there will be detrimental economic impacts on adjacent businesses. However, the restricted access allows more efficient signalization and traffic flow along the corridor, potentially providing more customers for the businesses. Although many studies on the effect on traffic operations exist, little research is available on the economic impact from raised medians on adjacent businesses and properties. The authors of this paper have completed three years of work on this project by developing and testing methodologies to collect and analyze data related to the economic impact of raised medians on adjacent businesses for the Texas Department of Transportation (TxDOT). This paper summarizes the findings of key economic indicators, as well as perceptions of business owners and managers. The research has found that installation of a raised median does not equate to economic losses by adjacent businesses. In fact, only two types of businesses (auto repair shops and gas stations) were found to generally experience losses in gross revenues. In almost all cases, employment increased in businesses surveyed. This research is anticipated to be valuable for transportation professionals in both the public and private sectors who must provide estimates and expectations of the economic impacts of raised medians.

INTRODUCTION

In recent years, transportation agencies have increased construction of raised medians on urban and suburban arterials. In addition to their use for access control, raised medians provide improved traffic operations and safety for a facility by separating opposing traffic flows and removing left-turning vehicles from the through lanes. With respect to access control, raised medians restrict left turns to mid-block and intersection median openings. While improving the operations and arterial signal coordination, the economic impacts of restricting these left turns may be felt by owners of businesses and properties adjacent to the arterial. Extensive research has investigated and quantified the costs and benefits of constructing raised medians with respect to initial costs and benefits to motorists in terms of reduced delay and increased safety. Prior to this research effort, however, limited research has been conducted to aid in estimating the economic impacts of raised medians on sales and property values for adjacent business and undeveloped landowners.

Research Methodology

Participants in the survey included owners and managers of businesses adjacent to the corridors of interest. The research team first conducted a “windshield” survey to determine which businesses and land uses were present along the corridors in which the survey was to be administered. Business information (e.g., address and contact name) for each location was then obtained from the chamber of commerce, appropriate neighborhood/business groups, county appraisal district office, and/or telephone directories. For all but one of the corridors, the research team sent a letter of support from the local chamber of commerce or neighborhood association encouraging the business owners and managers to participate in the survey. Finally, reminder cards were sent to the five case studies where mail-out surveys were administered to encourage individuals to return the surveys.

Corridor Descriptions

The case studies include corridors with a variety of business mixes. Most of the corridors are in suburban-type areas with shopping centers and strip retail development. One of the corridors, Grant Avenue in Odessa, is located in a central business district. The specific types of development on the individual corridors range from completely retail to a mix of office, institutional, and retail. These development mixes drove the numbers of potential survey participants on each corridor. In addition, the cities included in the study reflect a variety of population sizes. The populations range from approximately 35,000 in McKinney to approximately 1.8 million in the City of Houston. Table 1 summarizes several different characteristics of interest for each of the 11 case studies.

RESEARCH RESULTS

Impacts on Passerby Traffic

Changes in passerby traffic, or “impulse buyers,” are often of interest when considering the impacts of raised medians. A common perception of business owners, prior to construction, is that the raised median will restrict the amount of passerby traffic as motorists are required to take a more circuitous route to get to their business. It is interesting to note that the change in passerby traffic percentage observed by owners/managers is zero for those businesses that were present before, during, and after the raised median installation (group one). Conversely, the perception of those individuals on corridors where raised medians had not yet been installed (group two) ex-
expected an average of a five-percent increase in passerby traffic. In addition, those business owners that arrived during the median construction phase (group three) indicated that they expected a nearly three-percent decrease in passerby traffic. Finally, those individuals that arrived after the raised median installation (group four) indicated a perception that passerby traffic increased by 12.0 percent. The research also found some variances in passerby traffic observations and expectations, depending on the business type. Among group one businesses, it was observed that fast-food restaurants and other services indicated an increase in passerby traffic. Specialty retail, auto repair, and one gasoline station indicated a decrease in passerby traffic. Sit-down restaurants, medical, grocery, and durables retail businesses indicated no change in passerby traffic.

Importance of Access to Customers

One of the survey questions asked business owners to rank “accessibility to business” with other factors including distance to travel, hours of operation, customer service, product quality, and product price in ascending order that customers consider when selecting a business of their type. The results of this analysis by business group are shown in Table 2. Accessibility to store ranked fourth or lower for each business group. Further, some combination of customer service, product quality, and product price always ranked first, second, and third. This indicates that, according to the business owners, the most important of these elements used by customers to determine which businesses they will patronize are factors that may be controlled by business owners themselves.

<table>
<thead>
<tr>
<th>TABLE 1 Case Study Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Street Name</td>
</tr>
<tr>
<td>Texas Avenue</td>
</tr>
<tr>
<td>South Post Oak Road</td>
</tr>
<tr>
<td>Clay Road</td>
</tr>
<tr>
<td>West Fuqua Road</td>
</tr>
<tr>
<td>Long Point Road</td>
</tr>
<tr>
<td>Twin Cities Highway</td>
</tr>
<tr>
<td>9th Avenue</td>
</tr>
<tr>
<td>University Drive</td>
</tr>
<tr>
<td>Loop 281</td>
</tr>
<tr>
<td>Call Field Road</td>
</tr>
<tr>
<td>Grant Avenue</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TABLE 2 Relative Importance of “Accessibility to Store” by Business Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Business Group</td>
</tr>
<tr>
<td>-----------------</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>

Note: Business Group 1 = businesses present before, during, and after median installation; Business Group 2 = businesses present before the median construction and construction is yet to begin; Business Group 3 = businesses present during and after median installation; and Business Group 4 = businesses present only after the median has been installed.
Impacts on Employment, Property Values, Accidents, and Traffic Volume

Impacts upon employment, property values, accidents, and traffic volume are also of interest. Results of these factors by business group are shown in Table 3. For all business groups, after the construction period there has been at least a small growth in the number of full-time employees. Part-time employees decreased for business groups one and two after construction relative to before construction. Part-time employment also decreased during construction relative to before construction (i.e., “during” group is higher than the “before” group). Estimated property values were indicated as increasing 7.7 percent after the raised median installation by group one business owners, while the perception of the group two businesses was that there would be a decrease. The business owners also noted observing decreases in the numbers of accidents after the medians were installed. The group four businesses perceived that the number of accidents was likely higher by 6.7 percent. This is an interesting contrast to the group one business owners that were actually present before, during, and after the median installation. Finally, traffic volumes were indicated as higher after the raised median installation and lower during the construction, relative to before the construction, for all business groups.

Impacts on Customers Per Day and Gross Sales

Table 4 illustrates the impacts on customers per day and gross sales for the four business groups. “Gross sales where the median installed” refers to a question posed to business owners in which they were asked what they believe was/is the impact of the raised median for all businesses along the corridor where the median was installed. “Gross sales in the area” refers to a similar question that asked about gross sales for all other businesses in the area (not necessarily just the corridor) due to the raised median installation. One can quickly notice from Table 4 that the construction phase did seem to impact customers per day and gross sales as evidenced from the values in the “during” columns. Perceptions again seem to indicate a larger expected loss in the group two businesses, which predicted an 18.6 percent reduction, while those that were present before, during, and after the median installation (group one) noted a 10.7 percent reduction. Group one businesses also indicated an increase in customers per day and gross sales after the median installation while the group two businesses believed that there would still be a decrease. Group one also indicated an increase after the median was installed for all businesses along the corridor where the median was installed and in the community surrounding the roadway improvement.

<table>
<thead>
<tr>
<th>Business Group</th>
<th>Full-time Employees</th>
<th>Part-time Employees</th>
<th>Property Values</th>
<th>Accidents</th>
<th>Traffic Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>During</td>
<td>After</td>
<td>During</td>
<td>After</td>
<td>During</td>
</tr>
<tr>
<td>1</td>
<td>11.9%</td>
<td>0.1%</td>
<td>-2.3%</td>
<td>-3.3%</td>
<td>1.8%</td>
</tr>
<tr>
<td>2</td>
<td>-0.3%</td>
<td>0.3%</td>
<td>-0.2%</td>
<td>-1.0%</td>
<td>-8.2%</td>
</tr>
<tr>
<td>3</td>
<td>-8.3%</td>
<td>12.5%</td>
<td>-8.3%</td>
<td>0.0%</td>
<td>-7.0%</td>
</tr>
<tr>
<td>4</td>
<td>0%</td>
<td>7.1%</td>
<td>0.0%</td>
<td>6.3%</td>
<td>-15.6%</td>
</tr>
</tbody>
</table>

Note: Business Group 1 = businesses present before, during, and after median installation; Business Group 2 = businesses present before the median construction and construction is yet to begin; Business Group 3 = businesses present during and after median installation; and Business Group 4 = businesses present only after the median has been installed.

Note: The “during” column indicates impacts during construction relative to prior to construction, and the “after” column indicates impacts after construction relative to prior to construction.

<table>
<thead>
<tr>
<th>Business Group</th>
<th>Customers per Day</th>
<th>Gross Sales</th>
<th>Gross Sales Where Median Installed</th>
<th>Gross Sales in the Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>During</td>
<td>After</td>
<td>During</td>
<td>After</td>
</tr>
<tr>
<td>1</td>
<td>-12.1%</td>
<td>24.4%</td>
<td>-10.7%</td>
<td>0.2%</td>
</tr>
<tr>
<td>2</td>
<td>-0.5%</td>
<td>-5.9%</td>
<td>-18.6%</td>
<td>-0.8%</td>
</tr>
<tr>
<td>3</td>
<td>-16.7%</td>
<td>-8.6%</td>
<td>-20.0%</td>
<td>-0.1%</td>
</tr>
<tr>
<td>4</td>
<td>0.0%</td>
<td>50.0%</td>
<td>0.0%</td>
<td>0.3%</td>
</tr>
</tbody>
</table>

Note: Business Group 1 = businesses present before, during, and after median installation; Business Group 2 = businesses present before the median construction and construction is yet to begin; Business Group 3 = businesses present during and after median installation; and Business Group 4 = businesses present only after the median has been installed.

Note: The “during” column indicates impacts during construction relative to prior to construction, and the “after” column indicates impacts after construction relative to prior to construction.
Impacts on Customers per Day, Gross Sales, and Property Values by Business Type

Table 5 provides results of analysis for group one businesses that have been present before, during, and after the median installation. The table presents the average percent change, standard deviation, and sample size by business type. The data presented in the table indicate that the construction phase can have impacts upon customers per day, gross sales, and property values for many of the business types interviewed. It is interesting to note that business types such as specialty retail (e.g., clothing stores, bookstores, hobby-related stores, etc.), fast-food restaurants, and sit-down restaurants indicated increasing customers per day, gross sales, and property values after the median installation. The gas stations, auto repair, and other service businesses indicated decreasing customers per day and gross sales after the raised median was installed.

For mid-block, shopping center and specialty retail businesses, the number of full- and part-time employees was noted as being reduced after the installation of the raised median. The “before only” businesses of this type also had harsher expectations than experienced by those business owners present before, during, and after the installation of the raised median for property values, accidents, customers per day, and gross sales. These business owners also indicated a decrease in their customers per day during construction yet no change in their gross sales during the construction.

None of the corridors experienced decreasing gross sales after the construction phase except for McKinney, which experienced some decrease in gross sales the year following construction.

ADDITIONAL FINDINGS

It should be noted that the sample sizes upon which analyses were performed were often rather small; however, many observations and interesting points may be drawn from this research effort. The in-person surveys appear to provide more reliable data than the mail-out surveys, and these survey respondents appreciate the face-to-face opportunity to have their opinions heard. The average response rate for the in-person surveys was also much higher (62.0 percent) than the response rate for the mail-out surveys (9.0 percent).

When combining all business types, it was found that 93.6 percent of business owners whose businesses were present before, during, and after the median installation felt that their regular customers would be at least as or more likely to patronize their businesses. In contrast, those businesses that were interviewed prior to the installation of the raised median indicated this percentage slightly lower at 81.0 percent.

The construction phase seemed to impact customers per day and gross sales. Perceptions again seem to indicate a larger expected loss in the group two businesses (before only) indicating an 18.6 percent reduction while those that were present before, during, and after the median installation (group one) noted a 10.7 percent reduction. The “before” group also indicated an increase in customers per day and gross sales after the median installation while the “before only” businesses believed that there would be a decrease. Business types such as specialty retail, fast-food restaurants, and sit-down restaurants indicated increasing customers per day, gross sales, and property values after the median installation. Gas stations, auto repair, and other service businesses indicated decreasing customers per day and gross sales after the raised median was installed. (1)

REFERENCE

This entire paper is based on the following report:
Missouri: A Comprehensive Process for Developing a Statewide Access Management Program

DAVID J. PLAZAK, NORMAN BEEMAN, AND MAC FINLEY

The Missouri Department of Transportation (MODOT) is responsible for one of the largest state-jurisdiction road systems in the United States. Missouri has recently decided to embark on an access management program and has focused on utilizing access management mainly to meet safety, traffic operations, and economic development goals. The Missouri Access Management program development process involves a number of key steps. These include:

- Stakeholder identification and participation.
- Participant education on access management principles and impacts.
- Development of specific statewide goals for access management.
- Development of an easy to understand (and communicate) access management roadway classification system based on MODOT’s existing functional classification system.
- Development of a detailed set of access management standards and guidelines in the form of a guidebook.
- Development of administrative processes (such as the driveway permitting process).
- Identification of current and likely future access management problem corridors.
- Identification of promising “pilot” project corridors where access management principals could be applied.
- Access management awareness and training for stakeholder groups identified through a marketing plan.

This paper will provide an overview of the start-up and development of the Missouri access management program. It will also briefly cover a process for the identification of problem corridors using management information system data and geographic information systems (GIS) technology. This paper will be useful to other states and state DOTs wanting to address access management in a comprehensive fashion.

INTRODUCTION

(NOTE: The Missouri Comprehensive Access Management Planning Process is an ongoing project. All materials presented in this paper are subject to change.)

In all states, the roadway system plays a dual role. It provides service to through traffic while also providing access to adjacent properties, residences, and businesses. When these two roles are not properly balanced and managed, safety problems and operational issues result. These negatively impact both the traveling public and the adjacent landowners. Access management involves striking the proper balance between the dual roles roadways must play. This is done through the application of access management standards, which involve such features as spacing between driveways, driveway geometric design, internal circulation design for land developments, and installation of medians.

An extensive amount of access management research and programmatic activity is currently taking place in the Midwestern states. For example, Kansas is pursuing an aggressive corridor management program, while Minnesota and South Dakota are developing comprehensive access management programs. Iowa has commissioned several research projects designed to explore the relationships between access management and safety, traffic operations, and business vitality.

Missouri is the latest state in the region to begin working on an access management strategy.

The Missouri Department of Transportation (MODOT) is responsible for managing a far more extensive system of roads than its neighbors—over 30,000 miles in all. Unlike most other states in the Midwest, MODOT manages rural roads that are functionally classified as collectors and some routes that would be classified as local service routes in other states. Missouri’s “peer states” were identified based on the nature and extent of their road systems. These peer states are identified in Table 1 and were contacted to obtain their access management standards, classification systems, and administrative policies. States that are considered to be leaders in access management based on their presentations at the three past National Access Management Conferences were contacted for similar information.

Missouri’s State Constitution gives the Highways and Transportation Commission the authority to manage highway access:

“The highways and transportation commission shall have authority over all state transportation programs and facilities as provided by law, including but not limited to, bridges, highways, aviation, railroads, mass transportation, ports, and waterborne commerce, and shall have authority to limit access to, from, and across state highways where the public interest and safety may require.”(1)

Missouri has historically had a tax on motor fuel that is well below the average. This has led to a situation where Missouri’s roadways are replaced on a longer cycle that those in other states. This is important for access management for a number of reasons, not the least of which is that Missouri’s highways often have more curvature and greater profile change than other, nearby states. Combined with the rough topography of the state, this means that sight distance is often a major concern in locating driveways in both rural and urban areas. Missouri has not practiced access management in a comprehensive manner until now. Instead, it has largely approved or
disapproved individual driveway permits along its routes on the basis of desirable or minimum sight distance standards. Several types of variances to the sight distance standards have been issued at the district level in situations where only a minimum stopping sight distance standard could be met.

**PROJECT OBJECTIVES**

Missouri is taking a comprehensive approach to access management. Access management is being integrated into MODOT’s overall enterprise strategic plan. In particular, access management will be one of the most important strategies in the agency strategic plan for achieving improved highway safety. The main objectives of the Missouri access management comprehensive plan are to:

- Develop a comprehensive approach to access management in Missouri.
- Develop all necessary classifications, standards, guidelines, and administrative processes.
- Identify current and likely future corridors with access management problems.
- Provide access management training for the MODOT staff and other stakeholders.

**STAKEHOLDER ANALYSIS**

Key stakeholders for access management in Missouri were identified prior to the initial meeting for the project. Important groups to involve in the develop in access management planning and outreach for Missouri were: Missouri DOT District staff, Missouri DOT Central Office/Support Center staff from a variety of disciplines (including traffic engineering, right of way, planning, and highway design), land developers, economic developers, and city government officials. A key feature of the planning process involves the identification and involvement of local land use planning officials and private developers. These groups can either help or hinder the application of access management standards through their decisions.

**PLANNING PROCESS**

Separate oversight and technical committees were formed to guide the planning process. The oversight committee was established to:

- Provide high-level guidance for the study (e.g. setting goals)
- Direct the technical committee to address issues
- Discuss policy issues
- Consider different viewpoints, including business vitality, economic development, and land development, in developing the access management plan.

The oversight committee includes managers from various Missouri DOT divisions and district offices, plus experienced land developers and economic developers as well as city elected officials.

By contrast, the technical committee was established to:

- Develop technical standards and guidelines for access management
- Report these back to the oversight committee.

The technical committee is made up of Missouri DOT staff from several divisions and district offices plus local transportation planning and engineering professionals who are involved in access management.

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<table>
<thead>
<tr>
<th>TABLE 1 Missouri’s Peer States in Terms of State Highway System Extent</th>
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<tr>
<td>State</td>
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<td>Louisiana</td>
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Source: Federal Highway Administration
Notes for Table 1:
1/ Travel is estimated by FHWA; other data are for 1996.
2/ DVMT means Daily Vehicle Miles of Travel.
3/ AADT means Annual Average Daily Traffic. AADT/Lane is a system-wide average.
4/ Statewide totals for mileage, lane miles, and travel are found in HM-20, HM-60, and VM-2.
ACCESS MANAGEMENT GOALS

The following access management goals, shown in order of importance from highest to lowest, were set during an initial meeting by the oversight committee:

- **Increased safety.** Fewer crashes and lower crash rates are the main measures of success for this goal.
- **Improved traffic operations.** The expectation here is that access management can help reduce congestion, shorten travel times, improve mobility, and help protect the environment through salutary effects on energy use, air pollution, and land use.
- **Protection of the taxpayers’ investment.** Access management is hoped to be able to preserve past and present investments in expensive roadway assets and to defer the need for future investments.
- **Better operating conditions for non-auto modes.** Pedestrians, bicyclists, and public transportation users as well as motorists are expected to be beneficiaries of access management.

The MODOT access management project has already been closely integrated with the Department’s overall strategic plan. One of the main goals for the enterprise strategic transportation plan is safety. A strategy under safety in the enterprise plan is now to:

“Integrate access management at the local, regional, and statewide levels.”

The Division Engineers and the Traffic Division of MODOT have joint responsibility for this strategic element of the MODOT enterprise strategic plan.

CLASSIFICATION SYSTEM

Classification systems are a key part of the access management process. They allow access management standards to properly fit the present and future functional roles of highways. Classification systems are also useful for helping to explain access management concepts to the public and land and business owners.

Several other states’ access management classification systems were reviewed for applicability to Missouri’s highway system, current functional classification system, and jurisdictional arrangements. The technical committee adopted a system partially modeled on Colorado’s access management classification system. The main reason for adopting this system is that it is relatively simple to understand and explain; yet it reflects the continuum of roles that roadways must play. The proposed classification system is shown in Table 2.

DETERMINATION OF FEATURES TO BE MANAGED

A determination of features to be included in the access management standards for Missouri was made jointly by the oversight committee and the technical committee. The features for which standards are being developed are:

- Distance between interchanges on Interstates and other Freeways.
- Clearance of functional areas of interchanges.
- Distance between at-grade interchanges.
- Distance between traffic signals.

### TABLE 2 Proposed Missouri Access Management Classification System

<table>
<thead>
<tr>
<th></th>
<th>Urban</th>
<th>Rural</th>
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<tbody>
<tr>
<td>Interstate/Freeway</td>
<td>U1</td>
<td>R1</td>
</tr>
<tr>
<td>Principal Arterial (A)</td>
<td>U2</td>
<td>R2</td>
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<tr>
<td>Principal Arterial (B)</td>
<td>U3</td>
<td>R3</td>
</tr>
<tr>
<td>Minor Arterial</td>
<td>U4</td>
<td>R4</td>
</tr>
<tr>
<td>Collector</td>
<td>U5</td>
<td>R5</td>
</tr>
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</table>

R indicates Rural: the highway is not currently urban and is not in a 20-year forecast urban area.

- Driveaway spacing and density.
- Corner clearance and clearance of functional areas of intersections.
- Sight distance for driveaways.
- Driveaway geometrics and surfacing.
- Median openings.
- Guidelines for using two-way left-turn lanes, three-lane cross sections, versus raised medians.
- Dedicated right and left turn lanes.
- Frontage and backage road spacing from mainline routes.
- Parking on facilities.
- Accommodations of non-auto modes in conjunction with managing access.
- Connection depth (throat length) standards for major traffic generators.
- Designating features of interest (e.g., medians, lanes) for non-auto modes.

These standards are currently being developed by the technical committee for presentation to the oversight committee. In addition, the technical committee is developing a set of recommendations for local governments that have to do with matters that they control that impact access management. This set of guidelines includes such things as minimum lot frontages, encouraging joint and cross access, and avoidance of development practices such as “flag lots.”

PROBLEM AND PILOT PROJECT IDENTIFICATION USING GIS

An additional task of the planning process has involved the identification of problem highway corridors using geographic information system (GIS) technology and existing Missouri DOT safety management data. Right-turn and left-turn crash density and crash rates have been mapped statewide in Missouri using ArcView 3.1. Several of the maps produced are shown below in Figures 1 and 2. These maps are being used to identify places where access management retrofit projects would be most beneficial and also to identify places where past projects have had a positive impact.
ADMINISTRATIVE PROCESS

Once standards are in place, the next step will involve laying out an administrative process for applying them. A preliminary set of goals has been discussed with the oversight committee. These include:

- Making safe and operationally beneficial access decisions.
- Protecting the public investment in roadways.
- Providing a timely and predictable decision making process for landowners and developers.
- Encouraging uniformity of application of standards statewide, especially on interstates, other freeways, and strategic principal arterial routes.
- Making decisions based on clear and logical access standards.
- Allowing flexibility and engineering judgement where warranted (this can lead to stricter controls when they are needed).
- Keeping the number of variances at a reasonable level.
- Providing for an efficient appeals process.
- Setting good precedents for future access decisions.

Administrative process guidelines such as driveway permit fees, centralized versus decentralized decision making, and timelines for making permit and variance decisions will be established as a part of this phase of the project.

The concept of a hierarchy of features to be managed through the variance process has been adapted from a paper on variances presented at the second National Access Management Conference in 1996 (2). Some features, such as sight distance requirements, should be given the most scrutiny in reviewing potential variances since they are critical to maintaining a safe road system.

EDUCATION, OUTREACH, AND MARKETING

The Missouri access management project began and will end with education. The first completed task involved educating the oversight committee about the benefits and impacts of access management. National and regional information on access management and its applications was shared, and participants were encouraged to explore the variety of strategies available.

FIGURE 1 Left-turn and right-turn crashes in Missouri DOT District Five in the past three years
benefits were presented; in particular information from neighboring Iowa about the safety and business vitality impacts of access management was highlighted.

One of the last phases of the project will involve the development and use of educational materials designed to teach access management concepts and raise awareness. The educational materials will be targeted both internally within MODOT and externally to key stakeholder groups such as city officials, local land use planners, local transportation professionals, and developers.

RESULTS AND CONCLUSION

The Missouri DOT’s comprehensive access management planning process is ongoing. Considerable work remains to be completed. The success of Missouri’s access management plan will depend on three main factors. These include the ability to coordinate implementation within MODOT, the ability of MODOT to coordinate and cooperate with local governments on access management, and the ability of MODOT to persuade the development community of the value and importance of access management.

REFERENCES

1. Missouri Constitution, Article IV, Section 29, Highways and Transportation.
Access Management Programs in Selected States: Lessons Learned

WILLIAM E. FRAWLEY AND WILLIAM L. EISELE

The authors of this paper are currently investigating the development of access management programs in various states. This investigation is part of a research project to determine the legislative and regulatory requirements for the Texas Department of Transportation (TxDOT) to develop and adopt a comprehensive access management program. Researchers have interviewed officials from state DOTs in Colorado, Montana, Oregon, New Jersey, Michigan, and Wisconsin regarding their access management programs and other related practices, with particular interest in their development and implementation. This paper provides an overview of current access management programs in various states, explaining “lessons learned” during the development and implementation of the programs. Examples of the lessons learned include hiring a large enough staff dedicated to the program, creating a separate bureau/department/division for access management, and including a process to handle waivers. Specific recommendations from state DOT officials are also presented. This paper and presentation will be useful to states, provinces, and cities that are interested in developing or amending an access management program.

INTRODUCTION

As traffic volumes and congestion have increased in recent years, transportation officials have sought ways to protect their investments in arterial streets and freeways. The primary purpose of these facilities is the movement of vehicles. This purpose is in contrast to that of local streets, which are built to provide virtually unlimited direct access to businesses and residences. In order for arterial streets and freeways to operate most efficiently, access to and from those roads must be limited to specific points. This strategy reduces the potential conflict points of vehicles crossing lanes of traffic as they make turning movements into and out of driveways. The solutions to these problems are found in comprehensive access management programs. A comprehensive access management program includes tools such as driveway spacing, median treatments, auxiliary turning lanes, and grade-separated interchanges, as well as the policies for implementing these tools.

Several state DOTs around the country have established comprehensive access management programs. Certain states, such as Colorado, Florida, New Jersey, and Oregon, are well known for the success of their access management programs. These states have already completed the processes of creating, adopting, and implementing access management programs. Other states have begun to develop access management programs and are either proceeding with this work or have interrupted it. In all of these cases, there are valuable lessons to be learned by transportation agencies that are considering developing comprehensive access management plans. The “lessons learned” presented in this paper represent a variety of experiences and perspectives of transportation planners and engineers from around the country.

RESEARCH METHODOLOGY

While there has been very little research performed of this nature, there is a significant amount of documentation of various states’ programs, as well as the processes of and/or attempts to develop access management programs. In addition to conducting literature searches, research team members used professional contacts from previous related experience to gain additional knowledge of access management programs. These contacts provided at least basic background information about programs and the people involved with them.

Using information from the literature review and the original contacts, researchers began to investigate programs around the country, including programs both planned and under development. The research team considered each of the programs and identified several of these programs to develop into case studies. Case studies were developed by three means: personal interviews with state DOT staffs, telephone interviews, and literature review. Five states’ programs were targeted for in-depth investigations involving personal interviews with state DOT staffs at their offices.

RECOMMENDATIONS FROM OTHER STATES

Document Production

A common suggestion by DOT officials was to set out a work plan from the beginning. A work plan will help keep all parties involved in developing the access management program focused on the desired end results. DOTs commonly hire consultants to write laws, codes, and regulations as elements of their access management programs. One strong recommendation related to this practice is to also hire a good editor with quality technical expertise. These skills will provide consistency in wording throughout individual documents, as well as consistency among the various documents. Another related comment was to be careful about word choice. For instance, assigning an access management meaning to words if they already have another connotation can lead to confusion of all parties involved. In fact, “access” has been a difficult word for some agencies to technically define.
Implementation Timing

The transportation agency, including staff and administration, should not underestimate the amount of time that will be required to implement legislation. All parties need to understand this issue and allow time between the adoption of the legislation and the required implementation date. This interim time allows staff to properly develop the enacting regulations and procedures, as well as all of the detailed aspects, such as application forms and review checklists. The agency must also allow adequate time to hire and train staff.

Administrative Support

If a transportation agency is going to successfully develop and implement an access management program, there must be administrative support. The DOT administration must be patient and understanding of the time and resources required to establish an access management program. The bottom line is that the administration should at least allow, if not push for, the program development.

If the agency administration does not support the idea of an access management program from the outset, there are methods staff can utilize to sell the idea. From the beginning, there needs to be a consistent theme in the access management program that contains all of the necessary perspectives, including safety, design, right-of-way, etc. A consistent theme will provide a solid foundation for making decisions about the program.

At least one state has had success with having experienced people writing papers based on scientific information that provided supporting evidence of why access management is necessary and beneficial. In order to prepare such papers, the authors obtained numbers, such as accident rates and costs attributable to accidents (including property damage, injuries and fatalities). Additional support can be obtained by analyzing accidents related to intersections (including driveways) and by breaking out statistics between urban and rural roads. Such data should be tracked for several years. If possible, the author should compare accident histories of two similar roads built several decades ago—one with some type of median barrier and one without. Another issue to address is the cost of additional relief routes. This information is important when discussing the value of implementing access management techniques, in order to preserve the viability of existing or new roads.

Marketing Access Management

In addition to possibly needing to sell DOT administration on the idea of access management, it is necessary to market the benefits to other stakeholders as well. Marketing access management was a consistent theme among all of the DOTs interviewed in the research project. A long-time coordinator of one access management program stated that after many years he is still selling, still problem solving, and still acting like it’s a new program that is always under pressure. This interviewee added that in the early years, the best marketing tool was a set of a few hundred aerial photos, and a few ground photos showing the “good, bad and ugly.” Emphasizing the “bad”—this is the problem and access management is the solution—can be very influential when presenting access management to stakeholders. At the same time, it is important to keep in mind and show what good access management looks like—as if to say, “see, that doesn’t look bad, it’s not scary.” The person marketing access management should explain that it involves better decision-making and better utilization of current and proven engineering and design. Collecting and presenting accident-related statistics will also aid in marketing access management.

There are many opportunities to market access management to groups. However, there are also individuals and groups that may be more effectively targeted with printed materials. It is also constructive to develop a user-friendly document that most people can understand. Such a document needs to clearly explain the intent and contents of the access management program. Producing and distributing the document(s) will make the program development much smoother than it would proceed otherwise; it will help give the stakeholders the best opportunity to know exactly what is being proposed.

Program Operation/Maintenance

An access management program must have a full time specialist committed to it from the very beginning. This type of controversial, political, legal, and complex program will not run on its own. It will be one of the few regulatory programs within a DOT. One interviewee stated this idea very plainly by saying, “the program must have a specialist—unless you simply want a mediocre program with mediocre results.” The program needs a coordinator who can serve as the focal point for questions and concerns from everyone involved, as well as to ensure that the program develops and grows in a positive direction.

Once the program is up and running, it is vital to make sure there is cross-communication between project-oriented staff and permit-oriented staff. The coordinator of one well-established program reported having lost such cross-communication. This communication protects the specific interests of both parties. It allows the permit staff to know what is needed for certain road improvement projects, that would normally not be requested or necessary, and visa versa.

POTENTIAL BARRIERS AND OBSTACLES

While there are a myriad of barriers and obstacles that can and do present themselves when developing and implementing an access management program, interviewees in the research project mentioned several specific ones. Most, if not all, of these barriers and obstacles stem from two issues—money and people.

Money

Many officials’ experiences have shown that there will likely never be enough money to do everything in the best possible way, and there will always be competition for available funds. Being aware of the need for funding from the outset will stress the importance of proving the value that access management provides to the infrastructure and the motoring public. It is also important to keep in mind that political priorities internal to each agency will have great impacts on how funds are spent.
People

Staff

While the issue of money is relatively simple, there are several barriers and obstacles related to people. The consensus is that you need as many people as you can get. One person issue is similar to the general money issue—you need as many people as you can get. In addition to the dedicated access management program coordinator, there needs to be enough people to handle all of the work involved. People are needed for a variety of tasks, including processing permits and requests, reviewing sites and plans, performing legal work and research, and working with the public. All persons interviewed emphasized the need to have an adequate number of people on staff to handle access management issues.

Politics/Bureaucracy

Developing and implementing an access management program can be a politically sensitive issue, since it potentially affects many stakeholders. DOT officials interviewed stated the need to be aware of this matter so attempts can be made to not upset stakeholders, whether they are internal or external to the transportation agency. This can be accomplished by using appropriate, quality educational materials that explain all aspects of access management, including the benefits and costs. Program developers need to be aware of the specific concerns and lack of knowledge that stakeholders will likely have and be ready to address many issues as possible. Specially targeted efforts may be required in order to explain information to some people even though it is more easily understood by others.

In order to obtain and/or maintain internal administrative support, proper agency protocol must be respected. In some cases, it may be necessary to go through chains of command to talk to necessary people and make progress. This may occur in the implementation as well as the development of the program. Some examples of where protocol issues may be involved include obtaining authority for the access management coordinator to make decisions and requesting staff time from other divisions, departments, or agencies. More than one interviewee stressed that it is more work than one person can accomplish.

LEGAL ISSUES

There are a myriad of potential legal issues that may arise when developing and implementing an access management program. Decisions have to be made regarding legislation that authorizes and enacts the program. Other issues correspond to property rights, takings, and access rights. This section highlights a few of the concerns that were discussed in the interviews with state DOT officials.

Regulations

Writing clear, accurate and complete regulations in proper regulatory language and voice was suggested as a method to enjoy success related to legal issues. Testing all the ways the rules will be used, as well as running all the various scenarios to test the text and the standards is a way to ensure that this goal is met. One interviewee stated that the weaker the rule is, the faster it will be ignored.

Case Law

A state will not be able to change its case law. However, each state needs to understand its case law in order to write new law and regulation. A new access code/regulation will help change future decisions in case law. Knowing other states’ case law helps the state to understand the complexity.

It is important to have one attorney from the Attorney General’s office responsible for access management work. That way he or she will be able to learn a great amount about the engineering and planning issues that affect legal cases. Discussions with the Attorney General’s office, in order to determine who has authority if the State is going to give cities the right to review access management plans and related requests, are a vital part of the overall program. Clear rules related to these processes must be established and followed.

Waivers

Every access management program must be flexible enough to allow for situations that cannot be predicted and/or are out of the ordinary. It is not possible to create a specific rule or regulation for every potential scenario that may materialize. Therefore, the program must allow for waivers “on both sides of the counter,” for the public and for the transportation agency.

One concern that needs to be addressed is consistency among various waiver requests and responses. A suggestion to help provide some consistency it to establish a database in which all waiver requests and answers are entered. This will provide various application reviewers a means of referencing similar previous requests.

While it is necessary to provide flexibility through waivers, one interviewee emphasized the importance of keeping waivers to a minimum by stating that the Code is a tree and every waiver is a whack at the tree with an axe.

Another suggestion regarding the waiver process is to not include drawings, since they are difficult to amend. It was further stated that with such figures you not only bind the property owner, but you also bind the DOT.

“IF I COULD DO IT AGAIN”

One of the questions asked during the interviews was, “if you had it all to do over again, what would you do differently?” Some of these responses repeat points made previously, but are important enough to include in this section as well, since they were reiterated by the interviewees. These points were made more than once, and they may be some of the most important issues related to developing an access management program.

- Have more staff, a better developed program, and more money to
support projects to improve access locations with proven accident records.
- Spend more time on education.
- Start by trying to define what the law means (considering that we started with a law); a lot of issues have come up related to intent of the law.
- Broaden the stakeholders list.
- Establish where urban, suburban and rural standards begin and end. It is difficult to paint a suburban line on the ground.
- Develop the law and the program at the same time. In this way, all constituency groups are involved and laws and regulations are developed more smoothly. It would be beneficial to at least go a good way down the path with the two together.
- Make sure a reasonable time period is allowed for regulations to be adopted.
- Remember that the plan will not be perfect the first time. If you spend too much time trying to perfect it, you will never finish.
- Do not ignore highway projects. Make sure there is wording on how to implement the program other than through permits.
- Have actual legislation, instead of relying on the [State Transportation] Commission for everything.
- Develop an actual access management bureau or section within the state DOT to avoid as much political pressure as possible. Such a group would bring together staff with experience and expertise.

CONCLUSION

This paper has presented the majority of suggestions made by state DOT officials in states where access management programs are being successfully operated and in states where programs are being developed. The authors hope that these “lessons learned” will be useful to officials in cities, counties, states, and provinces where access management programs are being developed or refined. It is important to note that not every suggestion presented is applicable for every agency, but this collection of “lessons learned” provides a menu from which to choose.

SUE McNeil

Definitions of asset management commonly relate decision making for physical assets and the use of business principles commonly used in the private sector. New financial reporting requirements, issued by the Government Accounting Standards Bureau (GASB), have more closely linked these concepts for state Departments of Transportation, as they will soon be required to report the value of infrastructure assets in financial reports. Methods for assessing this value rely heavily on principles of asset management and supporting data. During the time frame that the GASB requirements have been under discussion, interest in asset management has also generated considerable activity in professional organizations—such as the American Association of State Highway and Transportation Officials and the American Public Works Association—agencies, and supporting organizations. This paper develops the relationships among the existing work on asset management, ongoing efforts on asset valuation, and the GASB requirements. This paper first reviews the GASB requirements and presents a rationale for their acceptance in terms of improved decision making and accountability and improved awareness of the need to preserve the existing investment. It then describes existing activities relating to asset management and asset valuation drawing on the following resources: 1) a survey of AASHTO member states and 2) an ongoing study for Transport Association of Canada focusing on measuring and reporting highway asset value, condition, and performance. The question “how well can the GASB requirements be met using asset management systems?” is then addressed. In conclusion, the paper presents a synthesis of the research related to asset management and asset valuation and makes recommendations regarding strategies for addressing the GASB requirements. Key words: asset management, asset valuation, financial reporting, infrastructure assets.

INTRODUCTION

There are many different definitions of asset management. The definitions have common elements related to decision making for physical assets and the use of business principles commonly used in the private sector. Asset management has received broad acceptance in the private sector (1) and is practiced in transportation agencies in Australia and New Zealand (2). In North America, most state departments of transportation are still struggling to determine what asset management means to them and are tentative as to whether this is an approach they want to adopt. However, external forces are influencing these decisions. Budget and legislative demands for better performance, public participation in the decision-making process, and regulatory requirements all point to the role asset management can play in the decision-making process.

In June 1999, the Government Accounting Standards Bureau (GASB) issued a reporting requirement that state and local governments show the value of the infrastructure assets that they own. Historically, public sector agencies have used revenue and expense reports and have not reported the value of their investments or assets. However, consistent with other business principles, there is considerable interest in moving to a balance sheet that includes assets and enhances public accountability. Also, asset valuation is a key element for evaluating success within organizations.

During the fifteen years that the GASB requirements have been under discussion, interest in asset management has also independently generated considerable activity in professional organizations—such as the American Association of State Highway and Transportation Officials and the American Public Works Association—agencies, and supporting organizations. This paper develops the relationships among the existing work on asset management, ongoing efforts on asset valuation, and the GASB requirements.

WHAT IS ASSET MANAGEMENT?

Asset management has been defined as follows (1):

“Asset management is a systematic process of maintaining, upgrading and operating physical assets cost-effectively. It combines sound business practices and economic theory, and it provides tools to facilitate a more organized logical approach to decision making. Thus, asset management provides a framework for handling both short- and long-range planning.” (2)

Other definitions of asset management place slightly different emphases on business strategies or go beyond physical assets. Examples include:

- “Asset management is a comprehensive business strategy employing people, information and technology to effectively and efficiently allocate available funds amongst valued and competing asset needs.” (3)
- “Asset management is a methodology to efficiently and equitably allocate resources amongst valid and competing goals and objectives.” (4)

Asset management clearly means very different things to different people. However, there is a unifying theme of efficiency. As state departments of transportation have worked to understand what asset

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management means for their organization, there are other common themes including (1, 5, 6):
· Asset management is not a black box.
· One size does not fit all.
· Asset management is a concept or framework rather than a thing.

There has also been a realization that these agencies already manage assets, and asset management is a way to use these existing systems to look at their physical assets in a more holistic way in term of the service delivered to customers (7).

In the United States, the American Association of State Highway and Transportation Officials (AASHTO) Task Force on Asset Management, formed in 1997, the American Public Works Association (APWA), although the task force on asset management was disbanded in 1998, and the Federal Highway Administration (FHWA) Office of Asset Management, formed in 1999, provide a focus for asset management activities related to transportation. The AASHTO Task Force on Asset Management (8) developed a strategic plan in 1998 that identifies the mission of the task force: Champion concepts and practices that integrate transportation investment decisions regarding operation, preservation and improvement of transportation systems for member agencies. Specific ongoing activities of the task force include a December 1999 workshop focusing on a state-to-state exchange and the development of an asset management guide through the National Cooperative Highway Research Program (NCHRP). The APWA Task Force on Asset Management was formed to explore the relationship between asset management and the APWA and to investigate the relevance of the concepts to the public works community. The task force disbanded on completion of their report in 1998 (9), which recognized the importance of asset management. Since the completion of the report, the APWA has organized a video conference (10). The FHWA Office of Asset Management was formed to provide leadership, technical expertise, and program assistance. The office is providing assistance to the AASHTO task force, exploring educational initiatives, and providing support for the NCHRP project that will develop the guide for asset management.

Perhaps the most interesting activities related to asset management are activities going on in individual states and agencies. For example, New York has recently produced a concept plan for asset management (11). To facilitate dissemination of these experiences, the focus of the 1999 AASHTO Asset Management Workshop was a peer-to-peer exchange (12).

THE GASB REQUIREMENTS

GASB is a private non-profit organization that determines commonly accepted practices for government financial reporting. Reporting of infrastructure assets has been an option since 1974, but less than 1% of agencies actually report and fewer actually depreciate assets. The intent of the new requirements, known as Statement No. 34, is to make annual financial reports more useful to legislators, investors, and creditors (13) and to support assessment of whether costs are being shifted to future generations, and the relative change in the agency’s financial position. Statement No. 34 requires government financial managers to provide a narrative, known as management’s discussion and analysis (MDA), summarizing the overall financial position and contrasting it with the situation of the previous year. Revised financial statements based on full accrual accounting for all government activities, specifically physical assets, will support the MDA.

Statement No. 34 requires public agencies to report the value of infrastructure assets such as roads, bridges, and tunnels (14). Although the requirements are effective June 1999, a transition period has been defined and the earliest implementation is June 2001. The value may be reported as an historical cost minus depreciation, or using a modified approach. Using the modified approach (13):

“Infrastructure assets are not required to be depreciated if 1) the government manages those assets using an asset management system that has certain characteristics and 2) the government can document that the assets are being preserved approximately at (or above) a condition level established and disclosed by the government. Qualifying governments will make disclosures about infrastructure assets in required supplementary information (RSI), including the physical condition of the assets and the amounts spent to maintain and preserve them over time.”

The asset management system must have an up-to-date inventory, include condition assessments and estimate the annual amount required each year to preserve these assets at some level of performance specified by the reporting agency.

No matter what approach is taken, Madeleine Bloom, the director of FHWA’s Office of Asset Management, summed up the issues in a report to the AASHTO Asset Management Task Force (15): “Adding highway infrastructure to the balance sheets of states will heighten the importance of these assets and draw attention to the need to maintain their condition, which is positive.”

RELATED ACTIVITIES

Which State Is Doing What?

To determine “who is doing what” a survey was sent to each of the fifty states. The survey was aimed at providing input for planning the peer exchange AASHTO workshop on asset management but also captured experiences in the responding states. The results of the survey are documented in (12). Responses should not be interpreted to indicate state practice. For example, several states reported using multiple investment analysis tools, but in reality individual tools are used for specific and limited applications such as pavement design and bridge painting. The survey was divided into three parts with questions addressing what states are doing and how in areas related to inventory, performance, management systems, and investment analysis. Thirty states responded to the survey. Table 1 provides a general summary of the responses. Many states are undertaking activities that form the building blocks for asset management in terms of inventories, condition assessments, performance measures, and management systems.

Surprisingly, ten states (39% of those responding to this question) said they value assets. States were able to check multiple methods. Eight states indicated that they use replacement cost, three indicated use equivalent value, and three indicated use of historical costs. Follow-up telephone calls to several of these states revealed that responding states did not have comprehensive procedures for valuing assets but used the techniques in an exploratory way for a subset of assets. The results provided considerable insight into the diversity of approaches and the different ways in which states are implementing and applying analysis tools:
A variety of tools are used by states in making decisions. Only one state did not use any tools and the majority of states using tools used more than one tool. In fact, three states used four or more methods. The most popular tools were lifecycle cost analysis (used by 88% of states using tools) and cost-benefit analysis (used by 85% of states using tools).

The questions focusing on how these tools were used indicated that a significant number, but not the majority, of states used tools such as benefit-cost analysis across modes to analyze maintenance expenditures, operational improvements, and impacts on system performance.

Respondents indicating the use of feedback mechanisms (~40%) usually cited bridge management systems as the application.

Transport Association of Canada Study

Some of the difficult issues related to asset valuation are being confronted in an ongoing project for the Transportation Association of Canada (TAC), titled “Measuring and Reporting Highway Asset Value, Condition and Performance” (16). The study has explored the applicability of different methods of valuation for different types of infrastructure. The study has also compiled information related to two Canadian experiences. In British Columbia, reorganization requires valuing assets to facilitate transfer from the owner to the operator. Amortized historical cost was used. In Alberta, assets have been capitalized using fixed values and a 50-year amortization with straight-line depreciation.

INTERPRETING THE GASB REQUIREMENTS

Like asset management, valuing assets can be interpreted in many different ways. The value of an asset depends on whether you are interested in the financial or the economic value. There are also many different methods for determining the value of an asset including (17):

- Book value—current value based on historical cost adjusted for depreciation,
- Written down replacement cost—current value based on replacement cost depreciated to current condition,
- Market value—price buyer is willing to pay,
- Equivalent present worth in place—historical cost adjusted for inflation and wear,
- Productivity realized value - net present value of benefit stream for remaining service life.

Statement No. 34 provides an example of asset value based on book value using an estimated historical cost and straight line depreciation as follows (14):

“In 1998, a government has sixty-five lane miles of roads in a secondary road subsystem, and the current construction cost of similar roads is $1 million per lane-mile. The estimated total current replacement cost of the secondary road subsystem of a highway network, therefore, is $65 million. The roads have an estimated weighted average age of fifteen years. Therefore, 1983 is considered to be the acquisition year. Based on US Department of Transportation, Federal Highway Administration’s “Price Trend Information for Federal Aid Highway Construction for 1983 and 1998”, 1983 constructions costs were 69.03 percent of 1998 costs. The estimated historical cost of the subsystem, therefore, is $44,869,500. In 1998, the government would have reported the subsystem in its financial statements to have an estimated cost of $44,869,500 less accumulated depreciation for fifteen years based on that deflated amount. … assume that the road system had a total useful life of twenty-five years. Assuming no residual value at the end of the time, the straight-line depreciation expense would be $1,794,780 per year, and accumulated depreciation in 1998 would be $26,921,700.”

In deciding on a method, the availability of data, and what the results will be used for are critical factors. The value of the asset can be used for establishing accountability, decision making and decision support. It is important to recognize that the value of an asset should also include the question “to whom?” Answering this question requires knowledge of the users of the asset and consideration of time in the sense of whether or not the value of the asset should reflect its value for future generations. For example, an underutilized section of roadway may be in the same condition as a heavily traveled section. To the user they have very different values, but their value based on condition may be the same.

Tennessee’s Experience

Using existing management systems data, Tennessee Department of Transportation has explored the effort required to value right of way, structures, pavement and buildings as required to meet GASB 34 (18). The exploratory analysis was based on the assumption that the modified method will be used with broad classes of infrastructure,
for example, long span bridges being grouped together. It was determined that adequate supporting data already available to be able to meet the reporting requirements including the RSI.

**Using Micro PAVER**

As illustrated by Tennessee DOT’s experience, much of the existing data to support the GASB Statement 34 requirements already reside in existing asset management systems. The Micro PAVER pavement management system (19) provides a simple tool that provides the data to meet the GASB requirements. Specifically, and like other pavement management systems, Micro PAVER includes inventory, condition assessment, and tools for estimating the investment required to meet a specified level of pavement performance. Micro PAVER also illustrates some of the differences between the GASB requirements and asset management. While Micro PAVER meets the GASB requirements, and it is an asset management system for managing a particular type of asset, it is not asset management in the broader sense of the word. It encourages decision-makers to focus on traditional stovepipe decision making and relies heavily on engineering judgement.

**Another Role for the Highway Economic Requirements System (HERS)**

One of the important concepts of asset management is that there is some value to looking at highway assets as a whole rather than in terms of specific types of assets such as pavements and bridges. FHWA is exploring the role the Highway Economic Requirements System (HERS) may play in this (20). HERS is an elaborate benefit costs analysis model used to make recommendations to congress regarding the federal highway budget and considers highway performance in terms of safety, pavement preservation and congestion. The calculation of residual value is particularly interesting but as it currently stands, represents an economic value of a particular segment, rather than a financial value. However, HERS clearly has raw building blocks that are appropriate for developing asset value and for providing supporting information so that agencies do not have to depreciate their assets.

**CONCLUSIONS**

Asset management supports the mission of transportation agencies in the twenty-first century as they deliver customer-oriented service using aging infrastructure with ever more constrained resources. At the same time, GASB Statement 34 provides a motivation for agencies to improve their accountability and disseminate financial information that is meaningful. Meeting the GASB 34 requirements does not necessarily mean that an agency is practicing asset management, nor does practicing asset management mean that the GASB 34 requirements will be met. Data collection, analysis and communication are key elements in making both elements successful.

**ACKNOWLEDGMENTS**

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**REFERENCES**

Quantifying Uncertainty in Bridge Condition Assessment Data

PRITAM DESHMUKH AND KRISTEN L. SANFORD BERNHARDT

The effectiveness of decisions in bridge management systems is based on the quality of data obtained regarding various processes in the systems. Hence, data play a crucial role in bridge management. In general, data collected has some amount of associated uncertainty. In order to assess the impact of this uncertainty on decisions, the uncertainty in the data should be quantified. In other words, by determining the level of uncertainty, we are judging the quality of the data, which is important for making effective decisions in bridge management systems. This paper describes a procedure for measuring the level of uncertainty in bridge condition assessment data. First, a bridge deterioration model was applied to historical data to estimate the current condition of a bridge and compared to current data. Next, reliability theory was applied to estimate the structural reliability of the bridge, again based on both historical and present data. Finally, the reliability of the bridge was compared to the results obtained from the deterioration model, using a coefficient of correlation. Because the deterioration model used Markov chains, which are probabilistic, and the reliability results are also reported as probabilities, the results can be compared.

INTRODUCTION

Bridge management is a rational and systematic approach to organizing and carrying out the activities related to planning, design, construction, maintenance, rehabilitation, and replacement of bridges (1). A bridge management system (BMS) should assist decision-makers in selecting the optimal alternative needed to achieve desired levels of service within the allocated funds and to identify future funding requirements. The most basic requirement for bridge management is a bridge inventory, which includes bridge location, type, functional classification, importance within the network, condition, and maintenance history. Therefore, because the decisions made in a bridge management system are based on information obtained from these data (2), the quality of the data impacts the effectiveness of the decisions. To determine the extent of these impacts, some quantitative methods are required.

This research examines the uncertainty associated with condition assessment data and quantifies it based on mathematical and statistical principles. The authors have developed a procedure using deterioration models and reliability models to compare predicted condition with actual condition. This paper describes the application of the procedure to a small database of three bridges.

BACKGROUND

Bridge Management

State and local agencies use a number of different bridge management systems. Most of these BMS employ probabilistic deterioration models to predict the future condition of a bridge. Pontis, one of the most widely used BMS, uses Markovian models, as do several other BMS. Markovian models are often considered to represent the bridge deterioration process most effectively (see, for example, 3).

Uncertainty

A significant body of research exists on data uncertainty, and much of the work has classified types of uncertainties and proposed models to express the level of uncertainty. However, these techniques have not been widely explored for bridge condition data. Much of the research to date, including the development of guidelines for identifying and reducing specific types of uncertainties, relates to industrial engineering. The accuracy and precision in manufacturing testing and maintenance is higher in industrial settings than in construction or other field-work, and the extensive use of computers and machines in industry reduces the potential for human error, as does the ability to control the environment.

A number of methods for classifying uncertainty have been proposed. For example, Ayyub attributes uncertainties in engineering systems to ambiguity and vagueness in defining the architecture, parameters, and governing prediction models for the systems (4). The ambiguity component is generally ascribed to noncognitive sources, such as physical randomness, statistical uncertainty due to limited information, and model uncertainties due to simplifying assumptions, simplified methods, and idealized representations of real performances. Vagueness typically is due to cognitive sources, such as qualitatively defined variables (e.g., “performance”), human factors, and interrelationships among variables of a problem, especially for a complex system. Similarly, Kikuchi and Parsula also classify uncertainty as cognitive (subjective and not be easily quantified) or noncognitive (typically associated with prediction) (5).

The National Institute of Standards and Technology (NIST) has developed guidelines for evaluating and expressing uncertainty in industrial engineering data. According to Taylor, uncertainty can be divided into two components – random uncertainty and systematic uncertainty (6). Random uncertainties are generally determined by applying reliability theory. The NIST work has addressed standards for and accuracy of data rather than any particular type of data.

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METHODOLOGY

This research addresses noncognitive, random uncertainty in bridge condition data. The methodology combines a comparison of predicted with actual data for both component condition and reliability of a bridge. A correlation coefficient is then used to quantify the level of agreement between the two, which is subsequently used to obtain an overall estimation of “accuracy.”

Condition Assessment

According to Aktan et al., condition assessment is a process, which can be summarized in the following steps (7):
1. Measure the extent of damage/deterioration.
2. Determine the effect of that damage/deterioration on the condition of facility.
3. Set the scale of parameters that describe the condition of the facility as a whole.
4. Compare the existing damage/deterioration with previous records of condition assessment.

Different structural types of bridges, such as reinforced concrete slab, steel stringer, prestressed concrete, and box-reinforced concrete slab, have similar response and loading mechanisms. However, no two bridges are similar in all respects, especially in their deterioration and aging characteristics. Therefore, it is difficult to assess all types of bridges using the same condition analysis framework. This research examines condition assessment data for concrete bridge decks, slabs, girders (or beams), columns, railings, and abutments.

Once the current condition of a bridge has been assessed, future condition can be predicted using deterioration models. Deterioration is a long-term, gradual degradation leading to a reduction in the performance of a member, a structure, and ultimately the entire facility. Considering bridges specifically, deterioration can be defined as a decline in bridge element condition (1). Most deterioration models are based on basic theories of mathematics – statistical regression and/or stochastic modeling.

A Markovian process, which is used in most BMS, is a stochastic process that takes the uncertainties involved in the bridge deterioration process into consideration. Current models do not, however, account for uncertainties in the original data. In a Markov process, the state probabilities (the percentage of the inventory predicted to be in a particular condition state) and the transition probabilities (the probability that the condition of a component will deteriorate from one state to a lower state) are used to predict the future condition of the bridge or bridge component.

The framework proposed here requires the use of deterioration models, and for the test cases presented, the deterioration models in Pontis (8) were used to determine the transition probabilities and to predict future condition. Visual inspection data provide the type and severity of element deterioration, which are recorded as the condition state for that element. Pontis uses a Markovian deterioration model to predict the probability of transition among condition states each year (8).

Reliability Model

Random uncertainty can be mathematically modeled using reliability theory (6). According to Gertsbakh, “The word ‘reliability’ refers to the ability of a system to perform its stated purposes adequately for a specified period of time under the operational conditions encountered” (9). The reliability of a system (in this case, a bridge) is based on the probability of failure of the system. The reliability of the entire system is a combination of the reliabilities of the components that comprise the system.

A bridge is a complex system in itself. In order to calculate the reliability of a bridge or a bridge component, Newton’s law (to every action there is equal and opposite reaction) is used. For all complex structures, two components – resistance and capacity – are used in the calculation of reliability. If the resistance (or “demand”) is greater than the capacity (or “supply”), the system will fail (10). The calculated probability of failure depends on the reasonableness of the underlying assumptions. It is based on empirical models and relies on observational data, as formulated below (10).

If the reliability of a structural system is

\[
\text{Reliability} = 1 - P_f
\]

where \( P_f \) is the probability of failure, then, for discrete variables,

\[
P_f = P(A < B) = \sum P(A < B/B=b)P(B=b)
\]

where \( A \) is the Capacity (Supply), \( B \) is the Resistance (Demand), and \( b \) is the Resistance at a given instance.

Generally, the variables whose functions are discussed above are normal random variables, and their distribution is normal. For calculating the reliability of a particular variable, two moments, the mean and the variance, are estimated. Only two moments of the random variables are considered practical, as large amounts of data are required to evaluate for further moments.

For a complex system, the capacity (supply) and resistance (demand) may each be functions of several other variables. Hence, the problem of calculating the reliability becomes complex, as the selected variable depends on various other random variables. Further, the complexity of the problem increases when the correlation between the variables is considered. To simplify the problem for the given bridges, only the two moments mentioned above are considered.

The total load effect \( S \) is

\[
S = D + L + I
\]

where \( D \) is the dead load, \( L \) is the live load, and \( I \) is the impact load.

All three loads are considered random variables, as the loads at any particular time are not constant. Failure for a particular component will occur when \( S \), the total load, exceeds the strength or resistance, \( R \). Thus,

\[
P_f = P[R < S]
\]

For this model, the mean \( g \) is the difference between resistance and capacity; that is,

\[
g = R - S
\]

And the variance \( \sigma_g \) is

\[
(\sigma_g)^2 = (\sigma_R)^2 - (\sigma_S)^2
\]

where \( \sigma_S \) is the difference between the variance of resistance and capacity, \( \sigma_R \) is the variance of resistance, and \( \sigma_L \) is the variance of load.

The failure probability is the region where \( g < 0 \), and, in discrete form,

\[
P_f = \sum P[g < g_i] \quad \text{for all values where } g < 0
\]

The safety index \( \beta \) is defined as the ratio of the difference between the resistance and capacity (the mean) and the variance:
The Correlation Coefficient \( r \) is calculated as:

\[
\beta = \frac{g}{\sigma_y}
\]  

(8)

Thus, the failure probability is the sum of probabilities over the range where the safety index obtained is negative \((10)\), and the probability of failure can be expressed as a function \( \Phi \) of the ratio of difference between the loads and the difference between variances.

\[
P_f = \Phi(\beta)
\]  

(9)

Substituting,

\[
P_f = \Phi(\gamma)
\]  

(10)

The quantitative relation between the safety index and the probability of failure is shown in Table 1 \((10)\).

### Table 1 Relationship Between the Probability of Failure \( (P_f) \) and the Safety Index \( (\beta) \)

<table>
<thead>
<tr>
<th>( P_f )</th>
<th>0.5</th>
<th>0.25</th>
<th>0.16</th>
<th>0.10</th>
<th>0.05</th>
<th>0.01</th>
<th>( 10^{-3} )</th>
<th>( 10^{-4} )</th>
<th>( 10^{-5} )</th>
<th>( 10^{-6} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \beta )</td>
<td>0</td>
<td>0.67</td>
<td>1.00</td>
<td>1.28</td>
<td>1.65</td>
<td>2.33</td>
<td>3.10</td>
<td>3.72</td>
<td>4.25</td>
<td>4.75</td>
</tr>
</tbody>
</table>

### Coefficient of Correlation

Correlation techniques are used to study relationships (associations) between variables. Correlation is calculated as the level and direction of a relationship between two variables X and Y. The range of values of a correlation coefficient is from “-1” to “+1”. The closer the value is to “+1”, the stronger the positive correlation, and the closer the value is to “-1”, the stronger the negative correlation \((11)\).

The Pearson product moment correlation \( (r) \) is the most common “Correlation Coefficient.” A number of assumptions must be made for the Pearson \( r \) \((11)\):

1. Data for both X and Y must be measured at regular time intervals (e.g. data are collected each year).
2. Both X and Y must be normally distributed.
3. The sample must be representative of the population.
4. The relationship between X and Y should be linear.

It is assumed that the data used in the research satisfies the above requirements.

The Correlation Coefficient \( (r) \) is calculated as:

\[
\frac{\sum (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum (x_i - \bar{x})^2 \sum (y_i - \bar{y})^2}}
\]  

(11)

where \( n \) is the sample size.

To summarize the procedure:

- Estimate the transition probabilities for the components of a bridge based on historical condition assessment data (for \( j \) years).
- Determine the transition probabilities for the same components including condition assessment data from year \( j+1 \).
- Compare the transition probabilities based on the historical data (for \( j \) years) and current data (for \( j+1 \) years) using a coefficient of correlation.
- Similarly, use a reliability model to calculate the current probability of failure based on the condition assessment data at time \( T = j \) years and time \( T = j+1 \) years.
- Compare the probabilities of failure by calculating the coefficient of correlation.
- Calculate a final coefficient of correlation. The uncertainty is quantified in terms of this correlation coefficient.

### CASE STUDY

Three bridges in the state of Missouri form a case study for the methodology described. The condition data for these bridges were obtained from the Missouri Department of Transportation (MoDOT). For this work, concrete bridges were selected based on the age, type, and availability of the required data. The raw data are in the form of condition ratings for bridge components and for the bridge as a whole. These data are used as input to determine the transition probabilities for the deterioration models in Pontis and for calculating the reliability of the components. The transition probability for each component is calculated using deterioration models in Pontis. Table 2 shows the transition probabilities calculated for one of the three bridges.

### Table 2 Transition Probabilities for Bridge H198

<table>
<thead>
<tr>
<th>Element</th>
<th>based on past data</th>
<th>based on present data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>93.28</td>
<td>92.42</td>
</tr>
<tr>
<td>Slab</td>
<td>95.86</td>
<td>93.71</td>
</tr>
<tr>
<td>Girder</td>
<td>94.77</td>
<td>92.18</td>
</tr>
<tr>
<td>Columns</td>
<td>96.82</td>
<td>95.67</td>
</tr>
<tr>
<td>Railings</td>
<td>97.52</td>
<td>97.81</td>
</tr>
<tr>
<td>Abutment</td>
<td>95.47</td>
<td>93.25</td>
</tr>
</tbody>
</table>

Next, the reliability of each component is calculated using past and present data. Tables 3 and 4 show the reliability of components for the same bridge, H198.

The correlation coefficients for the transition probabilities (between past and present data) are calculated for each bridge, as are the coefficients of correlation for the reliabilities. Table 5 shows the coefficients of correlation for each of the three bridges and the final coefficient of correlation for the whole data set.

### CONCLUSIONS

Based on the coefficient of correlation, the uncertainty in condition assessment data can be quantified. The coefficient of correlation varies from 0 to 1, and the closer the value of the coefficient to 1, the higher the correlation between the predicted and present probabilities. These values can be attached to the bridge data, and weights can be assigned to different data elements used in bridge management based on the coefficients of correlation, which would enhance decision making in bridge management.

In this research, the value assigned to uncertainty associated with the data is in the form of a coefficient of correlation. From the coefficients of correlation obtained for these bridges (Table 5), it is clear that the uncertainty is very low. In other words, the results obtained from two different data sets for the same bridge are very close. However, the data used in this example are for only three bridges. If data for a whole network of bridges are used, the procedure will be more efficient and effective, as uncertainty in any problem cannot be eliminated, but only can be reduced. Thus, as the number of iterations increases, the uncertainty in the result decreases.

Strengths of this methodology include the following:

1. Two different models are used for every bridge to calculate data uncertainty. By doing so, the possibility of simply validating the models is decreased.
2. The coefficients of correlation have no units and can be compared to obtain a numerical value for the uncertainty of the condition assessment data.

3. As the amount of data increases, the uncertainty in the procedure decreases (repetitive analysis). Hence, the procedure has great potential for network level bridge management.

While the methodology shows promise, weaknesses which must be addressed in future research include:

1. Additional methods, which are based on the data themselves and collection methods rather than on models, should be explored.

2. Uncertainty is present in the procedure itself, due to the assumptions made in simplifying the models and in calculating the correlation coefficients.

3. The methodology requires large amounts of historical data for bridges in the network to give good results.

ACKNOWLEDGEMENTS

The authors would like to thank the Missouri Department of Trans-
HWYNEEDS: A Sensitivity Analysis

JON LEONARD RESLER AND OMAR SMADI

County highway needs identified in Iowa’s Quadrennial Needs Study are used to determine the amount of funding allocated to each Iowa county for secondary highway improvements. The Iowa Department of Transportation (Iowa DOT) uses a computer algorithm called HWYNEEDS to compute Iowa’s secondary highway needs. Several highway operational, safety, and condition elements are collected and entered into HWYNEEDS. Variations in one of the condition elements, the pavement condition rating, were shown to significantly impact the resulting highway needs. Currently, pavement condition ratings are manually collected every ten years during the winter months. The Iowa Pavement Management Program (IPMP) provided a means to improve the condition ratings with improved data collection procedures. The IPMP uses an automated platform to collect pavement condition data at two-year intervals for the entire state during the year when pavement distresses are easier to recognize. A method was developed to compare the secondary highway needs resulting from the manual and automated collection procedures. First, a means to make the pavement condition data collected for the IPMP compatible with HWYNEEDS was needed. HWYNEEDS requires a pavement condition rating on a scale of one to five with five being a pavement showing no deterioration. As a result, the pavement condition data were converted to pavement condition ratings using equations developed through expert opinion. Second, a historical comparison of the highway needs derived from the manual and automated condition ratings was needed to indicate which data provided more consistent and accurate results. However, the IPMP was in its infancy; therefore, a historical database of condition data did not exist. As a result, pavement performance curves were developed for the automated condition data. Having created a historical database of automated condition ratings, the performance curves contained in HWYNEEDS were used to deteriorate the manual ratings. Pavement performance curves for both the automated and manual condition ratings allowed for more realistic comparisons of the data because the condition of the pavements could be deteriorated to common years. Finally, several data sets compiled from the automated and manual pavement condition ratings were entered into HWYNEEDS to compare the accuracy and consistency of the resulting highway needs.

INTRODUCTION

The majority of revenue allocated to Iowa counties for secondary highway improvements originates in Iowa’s Road Use Tax Fund (RUTF). A legislatively-determined formula designates 32.5 percent of the fund for secondary highway improvements. The amount of funding allocated to individual counties is largely based on the secondary highway needs of each county. Needs are defined as, “physical work necessary to improve, maintain, and administer roads and streets to standards of service essential to serve present and future traffic” (1). Of the 32.5 percent designated for secondary highway improvements, 70 percent of the funds are distributed to each county based on individual county needs relative to the needs of the entire state. The remaining 30 percent is apportioned to each county based on individual county land area relative to the land area of the state.

A method of determining secondary highway needs was necessary if allocations were to be based on needs. As a result, the state legislature required the implementation of the Quadrennial Needs Study as a planning and resource allocation tool (2). Iowa’s highway needs, as determined by the Quadrennial Needs Study, represent the cost of upgrading all roads, structures, and railroad crossings in Iowa to current system design standards plus the cost of maintenance, administration, and engineering for a given 20-year analysis period (3).

Quadrennial Needs Study

The Quadrennial Needs Study consists of a five-step process. The first and second steps deal with functional classification and design guides. Third, inventories of the existing roads, structures, and railroad crossings are taken. This inventory includes such items as lane width, shoulder width, surface type, and traffic. Condition ratings for drainage, pavements, shoulders, and foundations are also included in the inventory (4). The data obtained from the first three steps of the process are incorporated into a computer model called HWYNEEDS.

The fourth step in the process involves the determination of costs. Surveys asking for highway-related cost data are sent to all 99 counties, cities with populations over 5,000, and a sample of smaller cities. The information obtained from the surveys, along with information within the Iowa DOT, is used to develop unit costs (4). Costs for construction, maintenance, and administration are used to develop total dollar needs, which are computed by HWYNEEDS using the cost information developed by the Iowa DOT (4).

The final step in the process is an adequacy appraisal completed by HWYNEEDS. Each highway section, structure, and railroad crossing is compared to design guides to determine existing and accruing deficiencies over a 20-year analysis period (4). To determine accruing deficiencies, traffic levels are forecasted, and condition ratings are depreciated in yearly increments. During each five-year period, the operational, safety, and condition elements of the highway sections, structures, and bridges are analyzed for deficiencies (4). Certain deficiencies or combinations of deficiencies trigger various improvements. Upon completion of the 20-year analysis, the triggered improvements are evaluated for possible redundancies (5). Finally, the second-
ary highway needs are output in terms of construction, maintenance, and administration costs.

**Problem Statement**

The RUTF secondary highway distribution process must be consistent and accurate to ensure the routes comprising the secondary system are maintained and continue to serve the needs of the public. However, reports have shown that HWYNEEDS has produced inconsistent and inaccurate results over time making it difficult for counties to maintain and improve the routes on the secondary system they are responsible for (2,6). Certain counties have experienced shifts in funding between studies in as much as 30 percent (6). Engineers in those counties believe the funding shifts are not justified. Because the secondary highway system remains fairly static, and the time interval between studies is only four years, large funding shifts should not be a common occurrence.

**Objective**

HWYNEEDS uses several inputs relating to the operational, safety, and condition elements of secondary highways to compute the needs for each county. Certain inputs were shown by Iowa Highway Research Board Project HR-363 to significantly impact the results of the model. One of these inputs was the pavement condition rating (6). Currently, the pavement condition ratings used as input to the HWYNEEDS model are collected manually on a ten-year cycle. However, the Iowa Pavement Management Program (IPMP) collects pavement condition data using a service provided by Roadware Corporation. Roadware utilizes a mobile data acquisition platform and an automated process to collect pavement distress information on a two-year cycle. The objective of this paper is to explain how pavement condition data from the IPMP were converted to composite pavement condition ratings used by HWYNEEDS as a means to improve the accuracy and consistency of the resulting highway needs.

**METHODOLOGY**

**IPMP Database**

The IPMP database is the focal point of the pavement management program. Data are received from numerous sources and entered into the database. The Iowa DOT provides cartography and information from its Base Record Inventory System, which is entered into a set of base record tables. The Base Record Inventory System is the Iowa DOT’s statewide highway database consisting of information describing all public roads and structures. The system contains over 150 data fields describing highways; however, only those data fields relating to pavement management are included in the IPMP database (7). Counties, cities, and the Iowa DOT provide pavement section history information for facilities they operate. The information includes such data as the pavement surface type, pavement surface thickness, year of construction, traffic volume, and functional classification. This data is entered into a set of pavement history tables. Roadware provides the distress information for each pavement distress section on all Iowa federal aid eligible, non-national highways, many of which are secondary highways. The distress information is entered into a set of pavement distress tables. The pavement distresses collected by Roadware for both asphalt and concrete pavements followed the definitions given by the SHRP Distress Identification Manual for the Long-Term Pavement Performance Project for distress severity and extent. Roadware collects the following distresses:

- A measure of ride quality referred to as the international roughness index (IRI) for the left and right wheel paths measured in millimeters of roughness per meter of pavement
- Pavement rutting for the left and right wheel paths measured as the rut depth in millimeters
- Durability cracking measured as the number of pavement joints with durability cracking
- Joint spalling measured as the number of spalled joints
- Transverse cracking measured as the length of the transverse cracks in meters
- Patching measured as the area of patching in square meters and number of patches
- Longitudinal cracking measured both in the wheel path and non-wheel path as the length of the longitudinal cracks in meters
- Block cracking measured as the area of block cracking in square meters
- Alligator cracking measured as the area of alligator cracking in square meters
- Pot holes measured as the number of pot holes

Pavement segments used by the Quadrennial Needs Study are defined at three levels. The highways are first divided into needs routes. The needs routes are then divided into needs sections, which in turn, are divided into needs records. The needs sections are based on construction history, and the needs records are simply the base records maintained by the Iowa DOT. It is the intent of the Iowa DOT to determine highway needs at the section level.

Integrating the pavement distress data with needs study pavement data at the section level was accomplished using the base record, pavement history, and pavement distress tables mentioned above. In theory, the base record and pavement history tables should contain the same information. However, the data comes from different sources, so the information is not always the same. Base record information is integrated to the needs sections and used by the Iowa DOT when running HWYNEEDS. Therefore, the consistency of the information between the base record and pavement history data was investigated. When discrepancies were found between the data sets, the pavement history data were used because it arrived directly from the counties that maintained those highways. Finally, using the dynamic segmentation capabilities of the IPMP’s geographic information system (GIS), the needs study section and pavement distress section data were integrated.

**Data Conversion**

The pavement condition rating associated with each needs section is used by the computer model as input to assist in the determination of highway needs. HWYNEEDS does not allow for use of the raw distress data provided by Roadware as input. Therefore, the distress
data must be converted to pavement condition ratings on the same scale used by HWYNEEDS.

Transforming the distress data into pavement condition ratings was accomplished using expert opinion. Members of the Iowa County Engineers Association on the Functional Classification and Highway Needs Committee provided their expert opinions to assign weights to the various distresses. The county engineers agreed that concrete, asphalt, and composite pavements deteriorate differently and should be analyzed separately. Therefore, weights summing to 100 percent were assigned to the distresses of each pavement type. Weights were initially assigned to distress groups and later assigned more specifically to each distress. The distress weights can be seen in column F of Table 1.

Weights were now established for each of the distresses; however, many of the distresses are categorized by distress severity. Factors were applied to the severity levels with the notion that high severity distresses impact pavement quality more than low or moderate severity distresses. Column B of Table 1 shows the severity factors.

Having established distress weights and severity factors, the pavement condition ratings were calculated from the automated condition data. Because ride and rutting values are recorded for each wheel path, average values were calculated for each highway needs section. Next, a single value was obtained for each distress by multiplying the individual severity value by the assigned factor and summing the results. The values are shown in column C of Table 1.

At this point in the condition rating calculation process, the distress values represent the pavement condition of each needs section. However, needs sections vary in length and are comprised of a number of 100-meter distress sections. To provide a common reference, the distress values were divided by the number of distress sections comprising each needs section to obtain an average condition per 100-meter test section. Column D of Table 1 shows the number of distress sections comprising the example needs section. This calculation was not performed on the ride and rutting values because they were already averaged in this way through the dynamic segmentation process.

Distress threshold values, indicating a pavement in poor condition, were determined for the various distresses and are shown in column E of Table 1. Distress values equaling or exceeding the threshold were assigned a value of one and multiplied by the assigned weight since a poor rating is the worst possible pavement rating. In the majority of instances where the distress values did not exceed the threshold, the values were divided by the threshold value and multiplied by the assigned weight. The overall rating per needs section was obtained by subtracting the sum of the weighted distress values from a perfect rating of 100. Finally, to obtain a rating on the one to five scale used by

<table>
<thead>
<tr>
<th>Table 1 Automated Condition Rating Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavement Distress</td>
</tr>
<tr>
<td>IRI</td>
</tr>
<tr>
<td>left wheel path</td>
</tr>
<tr>
<td>right wheel path</td>
</tr>
<tr>
<td>Rutting</td>
</tr>
<tr>
<td>left wheel path</td>
</tr>
<tr>
<td>right wheel path</td>
</tr>
<tr>
<td>Alligator Cracking</td>
</tr>
<tr>
<td>moderate severity</td>
</tr>
<tr>
<td>high severity</td>
</tr>
<tr>
<td>Transverse Cracking</td>
</tr>
<tr>
<td>low severity</td>
</tr>
<tr>
<td>moderate severity</td>
</tr>
<tr>
<td>high severity</td>
</tr>
<tr>
<td>Longitudinal Cracking</td>
</tr>
<tr>
<td>low severity</td>
</tr>
<tr>
<td>moderate severity</td>
</tr>
<tr>
<td>high severity</td>
</tr>
<tr>
<td>Longitudinal Cracking (wheel path)</td>
</tr>
<tr>
<td>low severity</td>
</tr>
<tr>
<td>moderate severity</td>
</tr>
<tr>
<td>high severity</td>
</tr>
<tr>
<td>Block Cracking</td>
</tr>
<tr>
<td>moderate severity</td>
</tr>
<tr>
<td>high severity</td>
</tr>
</tbody>
</table>

Condition Rating = (100-37.64)/20
Condition Rating = 3.1
HWYNEEDS, the overall rating was divided by 20. Where the rating fell below 20, a rating of one was assigned as is done in a similar fashion by HWYNEEDS. This calculation is shown at the bottom of Table 1.

**Pavement Performance Prediction Equations**

Having obtained composite pavement condition ratings, pavement performance prediction equations were derived from the condition ratings and age of the pavements. The performance equations were intended to demonstrate the consistency of the automated pavement condition ratings over time. The equations would also assist in predicting the future condition of the highway needs sections. Finally, the performance equations could be used as a tool to compare the manual and automated condition ratings along with the resulting needs.

Due to the different performance characteristics of asphalt, concrete, and composite pavement, performance equations were developed for each pavement type. Age information was available for 77 asphalt, 64 concrete, and 17 composite pavement needs sections. Pavement age was calculated by subtracting the year of construction or most recent rehabilitation from the last year that the pavement condition was determined. The ages of the needs sections were plotted against the automated condition ratings for each pavement type. Regression was performed to determine the performance trends. Outlying values were eliminated from consideration and the performance trends were reestablished using regression.

Highway needs were calculated using six different pavement condition rating data sets. The first data set consisted of manual pavement condition ratings collected throughout the ten-year period prior to analysis year 1998. The second data set was comprised of the same manual ratings deteriorated to analysis year 1998 using the performance equations. The third data set consisted of automated ratings calculated using the two years of automated distress data collected prior to analysis year 1998. The final three data sets were comprised of the automated condition ratings deteriorated to analysis years 1998, 2002, and 2006 using the performance equations.

**ANALYSIS**

**Pavement Condition Rating Age**

A factor found to influence the amount of resulting highway needs was the age distribution of the pavement condition ratings. To determine the impact of utilizing current condition rating information on the resulting highway needs, a comparison was made of the highway needs calculated using the manual condition ratings collected over ten years and the manual ratings deteriorated to 1998. Deteriorated condition ratings increased total needs by over $13.5 million for analysis year 1998. Increases of more than $500,000 occurred in 12 of 36 corridors in the study. While the overall increase in needs was about 8.0 percent, the needs of several individual counties increased considerably. In two instances, the county highway needs increased by over 60 percent. A comparison of the highway needs calculated using the automated condition ratings collected in the two years prior to 1998 and the highway needs calculated from the same automated ratings deteriorated to 1998 revealed the same trend.

When considering that the results represented the highway needs of the pilot study group, the differences in needs were significant. This was especially true when looking at the results from the manual condition ratings with a wider age distribution. The pilot study consisted of about 675 miles of highway, which is slightly more than 5.0 percent of the total paved portion of the secondary system. Expanding the difference in needs obtained from the pilot study group to a level representing the entire paved portion of the secondary system resulted in a difference of about $260 million over 20 years for analysis year 1998.

**Consistency of Highway Needs**

The consistency of the resulting highway needs was demonstrated using the automated condition ratings deteriorated to analysis years 1998, 2002, and 2006. Having deteriorated the automated condition ratings while holding all other variables constant, the 20-year county highway needs were expected to increase from analysis year 1998 through analysis year 2006. The results confirmed that the total highway needs did increase through the three-year analysis period. The total needs increased by $4 million between the 1998 and 2002 simulated needs studies and by $7 million between the 2002 and 2006 studies. However, continuous increases in highway needs did not always occur at the county level.

The results showed decreasing needs in eight of thirty-six corridors considered. Corridors 6 and 9 experienced significant decreases in needs. Between the needs studies simulated for 2002 and 2006, corridors 6 and 9 experienced decreases in highway needs of $2.7 million and $0.9 million respectively. The results were unusual considering that pavements in worse condition had fewer needs. The resulting highway needs are shown in Figure 1. As a result, a manual analysis was performed on one highway needs section from corridors 6 and 9 to determine how HWYNEEDS was arriving at such unrealistic results.

**CONCLUSIONS**

The timeliness of the condition ratings had a significant impact on the resulting highway needs. Needs calculated from the deteriorated pavement condition ratings were significantly higher than the needs calculated using the condition ratings distributed over a number of years. For example, the needs resulting from the deteriorated manual surface ratings were $13.5 million higher than the needs calculated using the manual ratings collected over a ten-year period. The automated data has the advantage over the manual data in that the automated data is collected more frequently. However, the advantage could be reduced by manually collecting data more frequently or by using performance prediction curves to simulate pavement deterioration.

Improving the computer model would reduce the inconsistencies in the highway needs. Currently, HWYNEEDS may trigger several improvements for a needs section but always selects the improvement triggered first. Multiple improvements are selected only if subsequent improvements are not redundant. As a result, the selected improvement may not be the most cost effective. For example, the condition of certain elements may trigger a resurfacing improvement at year five and a reconstruction improvement at year ten. The
Pilot Study Highway Needs

![Highway Needs Bar Chart](image)

**FIGURE 1** Comparison of highway need resulting from deteriorated automated surface ratings

reconstruction improvement would improve the surface among other problems but will not be selected because it was not triggered first. A method is needed that will prioritize all triggered improvements.

**ACKNOWLEDGEMENT**

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**REFERENCES**

Reclaimed Fly Ash As Select Fill Under PCC Pavement

KENNETH L. BERGESON AND DAN MAHRT

With the support of the Iowa Fly Ash Affiliates, research on reclaimed fly ash for use as a construction material has been ongoing since 1991. The material exhibits engineering properties similar to those of soft limestone or sandstone and a lightweight aggregate. It is unique in that it is rich in calcium, silica, and aluminum and exhibits pozzolanic properties (i.e. gains strength over time) when used untreated or when a calcium activator is added. Reclaimed Class C fly ashes have been successfully used as a base material on a variety of construction projects in southern and western Iowa. Many of the soil types encountered for highway projects are unsuitable soils under the current Iowa DOT specifications. The bulk of the remaining soils are Class 10 soils. Select soils for use directly under the pavement are often difficult to find on a project, and in many instances are economically unavailable. This was the case for a 4.43-mile grading (STP-S-90(22)-SE-90) and paving project in Wapello County. They supported the use of reclaimed fly ash for a portion of the project. Construction of about three miles of the project was accomplished using ten inches of reclaimed fly ash as a select fill beneath the PCC slab. The remaining mile was constructed according to the original design to be used as a control section for performance monitoring. The project was graded during the summers of 1998 and 1999. Paving was completed in the fall of 1999. This paper presents the results of laboratory and field testing during construction.

INTRODUCTION

Reclaimed hydrated fly ashes are produced at sluice pond disposal sites at generating stations burning sub-bituminous coals (1). Raw Class C fly ash is collected from the electrostatic precipitators at the power plant. If the supply of the raw fly ash exceeds demand, the excess raw fly ash is transported to the sluice pond or other disposal site. At a sluice pond site, the raw fly ash is dozed into the sluice pond where it hydrates to form a cementitious, solid mass to create a working platform where additional raw fly ash is spread, water is added, and the product is compacted. Once the ash has hydrated, it is reclaimed using conventional recycling-reclaiming equipment to pulverize the material. The reclaimed fly ash is then stockpiled on site, ready for use as a construction material.

This project begins at the Alliant Utilities generating station in Chillicothe, Iowa and runs west to the Monroe-Wapello county line. The road will carry a significant amount of semi-tractor trailer traffic hauling coal from the generating station to a Cargill corn processing plant in Eddyville, Iowa. Select subgrade soils are not available on site, thus the pavement was to be constructed directly on a Class 10 subgrade. Approximately 3.1 miles out of the 4.43-mile project was constructed with 10 inches of reclaimed fly ash select fill beneath 9-1/2 inches of PCC pavement. The remainder of the project was constructed using typical construction practices, utilizing the Class 10 soils on site, and serves as a control section for performance evaluation.

The reclaimed fly ash was constructed 12 inches thick and full width (49 feet) during the grading process. After compaction of the reclaimed fly ash fill, a two to three-inch thick temporary surfacing of crushed limestone was placed. Prior to paving, approximately two inches of the reclaimed fly ash fill was trimmed to be used for shouldering material, leaving approximately 10 inches of select fill to support the pavement. Pavement thickness designs conducted by the Iowa Concrete Paving Association resulted in an allowable thickness reduction from ten to nine inches using reclaimed fly ash select fill. The Wapello County engineer elected to use a 9-1/2 inch slab as a conservative approach.

The reclaimed fly ash fill was constructed in one twelve-inch thick lift, using a sheepfoot roller for initial compaction. A steel or pneumatic wheel roller was used for final compaction to create a smooth surface. The reclaimed fly ash fill was specified to be compacted at ± 2% of the Standard Proctor optimum moisture content to 90% of Standard Proctor density for the bottom six inches, and 95% of Standard Proctor density for the top six inches.

CONSTRUCTION TESTING PROGRAM

Standard Proctor Testing

One-point standard Proctor testing was conducted daily to monitor variations in the reclaimed fly ash as the reclaiming depth increased. The testing was run in accordance with ASTM D698 – Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort, except the compaction samples were run at the moisture content at which they were collected, and only one compactive trial was made. Because a full suite of standard Proctor testing was completed prior to the actual construction, ranges of optimum moisture content and maximum dry unit weight were already known. Knowing the general compaction characteristics, the one-point Proctor tests were used to monitor daily variation of moisture content and dry unit weight. A summary of the results is presented in Table 1.

Moisture Content Testing

Moisture content determinations were made at least once daily during construction of the test road to ensure that the moisture content was within the specified range. During construction in the fall of 1998, moisture control was not a large problem. The fly ash that was
reclaimed was near the optimum moisture content and no water had to be added. During construction in the summer of 1999, however, the in situ moisture content of the hydrated fly ash was well below the optimum moisture content, and water had to be added to the material to increase the moisture content to near optimum. The in situ moisture content of the hydrated fly ash during the summer of 1999 remained around 18% to 19%. With an optimum moisture content near 24%, a large volume of water needed to be added to the reclaimed fly ash to increase the moisture content into the specified range. The average moisture contents of the reclaimed fly ash as placed are presented in Table 2.

<table>
<thead>
<tr>
<th>Construction Period</th>
<th>Number of Tests</th>
<th>Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fall 1998</td>
<td>19</td>
<td>23.8</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>1.8</td>
<td>1.8</td>
</tr>
<tr>
<td>Summer 1999</td>
<td>22</td>
<td>24.0</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>2.5</td>
<td>2.9</td>
</tr>
<tr>
<td>Overall Average</td>
<td>41</td>
<td>23.9</td>
</tr>
<tr>
<td>Overall Standard Deviation</td>
<td>2.2</td>
<td>2.5</td>
</tr>
</tbody>
</table>

### Particle Size Analyses

Wet sieve analysis tests were conducted daily during the construction periods to monitor changes in the gradation of the reclaimed fly ash. A summary of the results of particle size analyses completed during construction is given in Table 3.

<table>
<thead>
<tr>
<th>Construction Period</th>
<th>Number of Tests</th>
<th>Percent Passing Sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fall 1998</td>
<td>1</td>
<td>90.7</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>4.3</td>
<td>4.9</td>
</tr>
<tr>
<td>Summer 1999</td>
<td>21</td>
<td>90.3</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Overall Average</td>
<td>32</td>
<td>90.3</td>
</tr>
<tr>
<td>Overall Standard Deviation</td>
<td>4.6</td>
<td>4.9</td>
</tr>
</tbody>
</table>

### Rubber Balloon Compaction Testing

Density tests were completed on the reclaimed ash test sections shortly after completion of sheepsfoot rolling in accordance with ASTM D2167. All results are presented based on a Standard Proctor maximum dry unit weight of 98 pounds per cubic foot, which is the highest dry unit weight obtained from all compaction tests. The selection of 98 pounds per cubic foot as the maximum dry unit weight is a conservative approach. The maximum dry unit weight of the reclaimed ash was seen to vary, but there is no trend in the variation, therefore a single value of maximum dry unit weight was selected to compute compaction at each test location. A summary of the compaction test results is presented in Table 4. Overall, good compaction was achieved, with an average of 95.9% of Standard Proctor compaction achieved for the top six inches and 90.4% of Standard Proctor achieved for the bottom six inches.

### Nuclear Densometer Compaction Testing

A nuclear densometer was also used to monitor compaction during construction of the test road. Density testing was conducted in accordance with ASTM D2922, and moisture testing was done in accordance with ASTM D3017. The wet density results were generally slightly higher than the dry density determined using the rubber balloon method. The moisture content determined by the nuclear gauge was always much lower than the values obtained from moisture content determinations for the rubber balloon testing. The wet density value obtained from the nuclear densometer is believed to be slightly high because of high amounts of calcium in the material. Calcium absorbs more radiation than typical soil elements, which results in a wet density reading that is higher than the actual wet density. The density readings are only slightly higher than the actual density, and can be corrected without a large loss of precision. The variation in moisture content readings was random, with no clear trends. The mechanisms controlling this phenomena are uncertain and are still under investigation. Nuclear density testing should not be used for these materials.
Dynamic Cone Penetrometer (DCP) Testing

Dynamic cone penetration (DCP) tests were conducted on freshly placed reclaimed fly ash to evaluate the short-term strength of the material. The dynamic cone penetrometer consists of a 20 mm diameter, 60° cone mounted on a steel rod. A sliding mass of 17.6 pounds is dropped 22.6 inches to drive the cone into the test material. The number of hammer drops is recorded with respect to the depth of penetration of the cone. The numerical result of the DCP test is the DCP index, which is measured in millimeters of penetration per hammer drop. The DCP index has been correlated with California Bearing Ratio (CBR), and the DCP results presented herein are given in terms of the correlated CBR. DCP testing was completed on the reclaimed ash fill at selected time periods after initial compaction to monitor strength gain of the reclaimed ash as a function of time. The reclaimed fly ash that was placed in the fall of 1998 was re-tested in the spring of 1999, approximately seven months after placement, and was tested again in the late summer of 1999, or approximately nine months after placement. The reclaimed ash that was placed during the summer of 1999 was tested prior to paving operations in the fall of 1999, about three to four months after placement. A summary of the DCP results on the reclaimed ash fill is presented in Table 5.

Strength gain of the reclaimed fly ash fill over time is shown on Figures 1 and 2. Figure 1 presents the strength gain data for material placed in October of 1998, and Figure 2 presents data for material placed in July of 1999. Both Figures 1 and 2 present the increase in the average CBR over time. Error bars are given for each data point and represent plus and minus one standard deviation from the mean and the high and low values obtained from each test set. An average strength gain of approximately 70% per month is seen in Figure 1, and an average gain of 60% per month is seen in Figure 2. A dormant period is depicted in Figure 1 that extends from the time of placement until approximately April of 1999. This dormant period occurs because the ambient temperatures are too low for strength gain to take place in the fly ash. Fly ash needs available water and heat to gain strength. When temperatures are below freezing, pozzolanic reactions and strength gain stop, thus only minimal strength gain is expected between approximately October and late March in the Midwest because temperatures are frequently below freezing in this time period.

Dynamic cone penetration testing was also completed on the control section of the test project at different times of the year to determine the seasonal variation of CBR. The average DCP results for each test period are presented in Table 6. It is seen that the overall average CBR of the subgrade soils is 8.0%, with seasonal variation taking the CBR at the top six inches down to 4.2%. Many of the CBR values obtained are less than 6%, which is generally regarded as the minimum CBR to support construction equipment without rutting and shear failure of the subgrade soils (2).

![FIGURE 1](image1.png)

**FIGURE 1** Strength gain of reclaimed fly ash fill placed in October 1998

![FIGURE 2](image2.png)

**FIGURE 2** Strength gain of reclaimed fly ash fill placed in July of 1999

**TABLE 5** DCP Test Results on Reclaimed Fly Ash Fill

<table>
<thead>
<tr>
<th>Age Construction Period (months) of Tests</th>
<th>Number/Ash Fill</th>
<th>0-6&quot;</th>
<th>6-12&quot;</th>
<th>0-6&quot;</th>
<th>6-12&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fall 1998</td>
<td>0</td>
<td>23</td>
<td>23.8</td>
<td>18.8</td>
<td>9.7</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>8.70</td>
<td>8.00</td>
<td>3.30</td>
<td>3.40</td>
<td></td>
</tr>
<tr>
<td>Fall 1998</td>
<td>7</td>
<td>28</td>
<td>57.1</td>
<td>34.3</td>
<td>20.5</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>16.2</td>
<td>18.2</td>
<td>17.5</td>
<td>3.4</td>
<td></td>
</tr>
<tr>
<td>Fall 1998</td>
<td>9</td>
<td>12</td>
<td>92.3</td>
<td>68.9</td>
<td>16.5</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>50.3</td>
<td>43.0</td>
<td>12.2</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td>Summer 1999</td>
<td>0</td>
<td>79</td>
<td>34.0</td>
<td>30.5</td>
<td>21.9</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>19.9</td>
<td>17.3</td>
<td>12.8</td>
<td>8.2</td>
<td></td>
</tr>
<tr>
<td>Summer 1999</td>
<td>3.5</td>
<td>26</td>
<td>101.3</td>
<td>73.8</td>
<td>34.5</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>71.8</td>
<td>53.8</td>
<td>48.4</td>
<td>9.4</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 6** DCP Test Results on Control Section

<table>
<thead>
<tr>
<th>Construction Period</th>
<th>Number of Tests 0-6&quot;</th>
<th>6-12&quot;</th>
<th>12-18&quot;</th>
<th>18-24&quot;</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Late Fall 1998</td>
<td>22</td>
<td>5.0</td>
<td>7.2</td>
<td>9.5</td>
<td>9.7</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>2.0</td>
<td>2.9</td>
<td>4.0</td>
<td>3.7</td>
<td>2.7</td>
</tr>
<tr>
<td>Late Spring 1999</td>
<td>22</td>
<td>4.2</td>
<td>4.5</td>
<td>6.2</td>
<td>8.0</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>1.7</td>
<td>3.0</td>
<td>3.3</td>
<td>4.4</td>
<td>2.3</td>
</tr>
<tr>
<td>Late Summer 1999</td>
<td>8</td>
<td>19.8</td>
<td>16.1</td>
<td>13.0</td>
<td>9.3</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>5.5</td>
<td>5.6</td>
<td>5.9</td>
<td>4.7</td>
<td>4.1</td>
</tr>
<tr>
<td>Overall Average</td>
<td>52</td>
<td>6.9</td>
<td>7.4</td>
<td>8.6</td>
<td>8.9</td>
</tr>
<tr>
<td>Overall Standard Deviation</td>
<td>2.4</td>
<td>3.3</td>
<td>4.0</td>
<td>4.2</td>
<td>2.7</td>
</tr>
</tbody>
</table>
CONSTRUCTION

Fall 1998 Construction

Construction of the test road using reclaimed fly ash fill began on October 16, 1998. A total of 11 working days were used to construct a one-mile portion of the test road from station 0+00 to station 56+00. A total of 16,510 tons of reclaimed fly ash were placed, slightly higher than the 16,000 ton estimate. The peak production for this period was 7.6 stations, or 2,240 tons, which was placed on October 29. An average of 5.1 stations per day, or 1,500 tons was constructed per day for this construction period.

Summer 1999 Construction

Iowa State researchers met with representatives from the Wapello County Engineer’s Office, ISG Resources (the select fill and raw fly ash supplier) and the earthwork contractor on June 24, 1999 to devise a plan for the final stages of select fill placement. The main goal of the meeting was to determine a course of action to follow if problems with extremely soft subgrade and soft sections of select fill were encountered. Iowa State researchers ran DCP tests on the several areas of the subgrade, and suggested that five areas in particular, as shown in Table 7, be stabilized with raw class C fly ash before placing any select fill. The Wapello County Engineer’s office elected to stabilize three of these areas, as shown in Table 7. It was further decided that construction would begin at the east end of the project, proceeding westward, running the loaded haul units over the select fill to achieve further compaction.

![Table 7: Summer 1999 Subgrade Instability Areas](image)

TABLE 7 Summer 1999 Subgrade Instability Areas

<table>
<thead>
<tr>
<th>Subgrade Instability Areas</th>
<th>Fly Ash Stabilization</th>
</tr>
</thead>
<tbody>
<tr>
<td>Station to Station</td>
<td>Station to Station</td>
</tr>
<tr>
<td>138+00 to 141+00</td>
<td>139+00 to 140+00</td>
</tr>
<tr>
<td>141+00 to 144+00</td>
<td>141+00 to 144+00</td>
</tr>
<tr>
<td>174+00 to 178+00</td>
<td>Not Stabilized</td>
</tr>
<tr>
<td>184+00 to 188+00</td>
<td>Not Stabilized</td>
</tr>
<tr>
<td>205+00 to 208+00</td>
<td>205+00 to 208+00</td>
</tr>
</tbody>
</table>

Construction of the final two miles select fill began on July 13, 1999 at station 231+00. The work progressed westward to station 96+00, and finally commenced by completing stations 231+00 to 236+00. A total of 140 stations were constructed in 22 working days, for an average of 6.4 stations constructed per day. A total of 42,894 tons of reclaimed fly ash fill was placed during this time period, for an average of 306 tons per station. The select fill placement was completed on August 19, 1999.

Fall 1999 Paving

Paving operations began for the road in late September of 1999 and were completed in October of 1999. No problems were encountered that were directly related to the select fill material. Some areas of instability did develop in the select fill under traffic from loaded concrete trucks, and these areas were moistened and re-compacted with a vibratory steel-wheel roller prior to paving. Most of the unstable areas that were present at this time occurred at earlier stages of construction likely due to soft subgrade conditions, but had since “healed”, only to reappear under the heavy concrete trucks.

CONCLUSION

From the testing results and research done on this project, it appears that reclaimed fly ash is a suitable material to be used as a select fill on certain projects. The reclaimed fly ash fill is inexpensive compared to typical pavement base materials and unique in that it will gain strength over time.

From a construction standpoint, there are a few precautions that must be taken and a few general guidelines to follow that are somewhat different than those typically encountered. Moisture control of the reclaimed fly ash is one of the most important facets of construction with this material. As with any soil, when the moisture content is not in the optimum range specified compaction is typically not achieved and strength is decreased. When working with reclaimed fly ash in the fall of the year, it is also important to realize that the material will not likely gain strength until the next year. This is an important fact to consider when a design value of CBR or modulus of subgrade reaction is used that is dependent on some strength gain of the material. For compaction, a heavy sheepfoot roller, preferably vibratory, should be used for initial compaction. Final compaction should be achieved using a smooth-wheel roller such as a steel-drum roller or a pneumatic roller. The smooth wheels on these rollers smooth the surface of the reclaimed fly ash so that water will run off and not penetrate the material. Temporary surfacing material should be placed shortly after finish rolling the reclaimed fly ash pad. It should also be noted that although the reclaimed fly ash gains strength over time, it is not able to bridge extremely soft soils. If soft soils are encountered on a site, they should first be stabilized or replaced before placing reclaimed ash on top of them.

ACKNOWLEDGEMENTS

The authors would like to thank Gary Greene of ISG Resources for his help in coordinating meetings and other activities that were necessary to aid in construction of the project.

We would also like to thank the Iowa Fly Ash Affiliate Research Program, the Project Development Division of the Iowa DOT and the Iowa Highway Research Board (TR-425) for sponsoring the project, as well as the Wapello County Engineer’s Office for their support.

REFERENCES

The deleterious effects of deicers on concrete pavements and bridges have concerned concrete researchers for several decades. The present study experimentally investigates the effects of different deicers on concrete deterioration. Laboratory simulations of environmental conditions (wet/dry and freeze/thaw cycling) were conducted on highway concrete samples with various deicer chemicals (NaCl, CaCl₂, MgCl₂, calcium magnesium acetate (CMA) of 5 different Ca/Mg ratios, Ca-acetate, and Mg-acetate). Each deicer produced characteristic effects on the concrete samples by physically and chemically altering the dolomite coarse aggregate, the dolomite coarse aggregate-paste interface, and cement paste. Chloride solutions commonly promoted decalcification of paste and altered ettringite to chloroaluminate. Magnesium-bearing deicer solutions (e.g., CMA, Mg-acetate and MgCl₂) caused severe paste deterioration by forming brucite and non-cementitious magnesium silicate hydrate. For acetate solutions, the effects caused by Ca-acetate on concrete deterioration was much less severe than those caused by Mg-bearing acetates. For the experimental conditions utilized herein, NaCl solution was the least deleterious to the cement paste and aggregate. Key words: concrete, aggregate, deterioration, deicers, secondary minerals.

INTRODUCTION

Deterioration of concrete by deicers is related to complex processes associated with physical and chemical alteration in cement paste and aggregates. It is affected by factors such as the cation composition of the deicer, aggregate type, and aggregate reactivity. The present investigation evaluates these factors by studying the effects of different deicers on Iowa highway concrete. Cody et al. (1) determined the effects of NaCl, CaCl₂, and MgCl₂ solutions on the deterioration of concrete from Iowa highways and on Mg migration from dolomite coarse aggregate. In the present investigation, a more detailed study was conducted on the effects of NaCl, CaCl₂, MgCl₂, CMA, Ca-acetate, and Mg-acetate on the deterioration of concrete during freeze/thaw (F/T) and wet/dry (W/D) conditions. Special attention was paid to the secondary minerals that formed and the mineral changes that occurred as a direct result of deicers. Because acetates are less detrimental than chlorides to the environment and to steel reinforcement in concrete, the effects of three acetates (calcium acetate, magnesium acetate, and calcium magnesium acetate) were examined to determine if one was less detrimental to concrete than the others. In addition, the effects of different ratios of Ca-acetate to Mg-acetate were examined to determine the relative aggressiveness of Ca and Mg ions.

EXPERIMENTAL METHODS

Cores were taken from seven existing Iowa highway concretes of different service records, and small 3cm x 1.5cm x 1.5cm blocks were cut from the cores. Two blocks from each core were immersed in 100 ml of solution and sealed in cleaned polymethylpentene containers that were stored for 132 hours at 58°C in a constant temperature chamber. The solutions used were 0.75 M CaCl₂·2H₂O, MgCl₂·6H₂O, NaCl, calcium acetate Ca(CH₃COO)₂·H₂O, magnesium acetate Mg(CH₃COO)₂·4H₂O, and CMA based on a molar ratio of 3:7, i.e., 3[Ca(CH₃COO)₂·H₂O]·7[Mg(CH₃COO)₂·4H₂O], and distilled water. Experiments were also conducted with five solutions of 0.75 M CMA with different molar ratios of Ca-acetate and Mg-acetate (5:3, 7:3, 1:1, 3:5, and 7:3). All solutions contained 0.01% sodium azide to control bacterial growth.

Wet/Dry (W/D) experiments: After being immersed in solutions at 58°C for 132 hours, blocks were removed from the solutions, dried at 58°C (=135°F) for 24 hours, air cooled to 25°C, returned to their immersion solutions at 25°C, and again stored at 58°C for 132 hours.

Freeze/Thaw (F/T) experiments: Samples removed from the 58°C solutions after 132 hours were air cooled to 25°C and stored for 24 hours in a freezer at -4°C (25°F). The blocks were air warmed to 25°C, returned to their respective solutions at 25°C, and stored at 58°C for 132 hours.

All experiments were conducted with both transmitted and reflected light in order to identify specific areas to be studied by scanning electron microscopy (SEM) and to supplement observations of features difficult to observe with an SEM such as color changes on coarse aggregate margins. A Hitachi S 2460 reduced-vacuum scanning electron microscope was used. Back-scattered images were taken and energy dispersive analytical x-ray (EDAX) area mapping was performed for Si, Al, K, Na, O, Ca, Mg, S, Cl, and Fe. EDAX point analyses were obtained at high magnification for mineral identification. An accelerating voltage of 15 kV was generally used for imaging, whereas EDAX point analyses were obtained at 20 kV.

RESULTS AND DISCUSSION

Each deicer solution had characteristic effects on the concrete blocks under freeze/thaw and wet/dry conditions. CMA caused the most aggressive degradation of concrete in both W/D and F/T experiments. All CMA-treated samples showed deterioration after 15 cycles.
Magnesium chloride wet/dry experiments were ended after 23 cycles, indicating rapid deterioration, but freeze/thaw cycling was less deleterious. Longevity of the F/T treated blocks is attributed to precipitation of a protective coat of new mineral matter on the concrete surfaces. Beneath the white coating, the paste was brown-colored and crumbled, with random fractures. Calcium chloride wet/dry experiments were typically terminated between 33 and 49 cycles, indicating relatively minor deterioration, and freeze/thaw experiments were even less damaging and a shiny, blue-gray protective mineral coat formed on the concrete surfaces. Sodium chloride and distilled water were least destructive, giving similar durability in W/D experiments, but NaCl was slightly more deleterious in F/T experiments. NaCl-treated F/T blocks showed surface roughening (dissolution), edge crumbling, and a thin gray-white surface coating.

Effects of Chloride Solutions

Calcium Chloride

Calcium chloride deicing salts especially affected those concretes containing reactive dolomite aggregates by enhancing dedolomitization reactions that release magnesium to form brucite and magnesium silicate hydrate (MSH). Alteration rims observed after wet/dry and freeze/thaw conditions appear similar to those seen in untreated concrete, which had resulted from reactions between the reactive dolomite aggregate and paste during highway use. Major changes occurred in an outer light-colored dolomite aggregate alteration zone (FIGURE 1). The EDAX element maps show a decrease in Ca and
a concomitant increase in Mg in this region as a result of calcium chloride treatment. In these alteration rims, considerable volumes of calcite existed without significant quantities of brucite, but CaCl₂ produced abundant brucite and reduced the amount of calcite. An inner dark dolomite alteration zone and a light-colored cement alteration zone, both of which had developed during highway use, remained essentially the same as they were in untreated samples. In durable concrete with non-reactive dolomite aggregate, no alteration zone effects occurred during wet/dry or freeze/thaw cycling.

Calcium chloride affected the cement paste of both durable and non-durable concrete. The paste was markedly discolored. EDAX element maps show that Cl is concentrated in these areas (FIGURE 1). Chloride concentration may be due to the formation of calcium chloride hydrate (3CaO·CaCl₂·12 H₂O; CaO·CaCl₂·2 H₂O) or to adsorption of Cl⁻ by calcium silicate hydrate (CHS). Discoloration may be the result of iron released from calcium alumino ferrite hydrate (2).

**Magnesium Chloride**

Magnesium chloride produced distinctive alteration rims at the margins of reactive dolomite aggregate. Much brucite formed in an outer light-colored dolomite rim, a feature not seen in the corresponding alteration rims of untreated material. New brucite occurred in concrete subjected to wet/dry and freeze/thaw conditions and is morphologically very similar to brucite observed in CaCl₂-treated concrete.
Magnesium chloride produced significant concrete crumbling because of widespread replacement of Mg by non-cementitious MSH resulting from reactions between Mg$^{2+}$ and the cement phase (FIGURE 2). Abundant shrinkage cracks developed in the MSH. Using a modification of Bonen’s (3) equation, the general reaction for the formation of MSH from CSH in Mg-chloride solution can be written as follows:

$$x \text{CaO} \cdot \text{SiO}_2 \cdot \text{i} \text{H}_2 \text{O} + x \text{MgCl}_2 + m \text{H}_2 \text{O} = y \text{MgO} \cdot \text{SiO}_2 \cdot 2\text{H}_2 \text{O} + (x - y) \text{Mg(OH)}_2 + x \text{CaCl}_2 \cdot 2\text{H}_2 \text{O}$$

where $i + m = n + 3x - y$

This reaction suggests that displaced Ca$^{2+}$ leaches out into cement paste because CaCl$_2$ is highly soluble and forms portlandite, Ca(OH)$_2$, or calcite/aragonite, CaCO$_3$. Calcite associated with brucite in air voids or in cracks in some samples supports this conclusion. Needles of calcium carbonate and brucite precipitated on the surface of a MgCl$_2$-treated sample.

**Sodium Chloride**

Sodium chloride produced no enhanced or new alteration rim zones in either dolomite aggregate or cement paste in non-durable concrete. No reaction rims were observed in durable concrete. Brucite in the cement paste appears to be stable in NaCl-treated concrete.

All chloride solutions produced chloroaluminate. Its morphology and occurrence in air-entrainment voids suggest that it is tri-chloroaluminate, 3CaO·Al$_2$O$_3$·3CaCl$_2$·32H$_2$O, resulting from the replacement of pre-existing ettringite (FIGURE 1) in which Cl substitutes for SO$_4^{2-}$ ions in the ettringite structure:

$$3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaSO}_4 \cdot 32\text{H}_2\text{O} + 3\text{CaCl}_2 = 3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaCl}_2 \cdot 32\text{H}_2\text{O} + 3\text{CaSO}_4$$

Traces of gypsum detected in a surface coating of the sample block may have resulted during the above reaction. All chloride solutions also appear to cause paste deterioration by decalcification. The release of Ca$^{2+}$ from the paste is documented by the precipitation of calcite crystals in the solutions and by a surface calcite coating on blocks treated with chloride solutions. It is known that chlorides promote the leaching of Ca(OH)$_2$ and promote the formation of porous CSH, but the reactions involved are complex (4). Pre-existing brucite crystals in the cement paste are stable in chloride-treated concrete.

**Effects of Acetate Solutions**

Calcium magnesium acetate solutions were the most damaging of all solutions tested. Both wet/dry and freeze/thaw cycling in CMA produced widespread and severe damage. Alteration rims that formed at the interface between reactive dolomite aggregate and cement paste in CMA-treated concrete were almost identical to those found in MgCl$_2$-treated concrete. Both durable and non-durable concrete showed severe paste deterioration. The main causes of cement paste deterioration by CMA solutions probably are non-cementitious MSH resulting from the replacement of CSH and the formation of potentially expansive brucite. Brucite commonly formed in air voids and at paste-fine aggregate interfaces (FIGURES 3 and 4). Abundant shrinkage cracks developed in the MSH, especially under wet/dry conditions. Calcium ions displaced by the MSH-forming reaction precipitated as thin layers of calcite on the walls of open spaces such as air voids and between the aggregate and cement paste where brucite also precipitated (FIGURE 3).

Another aggressive feature of CMA treatment was debonding of fine aggregate particles. As shown in FIGURE 3, CMA solutions penetrate the cement paste, especially along the boundaries between the fine aggregate and the cement paste where they react with the CHS at the paste/fine aggregate interface and form a thin layer of non-cementitious MSH. This MSH layer resulted in debonding of fine aggregate from the paste. Secondary brucite was then precipitated in the large voids created by loss of fine aggregate. As a result of the newly created voids, a series of new minerals were observed frequently in those open spaces.

Coarse dolomite aggregate was generally not subject to debonding. In regions of the paste in which calcite had precipitated during highway use, the CSH does not appear to change to MSH, possibly because calcite precipitation during highway use conditions reduced paste permeability to CMA solutions. As in MgCl$_2$-treated concretes, major amounts of new brucite formed in air entrainment voids where ettringite only was seen previously, and the ettringite in the voids was reduced in abundance (FIGURES 2 and 4). According to Taylor (5), ettringite may be decomposed by magnesium ions because brucite formation consumes OH$^-$ ions in pore solution that lowers pH $< 10.5$; such conditions are outside the ettringite stability region. With CMA treatment, the destruction of pre-existing ettringite appears to be even more complete than that resulting from magnesium chloride solutions. Calcium acetate treated samples that contained abundant ettringite before treatment showed no loss of ettringite, thus confirming that decomposition of ettringite occurs due to the pH decrease caused by the formation of brucite rather than dissolution by acetate solutions. The reason for the greater deterioration by CMA compared to magnesium chloride is not clear.

Experiments with CMA solutions containing different molar ratios of Ca acetate to Mg acetate (5:3, 7:3, 1:1, 3:5, and 7:3) emphasized the role that the cation Mg$^{2+}$ has in concrete deterioration by CMA solutions. In these experiments, only concrete containing non-reactive Sundheim quarry dolomite coarse aggregate was used in order to reduce reactive aggregate effects.

**Magnesium Acetate**

Paste deterioration by Mg-acetate solution was similar to that occurring with CMA in which MSH and brucite formed at the interface between fine aggregate and paste and caused debonding of the two components. Brucite with thin calcite layers formed in air entrainment voids under both wet/dry and freeze/thaw condition. Non-cementitious MSH formed in the cement paste.

**Calcium Acetate**

Ca acetate solutions produced much less paste deterioration than CMA and Mg acetate. No new minerals formed in air entrainment voids, and the fine aggregates were present in their pre-treatment condition. The only observed change in the cement paste was that calcium increased in the paste adjacent to coarse dolomite aggregate during freeze/thaw experiments. Calcium increase was probably due to calcite precipitation.
Different Ca/Mg Ratios in CMA

These experiments showed that the Mg in CMA clearly is responsible for concrete damage. The rate of deterioration depended on the percentage of magnesium in the mixtures. Ca acetate produced little deterioration. For acetate solutions containing both Ca and Mg, the Ca/Mg ratio of 7:3 had the least effect on the concrete. In these solutions, the concrete blocks developed a brown coloration and showed slight paste deterioration with minor surface roughness and edge crumbling of the blocks. A Ca/Mg ratio of $\leq 3:5$ was very detrimental to concrete. The concrete blocks underwent major degradation by edge and surface crumbling, and loss of paste and fine aggregate. Freeze/thaw degradation was less pronounced than that in wet/dry conditions but still significant.

According to Dunn and Schenk (6), a higher ratio of magnesium to calcium should be beneficial for deicing use because of higher solubility and lower freezing point depression of magnesium acetate. However, the present study suggests that higher Mg:Ca ratios may be more destructive and may produce premature concrete deterioration.

CONCLUSIONS

In our experiments, concrete samples were exposed to freeze/thaw and wet/dry cycling in solutions containing different chloride and acetate salts. These salts are currently used as deicers or have been proposed as alternatives to those currently used. Our study observed that magnesium in any form was very damaging to the concrete. Magnesium chloride produced significant concrete crumbling because of widespread replacement of CSH by non-cementitious MSH. Calcium magnesium acetate solutions were the most damaging of all solutions tested. Wet/dry and freeze/thaw cycling in CMA produced widespread and severe damage with scaling from replacement of calcium silicate hydrate with non-cementitious magnesium silicate hydrate. Magnesium acetate produced similar damage and calcium acetate solutions produced much less alteration. On the basis of our experiments, the use of CMA with a high Ca:Mg ratio seems advisable to reduce potential premature magnesium-induced deterioration.

Calcium chloride deicing salts caused characteristic deterioration in concrete containing reactive dolomite coarse aggregate by enhancing dedolomitization reactions that release magnesium to form destructive brucite and MSH. Sodium chloride solutions were the least deleterious to concrete under our experimental conditions. All the chloride solutions produced chloroaluminate, apparently chiefly from replacement of pre-existing ettringite.

We must point out that the validity of extrapolating the results obtained in our experimental conditions to those occurring under road use conditions is uncertain, and that our results and conclusions should be taken as cautionary only. Long-term SEM/EDAX studies comparing highway concrete treated under road use conditions with CMA or other magnesium deicers with similar concrete treated only with NaCl conditions should be performed in order to evaluate the long-term safety of magnesium deicers. We also should emphasize that we obtained our CMA from mixtures of calcium acetate and magnesium acetate. Extrapolation of results obtained with our mixtures to those that might result from the use of proprietary CMA may not be valid.

ACKNOWLEDGMENTS

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REFERENCES

An Update on Kansas’ Experience with PCCP Smoothness Specifications and Incentives

JEFFREY HANCOCK AND MUSTAQUE HOSSAIN

The smoothness or riding comfort of Portland cement concrete pavements (PCCP) is the highest indicator of quality from the user’s perspective. Therefore, the smoothness of newly constructed PCCP is of high interest. Since its development in 1990, the Kansas PCCP smoothness specification has undergone several revisions. The 1996 revision changed the incentive/disincentive payment from a percent of bid unit cost for the PCCP paving basis to a dollar-based value. This revision of the PCCP smoothness specification is primarily an attempt to make this smoothness specification more compatible with the asphalt concrete smoothness specification, which has been based on dollar value. This paper primarily outlines the current PCCP smoothness specifications in use in Kansas and also updates this development. Key words: PCCP, smoothness, profilograph, specification, incentives, and disincentives.

INTRODUCTION

Pavement roughness can be described by the magnitude of longitudinal profile irregularities and their distribution over the measurement interval and consists of random multifrequency waves of many wavelengths and amplitudes. Longitudinal roughness has been defined as (1):

"the longitudinal deviations of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality and dynamic pavement load."

ASTM (2) defines roughness as:

The deviations of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage, for example, longitudinal profile, transverse profile and cross slope.

Pavement smoothness is a lack of roughness. This is a more optimistic view of the road condition. Pavement profiles and detailed recordings of surface elevations are frequently used to characterize smoothness. Different wavelengths will have different effects on ride quality depending upon vehicle characteristics and driving speed. Thus, smoothness is an important indicator of pavement riding comfort and safety. From an auto driver’s point of view, rough roads mean discomfort, decreased speed, potential vehicle damage, and increased operating cost. A 1995 National Quality Initiative (NQI) national customer survey showed the following priorities for improving highways (3):

- Pavement Conditions 36%
- Safety 22%
- Traffic Flow 16%
- Visual Appeal 11%
- Bridge Condition 6%
- Maint. Response Time 6%
- Travel Amenities 3%

It is clear that highway users demand a good pavement condition—the ride quality is a function of it. According to Hudson (4), the purposes for smoothness measurement are:

1. To maintain construction quality control
2. To locate abnormal changes in the highway, such as drainage, subsurface problems, or extreme construction deficiencies
3. To establish a statewide basis for allocation of road maintenance resources
4. To evaluate pavement serviceability-performance life histories for evaluation of alternate designs.

The road surface smoothness on newly constructed Portland cement concrete pavement (PCCP) is of major concern to the Kansas Department of Transportation (KDOT). The first PCCP with smoothness specification was built by KDOT in 1985, and the first standard specifications were adopted in 1990. The purpose of the specification was to maintain construction quality control.

There is a growing concern in the transportation industry for smoother and smoother pavements. In a 1990 NCHRP study, it was shown that of the 36 states reporting, 80 percent exercised smoothness criteria on new pavement construction (5). Just two years later, in another NCHRP study, it was shown that of the 22 states reporting, 91 percent utilized smoothness criteria on construction of new pavements (5). A trend toward smoother and smoother pavements will require specifications that are attainable and practical for the contractor.

Since KDOT adopted its first PCCP smoothness specification in 1990, the quality of concrete paving has improved in Kansas. The 1990 specification gives contractors either incentive or penalty payments based on a percentage of the contract bid item price. To make the specification more compatible with the asphalt concrete (AC) smoothness specification, which is based on a dollar amount of incentive or penalty, KDOT revised the 1990 PCCP smoothness specification in 1996 and based it on a dollar amount of incentive or penalty.

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DEVELOPMENT OF PCCP SMOOTHNESS SPECIFICATIONS IN KANSAS

In 1985, KDOT selected a 7.63 m (25 ft) California-type profilograph using the 5.1 mm (0.2 in) blanking band for evaluation of the profilograph used in determining the smoothness of newly constructed concrete pavements. At that time, KDOT developed a provisional set of specifications and tested their attainability on three projects over two construction seasons. The results indicated that the provisional smoothness specifications were attainable and resulted in better quality pavements. In 1990, the specifications shown in Table 1 were adopted as standards for quality control of as-built concrete pavement smoothness in Kansas (6).

Table 1 Schedule for Adjusted Payment for PCCP (1990 Specification 502.06)

<table>
<thead>
<tr>
<th>Profile Index mm/km (0.16 km section)</th>
<th>Price Adjustment Percent of Contract Unit Bid Price</th>
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<tbody>
<tr>
<td>48 or less</td>
<td>106</td>
</tr>
<tr>
<td>49 to 64</td>
<td>103</td>
</tr>
<tr>
<td>65 to 159</td>
<td>100</td>
</tr>
<tr>
<td>160 to 191</td>
<td>96</td>
</tr>
<tr>
<td>192 to 222</td>
<td>92</td>
</tr>
<tr>
<td>223 to 238</td>
<td>90</td>
</tr>
<tr>
<td>239 or more</td>
<td>88</td>
</tr>
<tr>
<td>(Corrective Work Required)</td>
<td></td>
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</tbody>
</table>

In 1990, there was a noticeable, high-frequency vibration on a PCCP reconstruction project on I-70. On a concurrent project on I-470, such a problem did not exist. Viewing the profilograph traces more closely revealed a sine-wave oscillation of about 2.44 m (8 ft) spacing with a 5.1 mm (0.2 in) amplitude. However, most of the surface deviations were covered up by the 5.1 mm (0.2in) blanking band width during the trace reduction. On the I-470 project the oscillation waves were spaced at about 9.14 m (30 ft) with an amplitude of 5.1 mm (0.2 in), which were again covered by the 5.1 mm (0.2 in) blanking band width (6).

The I-70 and I-470 projects of 1990 prompted KDOT to study the effects of the blanking band width on trace reductions. It was decided to use a “zero” blanking band width or “null” blanking band. A null blanking band is nothing more than a reference line placed approximately at the center of the trace. Each of the 1990 projects was reanalyzed using the null blanking band. By replacing the 5.1 mm blanking band with the null blanking band achieving bonus sections became more difficult. The change in the blanking band width resulted in a new specification for PCCP smoothness, 90P-111. The new specification was incorporated into the 1992 construction projects (6).

Revisions to the original 1990 specification continued to occur. In 1992, another revision was made to the PCCP smoothness specification, 90P-111. With the introduction of 90P-111-R1 in 1993, the maximum amount of bonus was increased from 6% of the unit bid price to 8% of the unit bid price, but the full pay range was narrowed to include slightly more rigid grind-back provisions. In 1994, 90P-111-R2 and 90P-111-R3 were intended to make pavements initially smoother by lowering the PRI values required for the highest, 108%, incentive payment. In 1996, the specification took a major turn and replaced the percent unit bid item price incentive with a dollar value incentive with 90P-111-R4. Revisions continued and are continuing to include such changes as requiring ProScan automated profilogram reduction software, grinding provisions, and a 7.62 mm (0.3 in) bump template (7).

ORIGINS OF THE DOLLAR-BASED INCENTIVE/DISINCENTIVE PAYMENTS, 90P-111-R4

In 1996, it was decided to introduce 90P-111-R4. This revision to the original PCCP smoothness specification changed the incentive from percent of unit bid price to a dollar value. Table 2 shows the PRI ranges and the incentive or disincentive dollar values associated with each. The specification took on a new form that had only previously been used for in the AC smoothness specification.

Table 2 Schedule for Adjusted Payment for PCCP (1996 Specification 90-111-R4)

<table>
<thead>
<tr>
<th>Profile Index mm/km (0.16 km section)</th>
<th>Contract Price Adjustment per 0.16 km section per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>160 or less</td>
<td>+$845.00</td>
</tr>
<tr>
<td>161 to 240</td>
<td>+$630.00</td>
</tr>
<tr>
<td>241 to 285</td>
<td>+$420.00</td>
</tr>
<tr>
<td>286 to 475</td>
<td>+$315.00</td>
</tr>
<tr>
<td>476 to 710</td>
<td>$0.00</td>
</tr>
<tr>
<td>711 or more</td>
<td>-$530.00</td>
</tr>
</tbody>
</table>

This revision to the PCCP smoothness specification was done for two reasons. The first was to make the PCCP smoothness specification more compatible with the AC smoothness specification, which has always been based on dollar value. The second reason was to put an actual value on smoothness. Contractors have a better understanding of what it is worth to KDOT to have newly constructed PCCP as per the smoothness outlined in the specifications. The 1996 revision did have a somewhat positive effect on the number of sections constructed in the bonus range and penalty range as evident by Figure 1.
The AC pavement smoothness specification has always been based on the dollar value incentive or disincentive. KDOT lets asphalt paving construction as unit bid item tonnage. Since asphalt prices are based on the tonnage of material used and not the actual amount of coverage, it was logical to base the AC smoothness specification on a dollar value that was related to the price of one ton of asphalt concrete production, placement, and compaction. This logic was the reason for the dollar value-based incentive and disincentive payments in the AC pavement smoothness specification.

The original PCCP smoothness specification was based on percent of bid item. The PCCP smoothness specification was designed this way because concrete pavement is usually bid as unit item square meter. However, in 1996, KDOT decided to make the PCCP smoothness specification more compatible with the AC pavement smoothness specification. The dollar value-based incentive and disincentive specification, 90P-111-R4, shown previously in Table 2, was adopted.

The derivation of the dollar values was based on the average cost of a square meter of concrete pavement (8). The average cost of construction was then increased by the previous PCCP smoothness specification percentages to determine the actual dollar amounts of bonus or penalty.

- The total number square meters in one 3.66 m (12 ft) lane 0.16 km (0.1 mile) long:
  \[3.66 \text{ m} \times 160 \text{ m} = 585.6 \text{ square meters}\]
- The cost of this section of concrete pavement (based on the statewide average cost of doweled, plain jointed concrete pavement for KDOT):
  \[585.6 \text{ square meters} \times \$18.03 \text{ per square meter} = \$10,558.36\]
- The price of this section including incentive payment (maximum 108 percent, based on 90P-111-R3 adjusted schedule):
  \[\$10,558.36 \times 1.08 = \$11,403.04\]
- The maximum amount of incentive for a 0.16 km (0.1 mile) section:
  \[\$11,403.04 - \$10,558.36 = \$844.68 \approx \$845.00\]

The dollar amounts for the other smoothness (profile index) ranges were determined in the same way. Although the penalty amount may appear extreme (-$530.00), it too was based solely on 95 percent of the average cost per section. When put in terms of dollars, contractors are expected to be more discouraged from committing contractor negligence.

**CONCLUSIONS**

The PCCP smoothness specifications in Kansas, adopted by KDOT in 1990, has been revised several times to take advantage of our growing knowledge in PCCP smoothness. The transition to the dollar-based incentive or disincentive PCCP smoothness specification has given KDOT and contractors a better understanding of the value of smoothness. Also, the PCCP smoothness specification is now more comparable to the AC pavement smoothness specification. This allows contractors and KDOT to better relate and compare the cost of AC pavement versus PCCP in terms of smoothness. By attaching dollar signs to bonus and penalty pavements, the desire to achieve smoother and smoother pavements has only increased. As the measure of pavement smoothness evolves, adjustments to the specifications will continue to be made.

**REFERENCES**

Evaluation of an Automated Horn Warning System at Three Highway-Railroad Grade Crossings in Ames, Iowa

STEVE J. GENT, SCOTT LOGAN, AND DAVID EVANS

Traditionally, locomotive engineers begin sounding the train horn approximately ¼ mile from the crossing to warn motorists and pedestrians approaching the intersection. To be heard over this distance, the train horn must be very loud. This combination of loud horns and the length along the tracks that the horn is sounded creates a large area adversely impacted by the horn noise. In urban areas, this area likely includes many nearby residents. The automated horn system provides a similar audible warning to motorists and pedestrians by using two stationary horns mounted at the crossing. Each horn directs its sound toward the approaching roadway. The horn system is activated using the same track signal circuitry as the gate arms and bells located at the crossing. Once the horn is activated, a strobe light begins flashing to inform the locomotive engineer that the horn is working. The purpose of this research was twofold: 1) to determine the effectiveness of the automated horn system in reducing the annoyance level for nearby residents; and 2) to determine the overall safety at the crossings with the new automated horn warning system. The research included collecting horn volume data to develop noise level contour maps, using before-and-after surveys to document opinions of nearby residents and motorists and a survey of locomotive engineers to document their perception of the new systems.

INTRODUCTION

In September of 1998, the city of Ames, Iowa (population 48,000) began operation of three automated horn warning systems. The systems were installed at crossings already equipped with automatic flashing light signals with gate arms and constant warning time circuitry. These systems were installed after nearby residents repeatedly expressed their concerns over the disturbance created by the loud train horns. Currently about 60 trains per day pass through Ames, and this number is expected to increase to around 100 trains per day within five years.

Traditionally, locomotive engineers begin sounding the train horn approximately ¼ mile from the crossing to warn motorists and pedestrians approaching the intersection. To be heard over this distance, the train horn must be very loud. This combination of loud horns and the length along the tracks that the horn is sounded creates a large area adversely impacted by the horn noise. Unfortunately, in urban areas, this area likely includes many nearby residents.

The automated horn system provides a similar audible warning to motorists and pedestrians by using two stationary horns mounted at the crossing. Each horn directs its sound toward the

<table>
<thead>
<tr>
<th>Sound Level (dBA)</th>
<th>Train Horn Area (acres)</th>
<th>AHS Horn Area (acres)</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 90</td>
<td>31</td>
<td>&lt; 1</td>
<td>98%</td>
</tr>
<tr>
<td>&gt; 80</td>
<td>171</td>
<td>5</td>
<td>97%</td>
</tr>
<tr>
<td>&gt; 70</td>
<td>265</td>
<td>37</td>
<td>86%</td>
</tr>
</tbody>
</table>

After conducting this part of the study, it became apparent that two additional issues related to horn volume should be addressed through future research. The issues are: 1) what horn decibel volume is required to adequately warn an approaching motorist, and 2) at what distance from the crossing does that volume need to be provided? To give a reference to the first question, the following are some typical decibel readings: a food blender at 3 feet, 87 dBA; a person shouting at 3 feet, 78 dBA; a gas lawn mower at 100 feet, 70 dBA; and a person speaking normally at 3 feet, 65 dBA.

When assessing the relative loudness of a given decibel level, it is helpful to understand the relationship between these two terms. The above typical decibel levels and the following excerpt were taken from the 1987 AASHTO Guide on Evaluation and Attenuation of Traffic Noise publication. It states, “An increase of 10 dBA in sound level will nearly double the loudness as rated subjectively by typical observers…A decrease of 10 dBA will appear to an observer to be a halving of the apparent loudness. For example, a noise of 70 dBA will sound only half as loud as 80 dBA, assuming the same frequency composition and other things being equal.”
The issue related to distance may be approached by looking at Table II-1, A Guide for Advance Warning Sign Placement Distances found in the Manual on Uniform Traffic Control Devices. This table gives a minimum sign placement distance of 450 feet for a “STOP AHEAD” sign on a 55 mph roadway. The distance is 300 feet for 45 mph roadways and 150 feet for 35 mph roadways. These distances provide adequate time for the driver to perceive, identify, decide and perform the necessary maneuver. For highway-railroad intersections, these minimum distances present a reasonable starting point for the establishment of a requirement for an audible warning distance.

To look at the variability in train horn volumes, from one train to another, 12 readings were collected 250 feet from the tracks. The twelve readings averaged 95.5 dBA, with a low reading of 90.6 dBA, a high reading of 102.8 dBA, and a standard deviation of 3.63 dBA.

**RESIDENT SURVEY**

Survey questionnaires were distributed to all residents living within an area located 1,000 feet perpendicular to the tracks and 1,500 feet longitudinal (each way) from the crossings. Surveys were distributed approximately two months before and two months after the automated horn systems were installed. The responses were overwhelmingly positive regarding the automated horn system. Figure 3 shows the before condition where 77 percent of the residents indicated the train horns had either a “negative” or “very negative” impact on their quality of life, compared to only 3 percent in the after condition.

**FIGURE 3 Impact of horn on residents’ quality of life**

At the end of each survey, the residents were solicited to write additional comments on the back of the form. Over half of the 550 returned surveys (approximately 1,000 total surveys distributed) provided comments. The following examples provide a good cross section of the issues and observations listed by the residents.

Before condition (train horns): “I understand the need for trains to make noise at intersections – to make their presence known to avoid accidents – but I don’t appreciate the engineers who feel the need to blow the horn for the entire length of their trip. I feel that is unneeded, especially at 3 a.m. when there is nobody out on the roads anyway!”

“The train whistles are way too loud and long in my estimation. If I’m on the phone or listening to the TV, the loud whistles are especially annoying. Also, my sleep is often interrupted many times during the night because of the loud whistles. It would be very much appreciated if the noise could be greatly softened while still keeping the crossing safe.”

After condition (automated horn system): “Installation of the automated horn system was a very positive step. There is an occasional train operator that still uses the train-mounted horn to make a statement as he/she passes through our neighborhood. This just reminds us of how much better the noise level is a majority of the time. Thank you for continuing to support our neighborhood in its efforts to improve the quality of life of the residents.”

“I have lived in this neighborhood nearly my entire life. I thought I was used to the train noise. However, with the many trains that go through now, and with the noisy horns, it was affecting my lifestyle. These new automated horns are great, and I really appreciate their installation. I used to worry when I had overnight company that they would be kept awake by the noise, and often they were. Now they aren’t, thank you.”

The comments received leave little question as to how appreciative the residents were of the automated horn system. To determine if the perpendicular distance from the tracks affected the survey responses, the distributed surveys were differentiated between the residents living within 500 feet of the tracks, and the residents living between 500 and 1,000 feet of the tracks. The residents living closer to the tracks were slightly more extreme in their survey responses. However, the residents living farther from the tracks shared the same concerns regarding the train horns and shared the same positive responses regarding the automated horns. Residents living farther than 1,000 feet were not included in the survey.

Figure 4 shows the residents’ rating of the before and after horn volume. In general, they felt the train horns were too loud, and the automated horns were not a problem.

**FIGURE 4 Residents’ rating of horn volume**

**MOTORIST SURVEY**

The motorists surveyed at the crossings generally liked the automated horn system and preferred this new system over the train horns. However, they did not feel as strongly as the residents about the need to reduce the volume of the train horns.

Figure 5 shows the results of the question, “What device first alerted you to the oncoming train?” The mix of responses indicates that each of the various warning devices (gates, flashing...
lights, horn, etc.) located at the crossings provides a valuable safety benefit.

**FIGURE 5** Warning device noticed first by motorists

Figure 6 shows the motorist opinion of the horn volume in the before (train horn) and after (automated horn warning system) situations. In both cases, the majority of motorist felt the volume should be left as is. It should be noted that some of the surveyed motorists were also residents living near the crossing. The number of residents was not determined during the survey.

**FIGURE 6** Motorists’ opinion of horn volume

One hundred and five motorists were surveyed in the before condition and fifty-one motorists were surveyed in the after condition. The after survey was conducted approximately one month after the automated warning system was installed. Seventy-five percent of the respondents indicated that they were aware that the automated horn system had been installed. The motorists preferred the automated horn system over the train horns 78 percent of the time, 8 percent of the motorists preferred the train horns over the automated horns, and 14 percent had no opinion.

**LOCOMOTIVE ENGINEER SURVEY**

In general, the locomotive engineer survey also provided positive responses regarding the automated horn warning system. The engineers completed the surveys in April of 1999, seven months after the installation of the automated horns. A total of 26 surveys were completed. Some highlights from the surveys include the following:

- The crossings were rated “safer” by 23 percent of the locomotive engineers, 69 percent rated them “about the same,” and only 8 percent rated the crossings with the automated warning systems to be “less safe” as compared to the before (train horn) condition.
- Only one locomotive engineer noted an increase in unsafe motorist behavior. The other 25 (96 percent) did not observe an increase.
- Seventy-three percent of the engineers admitted to blowing the train horn at least once at the subject crossings. The two primary reasons stated for blowing the train horns were concern related to motorist or pedestrian behavior at the crossing and that old habits are hard to break.

Figures 7 and 8 show the responses to two of the survey questions.

**FIGURE 7** Relative safety of crossing from engineers’ perspective

**FIGURE 8** Engineers’ reason for blowing the train horn

**SUMMARY**

This research project was initiated for the purpose of evaluating the effectiveness of the automated horn warning systems. The purpose was twofold: 1) to determine the effectiveness of the new system in reducing the annoyance level for nearby residents; and 2) to determine the overall safety at the crossings with the automated systems.

The effectiveness of the automated horn in reducing the annoyance level for nearby residents was addressed through the field collection of horn noise levels and through the surveys of
residents. The horn volume data that was collected near the crossings clearly demonstrates the significant reduction of land area negatively impacted by use of warning horns. In fact, the automated horn system reduced the area with noise levels greater than 80 dBA by 97 percent, from 171 acres, using the train horns, to less than six acres using the automated horn system. (For reference, a person shouting from a distance of three feet would produce a decibel reading of approximately 78 dBA.) The residents overwhelmingly accepted the automated horn system and appreciated the city staff for attending to their needs. In the before condition, 77 percent of the residents indicated the train horns had either a “negative” or “very negative” impact on their quality of life as compared to only 3 percent in the after condition. Regarding horn volume, 76 percent felt the train horn volume was “too loud,” as compared to the after condition where 82 percent indicated that the automated horn volume was “no problem”.

Because the city of Ames is only the third community to install automated horns, it is impossible to accurately determine the overall safety of the crossings. Only after more systems are installed can a study be conducted comparing the collision rates of crossings with similar exposures. Nonetheless, the motorist and locomotive engineer surveys provided valuable input into this issue. When the motorists were asked which system they preferred, 78 percent preferred the automated horn system, 8 percent preferred the train horns, and 14 percent had no opinion. Their responses also indicated that each of the warning devices (gates, flashing lights and train/automated horns) located at the crossings provides a valuable safety benefit. Twenty-three percent of the locomotive engineers rated the crossings “safer,” 69 percent rated them “about the same,” and only 8 percent rated the crossings with the automated warning systems to be “less safe” as compared to the before (train horn) condition.

In summary, the project found no evidence to suggest that the automated horns are less safe than the current practice of using train-mounted horns. The automated horn system provides the locomotive engineer with the option of sounding the train’s horn if unsafe behavior at the crossing is observed. This option may enhance the safety at the crossing because it provides an additional level of warning. For pedestrians and bicyclists, the automated horns appear to provide a better audible warning because of the intense nature of the horn volume during the early stages of the warning time. However, the automated horns do not provide an indication as to the direction of the approaching train, which is one of the reasons these systems should only be considered at locations already equipped with automatic flashing light signals with gate arms and constant warning time circuitry. Other jurisdictions considering these systems may also want to use other supplementary safety measures, such as median barriers.
The 1998 Lincoln Model Update

MAREN L. OUTWATER, SAJJAD RASHEED, MIKE MALONE, MIKE BRIENZO, ANTHONY KROON, AND KAZI ULLAH

The 1998 Lincoln Model Update was conducted primarily to support the development of the 20-year Transportation Plan and included innovative concepts in transportation modeling: a Geographic Information System interface, estimation of local model parameters using the 1995 National Personal Transportation Survey (NPTS), and use of observed travel time to develop input speeds and speed-based measures of effectiveness (MOE). The GIS interface was developed to transfer data between the ARCINFO database and the transportation model network in TP+ format. This allows the transfer of roadway attributes for use in the model and model volumes, speeds, and travel times for display using the GIS maps. The 1995 NPTS was used to develop model parameters for Lincoln in lieu of a local household survey. Trip rates, lengths and peaking factors were estimated using samples within the census district including Nebraska. Results yielded significantly different model parameters than those used previously (developed from the NCHRP 187) and from national averages estimated from the NPTS. Data on travel time were observed for both off-peak and peak conditions, covering 35 percent of total roadway mileage in Lincoln. The off-peak travel time was used to develop a lookup table with representative operational input speeds by averaging speeds weighted by distance for each combination of functional type and area type. These speeds incorporate effects of signal delay, density of access points, weaving, and driver characteristics. The peak travel times were used to develop speed-based MOEs and as a validation tool for the model output speeds. Key words: Geographic Information Systems (GIS), travel forecasting, travel time and speed, performance measures.

INTRODUCTION

The 1998 Lincoln Model Update was conducted primarily to support the development of the 20-year Transportation Plan. In addition, this model supported a series of transportation planning studies, including the North 84th Street Study, the South 84th Street Study, and the S1-S2 Subarea Transportation Study. The 1998 Model Update included estimation of new model parameters using the 1995 National Personal Transportation Survey (NPTS). In addition, the 1998 Model Update was based upon observed data for travel time and speed collected during 1998. As a result, these models are substantially different than the 1995 Lincoln Models.

This paper documents the GIS interface developed for the model network and GIS databases, the development of the travel model and validation results, and the use of the travel time and speed study data in model development and validation. This model update was a collaborative process between the consultants and city staff. The 1998 Lincoln Model was developed using a new software package, TP+ and VIPER. The 1998 Lincoln Model meets the vast majority of typical validation standards and those set forth by City staff. The results are reasonable in total and for stratification of functional class, area type, and volume group.

GIS INTERFACE

The City of Lincoln has a GIS database in ARCINFO format that is quite detailed for land parcels and roadway networks. The purpose of developing an interface between the travel model and the GIS interface was:

· To allow the GIS database to access model network attributes, such as volumes, travel times, etc., for display purposes;
· To allow the travel model to access GIS database attributes, such as road names, lengths, etc., for analytical purposes; and
· To allow continuous update of model inputs, such as counts, capacity, etc.

The methodology used was to develop a unique identifier for model network links and transfer this unique identifier to the corresponding GIS street segment. An ARCVIEW software system was developed using AVENUE to facilitate the initial development of this interface (1), then city staff carried out the actual development. There were a number of issues related to integration that were identified and resolved during the development and are presented in Table 1.

<table>
<thead>
<tr>
<th>Issue</th>
<th>Solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>GIS and model network were not in the same coordinate system.</td>
<td>Model network database coordinate system was transformed using linear regression.</td>
</tr>
<tr>
<td>GIS and model network topologies were not consistent.</td>
<td>Tools were developed to split GIS street segments to match model network topology.</td>
</tr>
<tr>
<td>Model network directionality needed to be represented in GIS.</td>
<td>Directional data were transferred to the GIS street segments in directional formats.</td>
</tr>
<tr>
<td>Model network link identifiers needed to be manually transferred to GIS street segments.</td>
<td>Custom tools were developed to transfer model network link identifier to GIS street segments.</td>
</tr>
</tbody>
</table>

TRAVEL MODEL DEVELOPMENT

Travel Characteristics Data

The 1998 Lincoln Travel Model was based on travel characteristics data derived from household travel surveys conducted by the U.S. Department of Transportation in a program called the National Personal Transportation Survey (NPTS). For this study, the 1995 NPTS data were obtained and analyzed for the West North Central Region, including Nebraska, South Dakota, North Dakota, Iowa, Kansas, Minnesota, and Missouri. Household surveys were collected for more than 42,000 households nationwide and 1,478 households in the West North Central Region. These data are a valuable source of travel characteristics data for cities and states that do not conduct individual household surveys.

The NPTS data were processed for use in estimating trip generation, trip distribution, and peaking factor models for the City of Lincoln. This data processing reduced the full national household survey dataset to only those households in the West North Central region reporting weekday travel. The results of this data processing are 974 households in the sample, weighted to represent 4,738,217 households in the West North Central region and 7,498 trips in the sample, weighted to represent 42,061,090 trips in the West North Central region. The NPTS data were evaluated using the two key household characteristics variables in the Lincoln Model: dwelling unit type and area type.

Trip Generation

The weekday vehicle trips from the 1995 NPTS data were compared to the households from the same sample to estimate average weekday vehicle trip rates by household category. The household categories were developed from the Lincoln land use data for two categories of dwelling units and three categories of area types. These trip rates were calculated for the West North Central region, and the full national database and are presented in Table 2. Table 2 also presents a comparison of these trip rates to the 1995 Lincoln Model trip rates. For this study, the 1995 NPTS data were obtained and analyzed for the West North Central Region, including Nebraska, South Dakota, North Dakota, Iowa, Kansas, Minnesota, and Missouri. The weekday vehicle trips from the 1995 NPTS data were compared to the households from the same sample to estimate average weekday vehicle trip rates by household category. These trip rates were calculated for the West North Central region, and the full national database and are presented in Table 2. Table 2 also presents a comparison of these trip rates to the 1995 Lincoln Model trip rates.

### Table 2: Average Vehicle Trip Rates from the 1995 NPTS and 1995 Lincoln Model

<table>
<thead>
<tr>
<th>Dwelling Unit Type</th>
<th>Area Type</th>
<th>1995 NPTS West North Central Region</th>
<th>1995 National Database</th>
<th>1995 Lincoln Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Family &amp; Duplex Urban</td>
<td>8.92</td>
<td>10.12</td>
<td>7.65</td>
<td></td>
</tr>
<tr>
<td>Rural</td>
<td>8.43</td>
<td>7.38</td>
<td>11.40</td>
<td></td>
</tr>
<tr>
<td>Subtotal</td>
<td>9.17</td>
<td>8.72</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Multi-Family Urban</td>
<td>7.32</td>
<td>7.25</td>
<td>6.50</td>
<td></td>
</tr>
<tr>
<td>Suburban</td>
<td>7.70</td>
<td>6.28</td>
<td>6.50</td>
<td></td>
</tr>
<tr>
<td>Rural</td>
<td>7.48</td>
<td>6.45</td>
<td>6.50</td>
<td></td>
</tr>
<tr>
<td>Subtotal</td>
<td>7.46</td>
<td>6.85</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Households Urban</td>
<td>8.55</td>
<td>8.96</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Suburban</td>
<td>9.64</td>
<td>7.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rural</td>
<td>8.34</td>
<td>7.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subtotal</td>
<td>8.88</td>
<td>8.16</td>
<td>10.0</td>
<td></td>
</tr>
</tbody>
</table>

Lincoln Model trip rates are lower than the NPTS West North Central region trip rates except for suburban and rural single family households, which are 35 and 14 percent higher respectively. The 1995 Lincoln Model trip rates were based on ITE Trip Generation Manual (5th Edition, 1991) and then modified to account for differences in urban and suburban rates for single family households. Rural households were not separated from suburban households in this evaluation.

The 1995 NPTS data was further evaluated by trip purpose in order to expand the trip purposes in the 1998 Model. The 1995 Lincoln Model had three trip purposes: home-based work, home-based other, and non-home-based. One of the advantages of using the 1995 NPTS survey data instead of the NCHRP 187 manual for deriving trip rates is that there are more choices of trip purpose categories than the three reported in the NCHRP 187 manual. These additional trip purposes can have significant effects on future forecasts of land development for specific uses such as shopping or recreation that would not be accounted for otherwise. There are also differences in trip purposes that are accounted for by using the more updated data source (1995 data instead of the 1978 data used in NCHRP) which probably accounts for the significant increase in non-home-based travel (see Table 3).

### Table 3: Trip Purposes for the 1999 Lincoln Model Compared to the 1995 Lincoln Model

<table>
<thead>
<tr>
<th>Trip Purpose</th>
<th>1998 Lincoln Model (percent of total trips)</th>
<th>1995 Lincoln Model (percent of total trips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data Source</td>
<td>1995 NPTS for West North Central Region</td>
<td>NCHRP 187 Quick Response Manual (1978)</td>
</tr>
<tr>
<td>Home-Based Work</td>
<td>24</td>
<td>25</td>
</tr>
<tr>
<td>Home-Based Shop</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Home-Based Recreational</td>
<td>13</td>
<td>66</td>
</tr>
<tr>
<td>Home-Based Other</td>
<td>19</td>
<td></td>
</tr>
<tr>
<td>Non-Home-Based</td>
<td>34</td>
<td>9</td>
</tr>
</tbody>
</table>
Trip Distribution

Trip lengths are defined by the average trip length as well as the trip length frequency distribution for each trip purpose. Observed data on trip lengths were developed from the 1995 NPTS data. Friction factors were developed using a gamma function to estimate the friction factors and application of the trip distribution model to identify the “best-fit” for the average trip length and trip length frequency distributions. The gamma functions used to develop these functions used the following equation:

\[ \text{Alpha} \times (1 + \frac{I}{\text{Beta}} \times e^{\frac{-I}{\text{Gamma}}}) \]

Where:

- Alpha, Beta and Gamma are coefficients, and
- I is the impedance, or trip length in minutes.

A comparison of average trip lengths by trip purpose is provided in Table 4. The 1995 NPTS average trip lengths were used to estimate friction factors for use in the 1998 Lincoln Model, but these trip lengths produced traffic volumes considerably higher than traffic counts in Lincoln. This would indicate the average trip lengths are slightly lower in Lincoln than in other areas around the West North Central region. Two other cities and the previous 1995 Lincoln Model trip lengths are provided for comparison. The 1995 Lincoln Model used friction factors provided by the Federal Highway Administration that were established in the 1990 model development effort (3). The home-based work and non-home-based trip lengths are the most similar, with the home-based shopping and recreational trip lengths being the most different from the 1995 Lincoln Model, confirming the benefits of incorporating separate trip purposes.

### TABLE 4  Average Trip Lengths for the 1998 Lincoln Model in Minutes

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Home-Based Work</td>
<td>12.4</td>
<td>12.9</td>
<td>11.2</td>
<td>17.7</td>
<td>12.8</td>
</tr>
<tr>
<td>Home-Based Shop</td>
<td>9.4</td>
<td>10.1</td>
<td>8.6</td>
<td>10.3</td>
<td></td>
</tr>
<tr>
<td>Home-Based Recreational</td>
<td>9.9</td>
<td>11.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Home-Based Other</td>
<td>10.4</td>
<td>11.2</td>
<td>10.4</td>
<td>12.3</td>
<td>11.4</td>
</tr>
<tr>
<td>Non-Home-Based</td>
<td>10.6</td>
<td>11.9</td>
<td>8.1</td>
<td>11.9</td>
<td>10.5</td>
</tr>
</tbody>
</table>

Screenlines are usually indicative of whether the trip distribution is reasonable because they will identify patterns of east-west or north-south movements. There are eight screenlines in the Lincoln Model. Table 5 presents the results of the screenlines compared to the results for these same screenlines in the 1995 Lincoln Model. In all but three cases, the screenlines meet the +/- 5 percent goal. In many cases, the 1998 Model has better results on the screenlines than the 1995 Model. The results of the screenline analysis indicate that the pattern of trip movements is reasonable compared to observed values.

### TABLE 5  1998 Model Validation of Screenlines

<table>
<thead>
<tr>
<th>Screenline</th>
<th>1998 Model</th>
<th>1995 Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 56th Street</td>
<td>-3.9%</td>
<td>1.6%</td>
</tr>
<tr>
<td>2 27th Street</td>
<td>1.9%</td>
<td>-2.2%</td>
</tr>
<tr>
<td>3 A Street</td>
<td>8.4%</td>
<td>8.5%</td>
</tr>
<tr>
<td>4 Adams Street</td>
<td>2.3%</td>
<td>-6.4%</td>
</tr>
<tr>
<td>5 Havelock Ave</td>
<td>-3.6%</td>
<td>-7.6%</td>
</tr>
<tr>
<td>6 Old Cheney Rd</td>
<td>-5.9%</td>
<td>-2.7%</td>
</tr>
<tr>
<td>7 84th Street</td>
<td>-2.3%</td>
<td>-19.5%</td>
</tr>
<tr>
<td>8 Coddington Ave</td>
<td>7.6%</td>
<td>-10.1%</td>
</tr>
</tbody>
</table>

Goal for Screenlines is +/-5%

Total 1.1% -1.5%

Trip Assignment

Trip assignment is typically validated by comparing traffic counts to model volumes for different market segments and to summarize system-wide variables. The summary of system-wide statistics is presented in Table 6 and reflects strong correlation between volumes, speeds, and travel times for the 1998 Model Update.

### TABLE 6  Summary of System-Wide Statistics for 1998 Model

<table>
<thead>
<tr>
<th></th>
<th>1998 Model</th>
<th>1998 Counts</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicle Miles Traveled</td>
<td>2,494,501</td>
<td>2,523,139</td>
<td>-1.1%</td>
</tr>
<tr>
<td>Total Vehicle Hours Traveled</td>
<td>934,51</td>
<td>94,590</td>
<td>-1.2%</td>
</tr>
<tr>
<td>Free Flow Hours Traveled</td>
<td>83,498</td>
<td>84,457</td>
<td>-1.1%</td>
</tr>
<tr>
<td>Delay Hours</td>
<td>9,953</td>
<td>10,134</td>
<td>-1.8%</td>
</tr>
<tr>
<td>Average Congested Speed (MPH)</td>
<td>27.86</td>
<td>26.67</td>
<td>4.4%</td>
</tr>
<tr>
<td>Average Speed (MPH)</td>
<td>29.87</td>
<td>29.87</td>
<td>—</td>
</tr>
</tbody>
</table>

The summaries of traffic counts and modeled volumes by functional class are presented in Table 7. These classifications also have established goals of percent deviation that are presented in the table. All of the classifications meet the goals for percent deviation. These results are also compared to the 1995 Lincoln Model in Table 7, but very few differences exist. This may be due to the fact that the 1995 Model employed facility-specific data on speeds and capacities that were adjusted to achieve the best fit to the traffic count data, where the 1998 Lincoln Model relied on categorical speeds and validation of the speeds to observed data. The 1998 Lincoln Model did separate divided highways from the principal arterial category and gravel roads from the collector category of roads.
TRAVEL TIME AND SPEED STUDY

Travel time and speed is a critical input in the Lincoln travel model and is also used to validate the travel model output. In recent research, travel models give more realistic results with operational speeds as inputs compared to the more traditional posted speeds. Operational speeds account for effects of signals, density of access points, and driver characteristics. Recent research and practice has also shown that speed-based performance measures are better indicators of system-wide performance than traditional level-of-service based performance measures. Speed provides a direct connection between system-wide transportation planning and project implementation. Speed is also an easier concept to understand by the general public and easier to measure in the field. As a result, the Mayor of the City of Lincoln’s Congestion Management Task Force (CMTF) has recommended speed-based performance measures be used to identify and select future transportation improvement projects.

Data Collection

A data collection effort was undertaken to obtain current speed data for the planning process. Representative corridors were selected in the sample of roads to be surveyed and speeds were observed with test runs during the midday and PM peak periods. These data were combined with travel time and speed data collected in a previous study for the City of Lincoln. A minimum of three runs (or sample size) was required for a confidence level of 95%, with a permitted error of ±5.0 MPH, assuming a range of 10 MPH in the average running speed. The sample included 146 miles of roadways, which constitutes about 35 percent of total roadway mileage in the Lincoln area. This sample size was considered statistically adequate to represent all functional classes of Lincoln area roadways. Speeds were analyzed by area type and functional classification to evaluate the reasonableness for each category. Table 8 summarizes the total mileage in the Lincoln area and the mileage included in the sample. Table 9 summarizes the number of link segments where speed and travel time data were collected by area type and functional classification.

### Table 7 1998 Model Validation by Functional Classification

<table>
<thead>
<tr>
<th>Functional Classification</th>
<th>Percent Difference</th>
<th>Goal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1998 Model</td>
<td>1995 Model</td>
</tr>
<tr>
<td>Freeways &amp; Ramps</td>
<td>4.3%</td>
<td>-3.9%</td>
</tr>
<tr>
<td>Divided Highway</td>
<td>1.8%</td>
<td>10.0%</td>
</tr>
<tr>
<td>Principal Arterial</td>
<td>1.8%</td>
<td>3.8%</td>
</tr>
<tr>
<td>Minor Arterial</td>
<td>-1.9%</td>
<td>-0.4%</td>
</tr>
<tr>
<td>Collectors</td>
<td>23.4%</td>
<td>-3.6%</td>
</tr>
<tr>
<td>Gravel Roads</td>
<td>-24.4%</td>
<td>N/A</td>
</tr>
<tr>
<td>Total</td>
<td>1.3%</td>
<td>1.2%</td>
</tr>
</tbody>
</table>

### Table 8 Miles of Roadway by Functional Class (within the Lincoln Cordon Area)

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>Total Roadway Mileage</th>
<th>Model</th>
<th>Sample Mileage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Urban/Rural Interstate</td>
<td>28.9</td>
<td>Freeways</td>
<td>11.5</td>
</tr>
<tr>
<td>Urban/Rural Principle Arterial</td>
<td>95.3</td>
<td>Principle Arterials</td>
<td>46</td>
</tr>
<tr>
<td>Urban/Rural Minor Arterial</td>
<td>110.9</td>
<td>Minor Arterials</td>
<td>68.5</td>
</tr>
<tr>
<td>Urban Collector</td>
<td>71.7</td>
<td>Collectors</td>
<td>11</td>
</tr>
<tr>
<td>Rural Major Collector (State)</td>
<td>3.5</td>
<td>Divided Highways</td>
<td>9</td>
</tr>
<tr>
<td>Rural Major Collector (County)</td>
<td>101.4</td>
<td>Total</td>
<td>146</td>
</tr>
<tr>
<td>Rural Minor Collector</td>
<td>11.8</td>
<td>423.5 Total</td>
<td>1,264</td>
</tr>
</tbody>
</table>

### Table 9 Count of Link Segments included in Speed Data Collection

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>CBD</th>
<th>Urban</th>
<th>Suburban</th>
<th>Rural</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway</td>
<td>-</td>
<td>4</td>
<td>45</td>
<td>4</td>
<td>53</td>
</tr>
<tr>
<td>Divided Highway</td>
<td>-</td>
<td>11</td>
<td>25</td>
<td>5</td>
<td>41</td>
</tr>
<tr>
<td>Principal Arterial</td>
<td>46</td>
<td>231</td>
<td>167</td>
<td>7</td>
<td>451</td>
</tr>
<tr>
<td>Minor Arterial</td>
<td>12</td>
<td>353</td>
<td>236</td>
<td>6</td>
<td>607</td>
</tr>
<tr>
<td>Collector</td>
<td>-</td>
<td>35</td>
<td>74</td>
<td>3</td>
<td>112</td>
</tr>
<tr>
<td>Total</td>
<td>58</td>
<td>634</td>
<td>547</td>
<td>25</td>
<td>1,264</td>
</tr>
</tbody>
</table>

Analysis

The analysis of speeds collected in Lincoln began with a comparison of the average speeds—for each functional class and area type category for the original 1995 Lincoln Model,—the midday speeds, and the PM peak speeds. These results were refined in cases where sample size for a category were too small to be reliable, and these speeds can be identified by the even numbers where the actual speeds from the speed study are identified by exact numbers (with one decimal place).

The speeds used in the 1998 Model were compared to the average weighted speeds contained in the 1995 Lincoln Model for the same functional class and area type categories in Table 10. The speeds in the 1995 Model were not used according to these functional class and area type categories, so this comparison is for information only. The speeds used in the 1995 Model were coded specifically for each link based on a combination of observed data and engineering judgment.

The comparison of average speeds resulted in the following conclusions:

- The average midday speeds (33.0 miles per hour) are almost the same as the 1995 original speeds (31.8 miles per hour) overall, but there are significant differences for freeways, rural arterials, collectors, and minor arterials in the Central Business District (CBD).
TABLE 10  Recommended Speeds for the 1998 Model Update

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>CBD</th>
<th>Urban</th>
<th>Suburban</th>
<th>Rural</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeways 1995 Model Speeds</td>
<td>50.0</td>
<td>50.0</td>
<td>57.6</td>
<td>60.0</td>
</tr>
<tr>
<td>Freeway Ramps</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
</tr>
<tr>
<td>Divided Highways 1995 Model Speeds</td>
<td>42.0</td>
<td>42.0</td>
<td>45.0</td>
<td>48.0</td>
</tr>
<tr>
<td>Principal Arterials 1995 Model Speeds</td>
<td>22.7</td>
<td>29.1</td>
<td>36.2</td>
<td>44.0</td>
</tr>
<tr>
<td>Minor Arterials 1995 Model Speeds</td>
<td>22.0</td>
<td>25.5</td>
<td>29.3</td>
<td>36.0</td>
</tr>
<tr>
<td>Collectors 1995 Model Speeds</td>
<td>20.0</td>
<td>24.9</td>
<td>29.7</td>
<td>34.0</td>
</tr>
<tr>
<td>Gravel Roads 1995 Model Speeds</td>
<td>31.0</td>
<td>31.0</td>
<td>31.0</td>
<td>31.0</td>
</tr>
</tbody>
</table>

- Peak speeds (31.1 miles per hour) are, as expected, lower than midday speeds overall but freeways actually show higher peak speeds than midday speeds (although this is probably within the margin of error for the number of samples collected).
- The average speeds using the highest of the midday and PM peak speeds (34.4 miles per hour) are 1.4 mph faster than the average midday speeds. There are 381 links out of the 1,264 total links (30 percent) that have a midday speed lower than PM peak speed. Of these, 16 percent (197 links) are more than ten percent lower than PM peak speeds. Recommended speeds for input to the 1998 Lincoln Travel Model were derived from the average weighted midday speeds, with minor adjustments, as follows:
  - The only freeway classification with a statistically significant sample is suburban freeways at 57.6 mph. Other freeway segments are set according to judgement and were validated using model volumes and traffic counts.
  - Divided highways also have low sample size for urban and rural classifications and are set based on the validation of model volumes and traffic counts for these facilities. Suburban divided highways are set according to observed values.
  - CBD minor arterials also suffer from low sample size and are considered not statistically different than urban minor arterials. As a result, CBD minor arterials are set at 22 mph rather than the 28.2 mph estimated for this category.
  - CBD collectors are set at 20 mph because there were no CBD collectors in the sample.
- All classifications of rural roads have insignificant sample sizes and are difficult to specify as a result. Further data collection of rural facilities is recommended. Rural divided highways and principal arterials are set at observed values because these seem reasonable. Rural freeways, minor arterials, and collectors are set at 60 mph, 40 mph, and 40 mph, respectively, based on judgement.

CONCLUSION

The Lincoln Transportation Studies project was to improve the travel demand forecasting model with specific planning purposes in mind. To that end, the GIS interface, model improvements, speed, and travel time study components of the project were all designed to address these specific planning purposes. The interface with GIS provides superior display capabilities and more accurate data processing for the travel model and facilitates the use of these data from one system to the other. The land use data used in the model were always derived from the GIS land-based parcel database, but the roadway system was improved by using the more accurate estimates of distance contained in the GIS.

The travel model improvements were designed to use more current travel characteristics data (from the 1995 NPTS household survey) and to provide more detail in terms of trip purpose, area type, and functional classification for improved accuracy of the results. This paper compares the results of these model improvements compared to observed data sources and the previous 1995 Lincoln Model to demonstrate the improved performance of the model. The model was subsequently used for the S1-S2 Subarea Transportation Study (5), the North 84th Street Subarea Study (6), and the South 84th Street Subarea Study (yet to be published), which confirmed the reasonableness and reliability of the model improvements.

Finally, the usefulness of the model has improved with the validation of speeds and travel times and the use of observed operational speeds as input to the travel model. Transportation alternatives can be assessed in real-world terms (minutes of travel time or speed) rather than traffic engineer lingo (level-of-service categories) to evaluate and select transportation projects. In addition, transportation planners can monitor and update these data with ongoing before and after studies for implemented projects.

REFERENCES

GIS Based Model Interfacing: Incorporating Existing Software and New Techniques into a Streamlined Interface Package

Jerry K. Shadewald

The ability to visualize data has grown immensely as both computing speed and Geographic Information Systems (GIS) functionality have increased. Now, with traditional modeling software and GIS, planners are able to visualize future traffic trends in their jurisdiction. With the creation of a streamlined interfacing program that seamlessly connects the modeling software and the GIS package, planners can spend less time computing and more time assessing needs. The interface also provides analytical tools to assist the user in validation and assessment of the traffic model, all of which are executed in a GIS environment. Tools such as the shortest path through the network, time radius from a zone or node, traffic origins and destinations from a select link, and screenline validation have all been completely automated. Through the use of pull-down menus and mouse clicks, activities that were previously time consuming events have become streamlined computer tasks, taking only a fraction of the original time. Key words: Geographic Information System, transportation modeling, analysis tools.

INTRODUCTION AND OVERVIEW

The ability to visualize data has grown immensely as both computing speed and Geographic Information Systems (GIS) functionality have increased. Now, with traditional modeling software and GIS, planners are able to visualize future traffic trends in their jurisdiction. With the creation of a streamlined interfacing program that seamlessly connects the modeling software and the GIS package, planners can spend less time computing and more time assessing needs.

The four-step transportation modeling process that exists today—trip generation, trip distribution, mode split, and traffic assignment—originated in the 1950s and has remained relatively unchanged since. During the 1960s and 1970s, main frame computers were utilized with programs such as UTPS and PlanPack to reduce the amount of hand calculations required to complete a model run. These innovations allowed the larger agencies responsible for transportation modeling to computerize the four-step modeling process. However, the cost to own and operate a main frame computer was often too high to allow smaller planning agencies to use these methods.

By the 1980s, personal computers had become both faster and cheaper than many main frame computers. Through the use of a transportation modeling package known as TranPlan, many smaller planning agencies were utilizing personal computers to run the four-step modeling process. Although TranPlan could quickly give output, this output was only in text format that required analyzing pages of printed material. Despite the effort required to analyze a network after it had been modeled, the benefit of having the modeling process automated was still enormous.

During the 1990s, visualization of data became much more prevalent. Through the use of spatially-oriented GIS packages, network data could be displayed visually. This new tool offered users the ability to analyze their modeled network data in a spatial or geographically correct format. The ability to look at data associated with a particular attribute could now be done through visual analysis rather than through the use of the printed text files. Although the GIS tool provided many new benefits, transferring data from TranPlan to the GIS was still a cumbersome process.

Although software packages that incorporated both GIS functionality and transportation modeling techniques have been created (TransCAD), many modelers have remained TranPlan users. Due to state-wide licensing agreements, TranPlan is available free of charge from some state Department of Transportation offices. The acquisition of the ArcView GIS package by modelers has been required due to a federal program known as Taz-up. With these two software packages available, an interface could be implemented to allow users to automate the data transferal process.

The first interfaces were DOS programs written in FORTRAN. These interfaces automated many of the data transfer processes, however, the robustness of these data transfer programs was often very minimal at best. The required inputs to the programs were very strictly formatted and often confusing. Input errors by the user would cause the interface to crash, forcing the process to be reexecuted.

Through the incorporation of Visual Basic and Avenue scripting, the GIS interface assists the transfer of data between TranPlan and ArcView. By using pull-down menus and mouse clicks, additional time savings have been accomplished by eliminating manually time consuming activities with streamlined computer tasks. Context sensitive help screens explain what input is required, while format-verifying functions notify the user where errors are occurring. The interface also provides analytical tools to assist the user in validation and assessment of the traffic model, all of which are displayed in a GIS environment. Tools such as the shortest path through the network, time radius from a zone or node, traffic origins and destinations from a select link, and screenline validation have all been completely automated.

This paper serves a three-fold purpose. The first section provides a sample of the GIS-based interface environment. Section two includes a list of calibration techniques and analysis tools, which have been implemented in the interface programming. These prescribed...
techniques should be quite useful, regardless of the GIS environment in which they are performed. Finally, section three provides a discussion on the advantages and disadvantages of the interface versus conventional modeling practices.

**A SAMPLE OF THE GIS BASED INTERFACE ENVIRONMENT**

As with traditional modeling processes, there is a large amount of data that needs to be accumulated before the traffic modeling process can begin. In this regard, the GIS-based interface is no different. To begin with, a representation of the city needs to be created. This includes acquiring information on Traffic Analysis Zones (TAZs) and major streets in the network, along with frictional and external data for the network. The formatting of the raw data for input into the modeling software can then either be done by the GIS or externally through an assortment of means. By choosing to format the data in the interface, the entire network can be transformed into a digital network by simply drawing the streets to be modeled. Once the links and nodes data files have been created, the formatting of the production/atraction file(s) and the external station data files should also be completed. Below is a list of all required data files for successful use of the GIS interface. The formatting should be consistent with the requirements of the traffic modeling software as discussed in the software user manual.

- Node data file
- Link data file
- Production/Attraction data file (may be stored in separate files)
- External Stations data file
- Friction file
- Turn Penalty/Prohibitor file (optional)

To begin the modeling process through the GIS interface, the modeled network must be created. By using the interface to format the street data, this process has already been completed. If the interface was not used to format the data, then the link and node data must be input into the interface. Other information about the network—such as the number of TAZs and coordinate information—are also prompted at this point. Successfully running this portion of the interface will result in the creation of a GIS database that contains attributes of the links and nodes making up the network along with a visual plot of the network. Figure 1 shows the nodes and links files in GIS.

The second step in the GIS modeling interface is to “load” traffic onto the newly created network through the use of traffic modeling software. The interface creates the required control files to operate the modeling software, then remotely runs the modeling program. However, the option to use preexisting control files is also given, allowing the user to customize the modeling process. Once the traffic modeling process is completed, which indicates the four-step modeling process is complete, then the interface creates a new GIS database containing the “loaded” network attributes, and a new visual plot of the network is overlaid upon the original network. Figure 2 shows the loaded links and the TAZ centroids, along with the original nodes and links files. Note that not all of the original links are contained in the loaded links. Centroid connectors are not included as part of the loaded links file.

**CALIBRATION AND ANALYSIS TOOLS**

A set of tools to visualize the loaded network is provided to help calibrate the newly created model. By comparing the model’s loaded volumes to actual ground counts in a visual manner through the use of GIS, the user is able to distinguish loaded links that are inconsistent with their respective counted volumes. The first tool for visualizing and calibrating is the validation plot. This tool, which follows guidelines set forth in a recent report describing new calibration
techniques, calculates the ratio between the loaded volume and the counted volume, then displays the link according to this ratio. The displayed link will appear blue for a low loaded to counted ratio or red for a high ratio. The link is also buffered according to the total loaded volume as shown in Figure 3. A similar display option is the calibration plot, which displays the total loaded volume on each link as one of five colors. Figure 4 shows an example of the calibration plot.

Finally, a select link analysis tool allows the user to specify a particular link and then quickly identify the origins and destinations of all trips using the selected link. A visual plot of the number of trips either originating or terminating at all TAZs in the network is produced. Figure 6 shows a plot of the percent of trips from each TAZ that are origins or destinations.

The GIS interface also includes several other analysis tools. The shortest path finder prompts the user to select an origin and destination TAZ. A control file for the modeling software is created which will identify the optimal path according to distance through the network. The modeling software is executed, and the results are displayed visually on top of the existing GIS network. A database table showing all links that make up the shortest path is also displayed. Figure 7 shows the shortest path file along with the attributes table.

The second tool implemented by the GIS interface is a screen/cut/cordon line display. The user can create up to ten screen lines at one time in the GIS environment. Screen lines are useful in identifying coding errors that appear as drastic differences in loaded volume between parallel and similar roadways. Each newly created screen line will produce a new visual GIS plot and database of all links in the network that the screen line intersects (see Figure 5).

The origin/destination desire line tool allows the user to identify the loaded volumes that are traveling between two TAZs. This line is visually displayed and then colored according to the loaded traffic volume between the two points. Through a desire line, any cell in an origin/destination matrix can be displayed as part of the visual network (see Figure 8).
Another analysis tool contained by the GIS interface is the turning movement display. Using this tool requires that during the four-step modeling process, the turning movements were saved to a file. To display the turning movements, the user simply selects the node in the GIS network. A visual plot of all turning movements going through the selected node will then be displayed, along with a database containing the volumes from the modeling process. An example of a turning movement display is found in Figure 9.

The last analysis tool is the time radius display. By selecting a node in the GIS network and then specifying a time, the interface will create a visual plot of all links that could be reached from the selected node within the specified time. Figure 10 shows an example of a time radius display.

As computation time becomes shorter and the demands on professional time become greater, the need for an innovative way to use computers to automate time-consuming processes becomes more apparent. Through the use of a GIS interface such as the proposed package above, such automation can be achieved. Even more valuable is the ability to visually identify the network with very little effort. A GIS can quickly and efficiently display large amounts of data in a form that allows for easy debugging by the user. Another advantage of the GIS interface is that modeling can be done without an in-depth knowledge of the modeling software. Since, data formatting and naming conventions are automated by the interface, many computational errors are eliminated.

Through the use of the analysis tools, more advantages from the interface can be obtained. The user can quickly locate useful information about the network without running a new model process or searching through previous model reports. By repeatedly running analysis tools, an in-depth picture of the underlying data can be found and errors identified. Due to the nature of a GIS, all the information about the network can be stored inside of the GIS project. This is a huge benefit when errors in the input data are identified. Corrections on the data can be done in GIS, and the modeling process can quickly be repeated. The result is a very short turn-around time in creating a more accurate model.
Similar to any new product, there are some disadvantages to the GIS interface. To users who are not familiar with GIS, there is a disadvantage to learning a new software program. Although most GIS packages are relatively user-friendly, there is still a noticeable learning curve when beginning a new GIS package. Another disadvantage is that familiarity with the data is no longer needed. Conventional modeling required much more preparation of the data before inputting it into the modeling process. With automated processes, the user has less of an idea where to begin looking for errors.

The greatest disadvantage to creating an automating interface such as the proposed package above is that many users will treat it as a “black box.” Rather than carefully analyzing the progress of the modeling process, a user can point and click their way to unreliable information. Since computers cannot produce good information from bad data, it is still the user’s ultimate responsibility to verify results. Although the interface provides tools to do such verification, the user must still be aware of the what, why, and how of the modeling process.

ACKNOWLEDGEMENTS

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Using GIS in Preliminary Geotechnical Site Investigations for Transportation Projects

ROCH PLAYER

The purpose of a preliminary geotechnical site investigation is to create a model of the geotechnical conditions and considerations facing a project. The model is then used to analyze the project and to make project decisions. Geographical information systems (GIS) can be used in preliminary geotechnical site investigations to develop and analyze a site model and to plan site activities. In preliminary geotechnical site evaluations, GIS can be used in four ways: 1) data integration, 2) data visualization and analysis, 3) planning and summarizing site activities, and 4) data presentation. GIS allows integration of preexisting data sets with project-specific data such as CAD files, survey points, and site reconnaissance photos. The integrated data can be displayed, manipulated, and analyzed using tools built into the GIS program, thus creating the site model. From this site model, decisions can be made for further site activities and the results of the site activities can be integrated into the GIS site model. GIS can also be used to create maps and figures for reports, displays, and field personnel use. GIS was used in the preliminary geotechnical site investigation for the US 63/US 34 Ottumwa Bypass highway. As a result of its use, areas of potential geological hazards that could impact the design and construction of the bypass were identified quickly, and changes were made to the alignment early in the design process before significant design effort had been invested. Key words: geotechnical, geographic, information, systems, GIS.

INTRODUCTION

The purpose of a preliminary geotechnical site investigation is to create a model of the geotechnical conditions and considerations facing a project. The model is then used to analyze the project and to make project decisions. The intent of this effort is to place the project in the context of its surroundings and to identify potential barriers to project completion early in the design process. A successful preliminary investigation may result in significant cost savings in design, construction, and longevity of the project.

Transportation projects particularly need effective preliminary geotechnical site investigations because of the significant investments of private and public funds, their long design lives, and their impacts to the public. This paper focuses on how a geographical information system (GIS) can be used in a preliminary geotechnical site investigation to develop and analyze a site model and to plan site activities. The US 63/US 34 Ottumwa Bypass will be used to illustrate application of the technology and benefits of its use in soils design.

PRELIMINARY GEOTECHNICAL SITE EVALUATIONS

The purpose of a preliminary geotechnical site investigation is to develop a working site model that is used to analyze the site and to plan site activities. This model is continually refined as more information is gathered and integrated into the existing model. The model consists of demographic, hydrologic, geologic, historic, and other types of information that place the project into the context of its surroundings.

The primary benefit of the investigation is the identification of potential barriers to successful project completion early in the design process. Some potential geotechnical barriers that a transportation project could face include:

- Excessively weak soils
- Potentially unstable slopes
- Distance to and type of borrow sources
- Geologic hazards
- Environmental hazards

Early identification of these barriers can avoid costly and time-consuming changes after significant site design has been completed. It also enables the design of facilities that avoid the potential problems or incorporate solutions into the site design.

GIS AND PRELIMINARY GEOTECHNICAL SITE INVESTIGATIONS

The conventional approach to these site investigations can be an arduous task. Existing data sources are found in a variety of hard copy and paper formats such as maps, reports, books, aerial photos, etc. Integrating these data together with photos, notes, borings, and other site specific data can require a significant portion of the effort expended during the preliminary investigation. Less time may be spent on data analysis and acquisition than on data integration. Also, reproducing the work may take as much time as the initial production. Using a geographic information system (GIS) to aid preliminary geotechnical site investigations can greatly improve the efficiency and effectiveness of these investigations.

GIS has been defined as “a fundamental and universally applicable set of value-added tools for capturing, transforming, managing, analyzing, and presenting information that are geographically referenced (1).” Most data utilized in geotechnical site investigations have spatial attributes, that is, they can be located at a point in space. The power of GIS is that it can link maps and photos directly to data describing their features and allows data to be searched and analyzed spatially. Layers of data, known as “coverages,” can be readily combined to provide a wealth of information about a site and can be added or removed from a base map by turning layers on or off. This
results in a flexible site model that can be reused for multiple applica-
tions with minimal duplication of effort.

GIS was used during the preliminary geotechnical site inves-
tigation for the US 63/US 34 Ottumwa Bypass project, and this project will be used to illustrate its capabilities in this paper. ARCWV GIS (ESRI, Redlands, California) was the GIS pro-
gram used during this effort. In preliminary geotechnical site evaluations, GIS can be used in four ways:

- Data integration
- Data visualization and analysis
- Planning and summarizing site activities
- Data presentation

**Data Integration**

To develop and refine the working site model, data from various sources need to be integrated. These data may consist of readily available existing information, such as soil surveys and topo-
graphic maps, and project specific information, such as proposed centerlines, project extents, survey points, aerial photos, and site investigation results. GIS provides tools for integrating these data. Figure 1 shows a proposed alignment for the Ottumwa Bypass that was generated in a CAD program, imported into ARCWV, and combined with preexisting road, drainage, sec-
tion, and topographic information. To this base map, additional data, such as former coal mine locations, can be added easily as shown in Figure 2. Photos can also be layered with maps to show details.

When integrating data from various sources, two important considerations are data limitations and project coordinate sys-
tems. Each data set has inherent limitations. The source of the data must be considered, positional accuracy may vary from tenths to hundreds of meters, and the applicability of the data to their intended use also needs to be considered. The site model is only as accurate as its components. In some cases, the data accuracy may be inadequate for detailed design, however it may be more than adequate for preliminary investigations.

For disparate data sets to be integrated, each must have the same base coordinate system. Readily available data sets may utilize a coordinate system such as the Universal Transverse Mercator (UTM) with varying datums. Project specific data sets may use a standard coordinate system or a project specific system. Most GIS programs contain routines for performing coordinate transformations relatively simply to enable integration of data sets in different coordinate sys-
tems.

The Ottumwa Bypass GIS model utilized two coordinate sys-
tems. For integrating preexisting data sets such as soil surveys, hydrology, and roads with the road design produced CAD files, the UTM 1927 datum coordinate system was used. When greater positional accuracy was needed for planning site activities and integrating project specific data sets, the project specific coordinate system used by the road designers and survey crews was used.

**Data Visualization and Analysis**

One of the primary purposes of a preliminary geotechnical site inves-
tigation is to identify potential barriers to successful project comple-
tion early in the design process. Using GIS to visualize and analyze site data can expedite this process, as shown in the Ottumwa Bypass preliminary investigation.

The proposed Ottumwa Bypass corridor is located in an area of actively eroding slopes and steep drainage ways feeding into the Des Moines River in Southeastern Iowa. This area also was extensively mined for coal from the late 1800s to the middle of the 20th century. As a result, potentially unstable slopes and former coal mines were concerns as the roadway alignment was being refined. A site model was created using GIS to identify areas of concern in relationship to the alignment and their potential im-

![FIGURE 1 Preexisting data sets (roads, drainage, section lines) integrated with project specific data sets (proposed alignment)](image1)

![FIGURE 2 Coal mine locations along proposed alignment](image2)
FIGURE 3 GIS used to create 800-meter radius buffer around proposed alignment to identify mines near alignment

added to the base map and model was queried for slopes in excess of 20%—the highlighted locations in Figure 5. These slopes were found along the alignment north and south of Bladensburg Road, within the footprint of the proposed road.

Planning and Summarizing Site Activities

After identifying potential problem areas in the office, the next step in the geotechnical site investigation is to field verify assumptions and perform site reconnaissance to collect more information. GIS can be used for both planning site activities and to integrate data collected during these activities into the site model, thus further refining it.

GIS was used for the Ottumwa Bypass investigation to plan the field investigation of both the Chet Akers Coal Mine and the potentially unstable area north of Bladensburg Road. In the case of the mine, a 1930s era mine map was obtained and digitized, using project specific coordinates. Figure 6 shows the mine extent, determined during a literature review superimposed on an aerial photo in GIS. Since this location was approximate, further investigation was needed to identify features associated with coal mining activities. The GIS program was used to plan the locations to be visited during the field investigation. Maps and directions for the field personnel were produced within the GIS program, based on the site model previously created.

Features such as tailing piles, rock outcroppings, sinkholes, and abandoned equipment were noted, and photos and survey shots of these were taken. This information gathered in the field was incorporated into the GIS model in the office, and Figure 6 shows features related to coal mining discovered during the field investigation. The locations of photos taken in the field were stored as a point theme in the GIS model, and each point was linked to an electronically stored copy of the photo. These photos can be retrieved and displayed by clicking points on the screen.
Data Presentation

Another benefit of using GIS is data presentation. Layouts can be created for use in reports, papers, posters, and presentations in varying page sizes and formats. Labels, symbols, scale bars, north arrows, and text can be added to maps to provide clarity and improve information transfer. The figures used in this paper were created in the GIS program and exported in a graphic format.

CONCLUSION

GIS is a versatile tool that can be used to aid preliminary geotechnical site evaluations. It was used on the Ottumwa Bypass to identify locations of potential stability problems and possible geologic hazards due to past coal mining. It was used to guide field activities and to merge field data with existing information. This provided an accurate, flexible site model that allowed improved site analysis. As a result, the roadway alignment was shifted away from potential barriers to successful project completion early in the design process, before substantial design effort had been invested.

ACKNOWLEDGEMENTS

The author would like to thank the Iowa Department of Transportation Soils Design Section and CH2MHILL for the opportunity to do this work.

REFERENCES

Capacity of Freeway Work Zone Lane Closures

T. H. Maze, Steve D. Schrock, and Ali Kamyab

INTRODUCTION

The Iowa Department of Transportation (DOT), like many other state transportation agencies, is experiencing growing congestion and traffic delays in work zones on rural interstate highways. The congestion has resulted from unprecedented growth in traffic on rural segments of Iowa interstates. Traffic volumes have reached levels that are unlike those experienced in the past. The congestion on rural interstates is particularly problematic because in rural areas there are few, if any, parallel diversion routes and through traffic. Traveling long distances, may be relatively unfamiliar with local conditions and alternative routes. In addition, drivers are generally unaware of the work zone and do not expect heavy congestion in rural Iowa.

The congestion results in unproductive and wasteful delays for both motorists and commercial vehicles. It also results in hazardous conditions where vehicles, stopped in queues on rural interstate highways, are being approached by vehicles upstream at very high speeds. The delay also results in driver frustration, making some drivers willing to take unsafe risks in an effort to bypass delays. To reduce the safety hazards and unproductive delays of congested rural interstate work zones, the Iowa DOT would like to improve its traffic management strategies at these locations in the future.

During the summer of 1998 the Center for Transportation Research and Education (CTRE) at Iowa State University observed a work zone on a rural Iowa interstate highway to measure the volume of vehicles that can pass through a work zone lane closure prior to and during congested operations and to better understand related driver behaviors. One unique aspect of the research we conducted at this lane closure was to observe the rate at which the queue grows (more cars joining the end of the queue than leaving the front of the queue) and the rate at which the queue declines in length. It was found that the queue grows and declines in surges. When it grows, the queue moves backward, and when it shrinks it moves forward. Backward-moving queues grow at rates as high as 30 to 40 miles per hour. This means that as a vehicle approaches the end of the queue at normal highway speeds, for example, 65 miles per hour, a backward-moving queue could be moving toward them, for example, at 35 miles per hour. This results in the end of the queue approaching a vehicle at 100-miles per hour which violates the expectations of the driver and creates a relatively unsafe condition.

This paper reports on part of the research that was done by CTRE for the Iowa DOT to evaluate the capacity of lane closures and driver behavior. A companion paper describes a simulation model that was developed to analyze the driver behavior and emulate the benefits of advanced traffic control at work zone lane closures (1).

PRIOR MEASURES OF CAPACITY AT LANE CLOSURES

Most highway agencies simply use the methods described in the Highway Capacity Manual to determine the capacity of a lane closure at an interstate work zone (2). The capacity estimates in the Highway Capacity Manual are based on the work done at the Texas Transportation Institute (TTI) by a variety of investigators over a number of years from the late 1970s and the mid-1980s. This work is based on data collected as part of the Texas Department of Transportation’s “Study 292.”

Queue and User Cost Evaluation of Work Zones (QUEWZ) is a software package used by many state transportation agencies to determine estimated delays, the length of queues, and user costs due to work zone lane closures. QUEWZ also originated from the same research program conducted at TTI. Later (1987-1991), field data collections were conducted by TTI to update the capacity values and to revise and improve QUEWZ (3). One of the more significant impacts of the updates was to change the factor for equating heavy trucks to passenger cars from 1.7 to 1.5. More recently, two studies have been done in North Carolina and in Indiana to try to determine the capacity of lane closures on interstate highways in those states.

Before investigating prior estimates of capacity, it should be recognized that not all estimates of capacity at lane closures are measured using the same criteria. The work done by TTI defined capacity as the hourly traffic volume under congested traffic conditions (4). The TTI researchers identified capacity as full-hour volumes counted at lane closures with traffic queued upstream. They considered consecutive hours at the same location as independent studies. A Pennsylvania study defined the hourly traffic volume converted from the maximum-recorded five-minute flow rate as the work zone capacity. A California study measured volumes for three-minute time intervals during congested conditions. Two, three-minute time intervals, separated by one minute, were then averaged and multiplied by 20 to determine the one-hour capacity values (5). All of these studies considered the flow passing through a lane closure under congested conditions to be the capacity.

Dixon and Hummer define work zone capacity as the flow rate at which traffic behavior quickly changes from uncongested conditions to queued conditions (6). Jiang defines capacity as the flow just
before a sharp speed drop followed by a sustained period of low vehicle speeds and fluctuating traffic flow which defines the formation of a queue. It is Jiang’s contention that what TTI was measuring was not capacity of the bottleneck but rather the queue discharge rate (7).

TTI work zone capacity research published in 1982 was used as a basis for the methods for determining work zone capacity as described in the 1985 Highway Capacity Manual (as well as the 1994 manual). This work was based on hour-long data collected on urban Texas freeways with lane closures. The applications of these data may be difficult to extrapolate to other locales due to the differences in driver behavior and differences in design of urban Texas interstate highways. Texas makes extensive use of frontage roads, making it much easier for motorists to bypass congested segments of highway.

The work conducted by TTI as part of Study 292 and work by other institutions and other individuals have resulted in a wealth of literature reporting on the measurement of queue discharge rates at work zones under a variety of factors which impact capacity. For example, one study investigated the sensitivity of capacities to the use of shoulders during lane closures and to splitting traffic when a center lane is closed (8,9). Some have looked at the type of traffic control devices and their placement and how they impact capacity and delay. Others have investigated pavement conditions, night versus day, traffic volumes and traffic composition, merge discipline and speed control strategies, and the duration of work zones (short-term versus long-term) (8,10,11). Still others have investigated the relationship of the location of construction work to the traffic lane (6,8,12).

The work Dixon and Hummer completed on capacities and delays at work zones conducted for the North Carolina Department of Transportation in 1996 probably provides the most significant inference for Iowa. The North Carolina study included field data collected under conditions similar to those of interest to Iowa: lane closures on two-lane rural interstate highways. The North Carolina study used a more relevant measure of capacity for a lane closure than the TTI researchers. The North Carolina researchers defined capacity as the traffic volume immediately before queuing begins.

An important and unique finding of the North Carolina study is the identification of the location within a work zone that governs maximum traffic flow through the work zone. The location tends to vary with traffic conditions and with construction work activities. The work done by TTI has assumed that the feature governing the maximum traffic flow is the point at the end of the taper. Dixon and Hummer report that the maximum flow is governed by three locations. The segment of the work zone travel path adjacent to the work area controls the maximum flow where the construction work activity is heavy, meaning large equipment and workers adjacent to the travel path. Under conditions where the work activity is low, then the maximum flow (prior to queue formation) is governed by the end of the merge taper. However, when the work activity is heavy, the maximum traffic flow was found to be about seven percent less than the maximum flow at the taper end for work zones on two-lane rural interstate highways. When a queue has formed, the maximum flow is governed by the merging activity upstream from the work zone. In other words, once a queue has been formed, the maximum flow of the entire work zone is governed by the rate at which traffic can be discharged from the queue, which is generally at a lower rate than the capacity of the taper end, accounting for capacity drop when a queue is formed.

Shown in Figure 1 is the traditional flow-speed relationship where the maximum flow \(Q_m\) is roughly half the free flow speed. This symmetrical relationship was reflected in flow-speed relationships identified in the Highway Capacity Manual until the 1990 interim edition. Starting with the 1990 interim manual, the top half of the curve was shown to be more flat, and the maximum flow is reached when speed declines by 14 percent rather than 50 percent. Figure 2 shows a more realistic representation of the flow-speed relationship with three distinct portions of the relationship. The top half of the curve represents flow under uncongested conditions. The bottom portion of the curve represents flow during congestion. The reduction in maximum flow when traffic operation drops from uncongested to congested is the capacity drop. In other words, immediately before flow breakdown occurs, the flow rate is greater than after a flow breakdown. Therefore, when the queuing condition is reached, a capacity reduction (or drop) occurs \((13,14)\). The capacity drop is due to turbulence in the traffic flow that results after a breakdown.

We did not observe a capacity drop in the data we collected at an Iowa work zone. However, in similar studies in North Carolina and Indiana, a significant capacity drop was observed \((7,15)\). The capacity drop illustrates the importance of not allowing the traffic operations at a work zone lane closure to decline from uncongested to congested.

**FIELD DATA COLLECTION**

The site selected for data collection during the summer of 1998 is located on Interstate Highway 80 between U.S. 61 and Interstate Highway 74. Data were collected to determine the following:
1. Traffic flow characteristics (speed, density, and volume) at the end of the lane closure taper.
2. Traffic flow characteristics upstream from the lane closure (500 feet or 152 meters upstream of the taper).
3. The length of the queue when congestion occurs. This is a measure of storage and the difference in queue length from one time to the next is the speed that the queue grows or is discharged.

Two trailers with 30-foot (9.14 meters) booms and two cameras on top of the booms were used to collect video. The video images were processed to derive the traffic flow data. A picture of one of the trailers is shown in Figure 3. A schematic of a typical data collection layout is shown in Figure 4.

The data collection trailers were positioned at the site for 19 days during the summer of 1998. Congestion was observed on only four days. Shown in Figure 5 is a plot of the data collected on July 2, 1998. The upper plot line shows the flow values in passenger car equivalents, summarized in 15-minute intervals. The lower line is the average speed over the same 15-minute interval. When queuing conditions exist (starting at 15:00) the average speed drops precipitously while the volume stays nearly constant before and after queuing. In other words, we did not observe a capacity drop.

To determine the maximum capacity of the lane closure, we took the average volume of the ten highest volumes immediately before and after queuing conditions. The lane closure capacities observed are listed in Table 1. Shown are both the highest and an average of the ten highest volumes to pass through the lane closure during an uncongested 15-minute period. Also, separate columns are shown for flows in vehicles and in passenger car equivalents.

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</table>

The length of the queues for these dates was monitored, with the length to the nearest 0.05 miles recorded every minute. To accomplish this, the project team drove a vehicle on the opposite shoulder of the interstate in the direction of the westbound traffic. The team kept the vehicle even with the upstream end of the eastbound queue, and recorded the milepost readings from the delineator posts in the ditch. The lengths of the queues over time are shown for one day in Figure 6 for data collected on August 6, 1998. In Figure 6, the change in the length of the queue over the one-minute data collection interval was an indication of the speed with which the queue grows or dissipates. The speed of change in queue length is shown in Figure 7. Even averaged over an entire minute, speeds were recorded in excess of 30-miles per hour.

CONCLUSION

We found, through limited data collection, that capacities in rural Iowa work zone lane closures varied from roughly 1,400 passenger car equivalents to 1,600. We also found that through queues can move backward and forward at very swift rates, meaning that queuing vehicles at lane closures presents a very serious safety condition. However, the data collected by automated traffic recording devices along the I-80 corridor also show very consistent day of the
week and time of day repeating patterns in traffic volumes. This means that given the likely traffic volume and measure of capacity, we can begin to contrast the historical volumes and begin to predict when congestion is going to occur and apply traffic management strategies to mitigate congestion. Possible strategies might include identifying diversion routes and informing drivers well in advance so they may select alternative routes or alternative times to travel. This information may be provided to drivers through a variety of possible traveler information systems.

ACKNOWLEDGMENTS

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Evaluation of Rural Interstate Work Zone Traffic Management Plans in Iowa Using Simulation

STEVEN D. SCHROCK AND T. H. MAZE

Traffic levels on Interstate Highway 80 in eastern Iowa increased 44 percent between 1988 and 1997, with summer traffic volumes approaching 40,000 vehicles per day. These traffic levels and the expected continued growth create special problems when developing a work zone traffic management plan (WZTMP) at a long-term work zone. Motorists at past work zones on this highway have experienced long delays. A methodology for evaluating the cost-effectiveness of alternative delay-reducing WZTMPs was developed and tested on a past work zone to determine its effectiveness. Using Traffic Software Integrated System (TSIS) 4.2, four WZTMPs were modeled to evaluate their effectiveness at reducing motorist delay. These models were based on a case study of a long-term work zone on Interstate 80 in 1997, where queues developed on 34 different days from May 31, 1997 to September 13, 1997. Of the competing alternatives, the most cost-effective alternative is to direct the contractor to implement a nonstop work schedule until project completion. The results for this alternative indicated that about 9,044,000 vehicle-minutes of delay could have been avoided. This represents an 86% reduction in delay reduction compared with the “do-nothing” alternative. This methodology was developed as a planning tool to determine the potential benefits of alternative traffic management plans at work zones. Highway agencies using this methodology can determine the potential cost-effectiveness of alternative WZTMPs at upcoming work zones. Key words: Corsim, delay, simulation, work zone.

INTRODUCTION

By 1996, the interstate highway system was 99.9% complete (1). As the original sections of interstate highway reach the end of their serviceable lives, reconstruction and rehabilitation has become more common. During this same time, there has been rapid growth of vehicle miles traveled on the nation’s highway system. Traffic volumes on portions of Interstate Highway 80 in rural eastern Iowa, for example, increased by about 44 percent from 1988 to 1997 (2).

Several studies have indicated that capacities are significantly reduced when one or more lanes are removed due to repair or reconstruction (3,4,5). When traffic demand exceeds the reduced work zone capacity, congestion and delays result, leading to increased driver frustration (6), reduced safety (7), and increased user delay cost (8).

The quantity and cost of delay caused by a work zone must be determined in order to choose the appropriate mitigation technique. If there is too little improvement, the desired results are not achieved. Too large an investment for a more substantial alternative could reduce the traffic congestion, but would be an inefficient allocation of the finite resources available to a highway agency. Clearly, the delay imposed on the driving public must be determined in order to conduct an analysis of the available alternatives.

This research used microscopic simulation to evaluate the cost-effectiveness of using alternative work zone traffic management plans (WZTMPs) at rural interstate locations where work zones reduce the number of traffic lanes from two to one in each direction. Computer simulation provides data required for benefit-cost analyses that would be difficult to obtain from other sources.

To prove the effectiveness of this process, data from one Interstate Highway 80 work zone in Iowa County, Iowa in 1997 was analyzed. Four alternative WZTMPs were studied: 1) the “do-nothing” alternative where no changes were made, 2) accelerating the pace of the contractor to a nonstop work schedule, 3) paving additional lanes through the work zone to allow four lanes of traffic at once, and 4) diverting a small percentage of traffic onto a detour route to reduce traffic demand on the interstate.

CASE STUDY BACKGROUND

The work zone chosen for the case study was in place in 1997 on Interstate Highway 80 from mile marker 215 to 221 in Iowa County, Iowa. The project included pavement reconstruction of all four traffic lanes, for a total cost of approximately 10.8 million dollars. On May 31, 1997, interstate traffic was modified into a two-lane, two-way operation (TLTWO) and remained in this configuration until September 13, 1997.

A detour route was established by the Iowa DOT to accommodate traffic during periods of work zone congestion and to serve as a contingency route should the work zone become impassable. The diversion route consisted of using Iowa Highway 21 and US Highways 6 and 151 to bypass the work zone. Figure 1 shows Interstate Highway 80 and the detour route during this construction project. The Iowa DOT’s WZTMP did not divert motorists from the interstate and onto the detour route except in emergencies within the work zone.

S. Schrock, Texas Transportation Institute, Texas A&M University, 3091 CEE/TTI Bldg., College Station, Texas 77845. T. Maze, (formerly of Center for Transportation Research and Education), Howard R. Green Company, 4685 Merle Hay Road, Suite 100, Des Moines, Iowa 50322-1966.
Four alternative WZTMPs were prepared for analysis in the case study. These alternatives were developed with the assistance of Iowa DOT engineers, and although these were not implemented, they represent potential solutions deemed worthy of investigation at this site. Each alternative is summarized below.

**Do-Nothing Alternative**

The first alternative represents the traffic management plan that was actually used during reconstruction in 1997. For modeling purposes, the FRESIM component of TSIS was used to simulate Interstate Highway 80, from mile marker 211 to 226, with one lane blocked in each direction by an incident to simulate the work zone location. This alternative was modeled for comparison to the remaining alternatives.

**Four Traffic Lanes through the Work Zone**

The third alternative investigated provided two additional traffic lanes through the work zone. This involved strengthening and widening the shoulders along the interstate through the work zone, and widening four bridges to accommodate the additional traffic lanes. In this manner, the basic TLTWO would be modified to allow two lanes in each direction. No other modifications to the traffic control plan would be made for this alternative compared with the do-nothing alternative. The Iowa DOT estimated the cost of paving all of the shoulders to a width of 12 feet for six miles and widening the four bridges in this case study to be about $4,946,000.

**Diversion Route Alternative**

In this alternative, the Iowa DOT would divert a small percentage of vehicles off the interstate and onto the existing diversion route. This alternative was divided into three separate sub-alternatives. One sub-alternative assumed that 5% of all vehicles were diverted, the second assumed a 10% diversion, and the third assumed a 15% diversion.

The benefit of this alternative was a reduction of delay over the entire network by reducing the traffic levels on the interstate. The delay for vehicles on the diversion route, however, would increase because local non-interstate traffic would be delayed by an increase in traffic on the detour route and because the diverted interstate traffic would have a longer path than if it stayed on the interstate. In order for this alternative to be beneficial, the interstate delay savings must outweigh the detour’s delay increase.

**Nonstop Work Alternative**

The second alternative accelerated the pace of the project by requiring the contractor to work 24 hours per day, 7 days per week during the time the TLTWO traffic plan was in place. The benefit of such an alternative would be the elimination of delays between the theoretical early finish date and September 13, 1997, when the TLTWO was actually removed. No other modification to the do-nothing traffic plan was made in this alternative.

After a review of the Iowa DOT records concerning construction schedule and reported weather for this project, estimates by Iowa DOT engineers, and estimates by a Des Moines-based paving contractor, a revised schedule was created with an early finish date of July 19, 1997, 56 days ahead of the do-nothing schedule. The contractor believed that for the case study, a premium of $1,000,000 would be required for a nonstop work schedule.

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The costs associated with diverting traffic onto the detour route are not well defined. Discussions with Iowa DOT traffic engineers revealed that the state has little experience with diverting rural interstate traffic for non-emergency situations. To estimate the costs for such an alternative, several assumptions were made. First, it was assumed that additional manpower would be required to implement...
this alternative. It was assumed that 6 to 10 people would be occupied full-time during TL Two operation to run traveler information equipment, hand out fliers at rest areas, or other work to convince drivers to divert. It was assumed that the 5, 10, and 15% diversions could be successfully accomplished for $370,000, $470,000, and $670,000, respectively.

Traffic Data

The interstate traffic volumes used in this research were obtained from the Iowa DOT’s Office of Transportation Data. The traffic volumes were recorded from an automatic traffic recorder (ATR) permanently installed about 2 miles from the east end of the case study. Twenty percent of the traffic stream was assumed heavy trucks, based on a previous study of Interstate Highway 80 through eastern Iowa (9). The traffic volumes on the detour route were obtained from ATR sites on US Highway 6. This information allowed the model to include “background” traffic on the network, with which the diverted interstate traffic would interact.

Determination of Congested Days

From the Iowa DOT project files, it was determined that congestion due to high traffic volumes was reported on 34 days during the project. Table 1 shows the dates and times when the CMS and HAR units were active due to high traffic volumes. These 34 days were modeled for each of the alternative traffic management plans. Each of these days were modeled using the available traffic data starting one hour prior to the onset of congestion and terminating one hour after congestion subsided.

SIMULATION MODEL

In this research, microscopic simulation was used to determine the amount of motorist delay that would accrue using several alternative traffic management plans over the life of a work zone. Traffic Software Integrated System (TSIS), developed by the Federal Highway Administration, was chosen for this research because of its ability to simulate a work zone environment and provide the measures of effectiveness needed to effectively compare alternative traffic management plans.

The model was constructed in two parts: the interstate portion was constructed using the FRESIM portion of the TSIS software package. The interstate consisted of two freeway lanes in each direction with a speed limit of 65 mph. The work zone was created using the incident function in FRESIM. This function was originally developed to model traffic crashes or incidents. The work zone was created by blocking the left lane of the interstate in each direction with an incident the length of the work zone and with duration as long as the simulation run time. Highways of the detour network were created using the NETSIM portion of the TSIS software.

Model Calibration

The work zone portion of the model was calibrated to achieve the proper traffic behavior by changing the headway between vehicles at the incident location. The headway between vehicles can be increased in TSIS by from 0 to 100% using the incident function. To determine the appropriate headway adjustment factor for this research, a two-lane interstate test section was simulated, consisting of 6,000 feet of two-lane freeway and representing one direction of a rural interstate. A work zone was placed in the center of the model, which closed one lane. The speed limit was established as 55 mph through the work zone and 65 mph for the remainder. The work zone headway was increased by 20% for the first iteration. Traffic volumes were input into the traffic generator at the upstream end of the model, starting at a rate of 1,000 vehicles per hour (vph) with 20% trucks. This volume was increased by small increments to 1,500 vph at regular intervals. This test was repeated with the work zone headway increased to 30%, 40%, 50%, and 60%. The results were

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<tr>
<td>7/25/97</td>
<td>15:00</td>
<td>21:30</td>
</tr>
<tr>
<td>7/26/97</td>
<td>11:30</td>
<td>15:00</td>
</tr>
<tr>
<td>7/27/97</td>
<td>12:30</td>
<td>21:15</td>
</tr>
<tr>
<td>8/1/97</td>
<td>9:35</td>
<td>01:30 (8/2)</td>
</tr>
<tr>
<td>8/2/97</td>
<td>9:00</td>
<td>21:30</td>
</tr>
<tr>
<td>8/3/97</td>
<td>11:10</td>
<td>22:04</td>
</tr>
<tr>
<td>8/7/97</td>
<td>11:00</td>
<td>15:00</td>
</tr>
<tr>
<td>8/8/97</td>
<td>10:35</td>
<td>00:25 (8/9)</td>
</tr>
<tr>
<td>8/9/97</td>
<td>9:05</td>
<td>20:20</td>
</tr>
<tr>
<td>8/10/97</td>
<td>2:00</td>
<td>22:50</td>
</tr>
<tr>
<td>8/15/97</td>
<td>11:30</td>
<td>02:50 (8/16)</td>
</tr>
<tr>
<td>8/16/97</td>
<td>10:15</td>
<td>17:30</td>
</tr>
<tr>
<td>8/17/97</td>
<td>11:55</td>
<td>22:30</td>
</tr>
<tr>
<td>8/24/97</td>
<td>16:15</td>
<td>22:30</td>
</tr>
<tr>
<td>8/29/97</td>
<td>12:00</td>
<td>21:25</td>
</tr>
<tr>
<td>8/30/97</td>
<td>11:10</td>
<td>15:30</td>
</tr>
<tr>
<td>9/1/97</td>
<td>12:50</td>
<td>21:30</td>
</tr>
<tr>
<td>9/6/97</td>
<td>10:20</td>
<td>20:50</td>
</tr>
</tbody>
</table>
then analyzed to determine when delays began for each percentage of headway increase. Figure 2 shows the results of the calibration process.

A 40% increase in work zone headway showed an increase in vehicle delay beginning at about 1,250 vph. This work zone capacity is similar to that observed by Dudek and Richards for Texas work zones in of this type (4). Additionally, Iowa State University’s Center for Transportation Research and Education (CTRE) studied the capacity of rural Iowa interstate work zones in the summer of 1998 and found a capacity of between 1,216 to 1,302 vph for a rural Iowa work zone (9). The 40% headway increase was determined to be the most reasonable value to calibrate the model and was used throughout the research.

CASE STUDY ANALYSIS

Each alternative simulation was executed using the CORSIM software. Simulation runs were performed to model the 34 congested days for the 6 different alternatives. Five simulation runs of each day for each alternative were completed using different random seed numbers for each simulation run. A statistical summary of the total delay was performed to show the average total delay, standard deviation, maximum value, minimum value, and range for each alternative. Table 2 summarizes the case study descriptive statistics.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Average Delay</th>
<th>Standard Deviation</th>
<th>Min. Value</th>
<th>Max. Value</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Do-Nothing</td>
<td>10,473</td>
<td>249</td>
<td>10,030</td>
<td>10,620</td>
<td>590</td>
</tr>
<tr>
<td>Nonstop Work</td>
<td>1,428</td>
<td>11</td>
<td>1,416</td>
<td>1,439</td>
<td>23</td>
</tr>
<tr>
<td>4-Lane Work Zone</td>
<td>1,177</td>
<td>8</td>
<td>1,167</td>
<td>1,187</td>
<td>20</td>
</tr>
<tr>
<td>5% Diversion</td>
<td>7,092</td>
<td>18</td>
<td>7,071</td>
<td>7,111</td>
<td>40</td>
</tr>
<tr>
<td>10% Diversion</td>
<td>4,589</td>
<td>290</td>
<td>4,426</td>
<td>5,107</td>
<td>681</td>
</tr>
<tr>
<td>15% Diversion</td>
<td>3,029</td>
<td>149</td>
<td>2,913</td>
<td>3,218</td>
<td>304</td>
</tr>
</tbody>
</table>

Benefit-Cost Analysis

The average delay for each alternative for each day was determined in order to calculate their value. Using a previously prepared study on the value of time for the Iowa DOT (8) and converting these to 1997 values (10), a dollar value was placed on the delay for each alternative. The values of delay for automobiles are shown in Table 3. This value varies depending on the length of delay, as laid out in the AASHTO’s Manual on User Benefit Analysis of Highway and Bus-Transit Improvements (11). The incremental cost as well as the incremental benefit for each alternative is shown in Table 4.

<table>
<thead>
<tr>
<th>Time Savings Increments</th>
<th>Average Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 5</td>
<td>$5.37</td>
</tr>
<tr>
<td>5 to 10</td>
<td>$8.06</td>
</tr>
<tr>
<td>11 or more</td>
<td>$10.74</td>
</tr>
</tbody>
</table>

| Incremental Benefit | $0 | $480 | $909 | $1,123 | $2,034 | $2,026 |
| Incremental Cost    | $0 | $370 | $470 | $670   | $1,000 | $4,947 |

After determining the benefits and the costs of each of each of the competing alternatives, the incremental benefit-cost analysis was then performed. The results of the incremental benefit-cost analysis are presented in Table 5. The nonstop work alternative had an incremental benefit-cost analysis of 2.03 when compared to the do-nothing alternative and remained the most cost-effective compared to the competing alternatives.

CONCLUSIONS

This research indicates that there are cost-effective measures that can be implemented to reduce rural interstate work zone delays. While the results of this analysis should not be construed to mean that requiring contractors to work nonstop would be the best alternative in all work zone situations, it does indicate that cost effective congestion reducing measures can be found using simulation.

Additionally, research showed that microscopic simulation is an effective tool in determining appropriate WZTMs at interstate work zones. Determining the delay values for competing WZTMs for upcoming projects would allow highway agencies to have quantifiable estimates of what these delays will cost and provide them with the information needed to choose the most appropriate alternative for a given situation.
## TABLE 5 Incremental Benefit-Cost Analysis Results

<table>
<thead>
<tr>
<th>Step 1</th>
<th>Defender</th>
<th>5% Diversion</th>
<th>Challenging Alternatives</th>
</tr>
</thead>
<tbody>
<tr>
<td>Incremental Benefit</td>
<td>Do-Nothing</td>
<td>N/A</td>
<td>10% Diversion</td>
</tr>
<tr>
<td>Incremental Cost</td>
<td>N/A</td>
<td>$480,000</td>
<td>$909,000</td>
</tr>
<tr>
<td>B/C Ratio</td>
<td>1.30</td>
<td>1.93</td>
<td>1.68</td>
</tr>
<tr>
<td>Outcome</td>
<td>Becomes New Defender</td>
<td>Remains a Challenger</td>
<td>Remains a Challenger</td>
</tr>
<tr>
<td>Step 2</td>
<td>5% Diversion</td>
<td>10% Diversion</td>
<td>15% Diversion</td>
</tr>
<tr>
<td>Incremental Benefit</td>
<td>N/A</td>
<td>$429,000</td>
<td>$643,000</td>
</tr>
<tr>
<td>Incremental Cost</td>
<td>N/A</td>
<td>$100,000</td>
<td>$300,000</td>
</tr>
<tr>
<td>B/C Ratio</td>
<td>4.28</td>
<td>2.14</td>
<td>2.47</td>
</tr>
<tr>
<td>Outcome</td>
<td>Becomes New Defender</td>
<td>Remains a Challenger</td>
<td>Remains a Challenger</td>
</tr>
<tr>
<td>Step 3</td>
<td>10% Diversion</td>
<td>15% Diversion</td>
<td>Non-Stop Work</td>
</tr>
<tr>
<td>Incremental Benefit</td>
<td>N/A</td>
<td>$214,000</td>
<td>$1,125,000</td>
</tr>
<tr>
<td>Incremental Cost</td>
<td>N/A</td>
<td>$200,000</td>
<td>$530,000</td>
</tr>
<tr>
<td>B/C Ratio</td>
<td>1.07</td>
<td>2.12</td>
<td></td>
</tr>
<tr>
<td>Outcome</td>
<td>Becomes New Defender</td>
<td>Remains a Challenger</td>
<td></td>
</tr>
<tr>
<td>Step 4</td>
<td>15% Diversion</td>
<td>Non-Stop Work</td>
<td></td>
</tr>
<tr>
<td>Incremental Benefit</td>
<td>N/A</td>
<td>$911,000</td>
<td></td>
</tr>
<tr>
<td>Incremental Cost</td>
<td>N/A</td>
<td>$330,000</td>
<td></td>
</tr>
<tr>
<td>B/C Ratio</td>
<td>2.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outcome</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The non-stop work alternative is the most attractive alternative.

Finally, the methodology used in this research could be used to predict congestion at future interstate work zones, allowing highway agencies to determine the appropriate strategy to minimize congestion cost-effectively. Only two modifications would need to be made in the methodology shown in this research to apply the process to future work zones. First, future traffic volumes would need to be estimated by inflating current data by a reasonable growth factor. Second, without knowing in advance the congested days, a reasonable assumption of when congestion is likely—such as time-of-day and day-of-week—would need to be determined based on past experience on the highway in question.

## REFERENCES

Evaluation of Speed Reduction Techniques at Work Zones

Alireza Kamyab, T.H. Maze, Stephen Gent, and Christopher Poole

The goal of the Midwest States Smart Work Zone Deployment Initiative (MwSWZDI) is to develop better ways of controlling traffic through work zones, which improves traffic safety and traffic operating efficiency of work zones. To achieve this goal, the program is currently evaluating 20 different traffic control and traffic management strategies. This paper describes the evaluation of three traffic control and traffic management strategies that involve ITS technologies. To achieve this goal, the program is currently evaluating 20 different traffic control and traffic management strategies that involve ITS technologies. In summary, the Wizard CB Alert System broadcasts a CB message warning motorists monitoring the CB of an approaching work zone. The Safety Warning System transmits a message to vehicles with Safety Warning receivers, informing those motorists of the approaching work zone. The Safety Warning System also acts like a drone radar system, alerting vehicles equipped with radar detectors, making drivers believe that radar-equipped enforcement officials may be present. The Speed Display Monitor uses radar to detect the speed of passing vehicles and displays their speed on a two-character variable message sign. Because the device uses radar, it also acts like drone radar and alerts vehicles equipped with radar detectors.

Each of these systems is evaluated in a freeway work zone environment. The Wizard CB Alert System is evaluated in a moving work zone on a rural interstate highway, while the other two systems are tested at a long-term lane closure at a rural interstate highway reconstruction location.

WIZARD CB ALERT SYSTEM

The Wizard CB Alert System is used in conjunction with a work project performed by an Iowa DOT striping crew on Interstate Highway 35. The purpose of this field test was to examine whether the Wizard CB assists in advance warning of a lane closure for truck operators in particular.
Evaluation Case Study

The painting crew consists of four to five vehicles spread out over approximately one mile and traveling at 25 miles per hour (mph). The lead vehicle is the stripping truck and the trailing vehicle is a pickup truck that carries a flashing sign which read “CENTERLINE / EDGELINE PAINTING AHEAD.” The Wizard unit was placed in the trailing vehicle in order to give sufficient warning to the paint crew of approaching vehicles.

The Wizard was set to broadcast over CB channel 19, the most commonly used frequency by truck drivers. The 30-second interval between broadcasts was chosen to insure that approaching truckers heard the message at least once.

Evaluation Operations

Two people collected data for this project. One person stayed with the Wizard in the trailing vehicle of the interstate paint crew. This person monitored the CB and recorded truckers’ responses to the warning message. The second person was stationed beyond the paint crew’s work site at the next interstate rest area to interview truckers who stopped there.

A number of different broadcast messages were utilized and tested, however, the message that offered the most positive response was: “This is an Iowa DOT road work alert. Northbound drivers on interstate 35: you are approaching a slow-moving paint crew in the right lane. Please use caution.”

This message presented all pertinent information clearly and concisely. Also, the message would only need to be changed when and if the roadwork changed direction or roadways.

Evaluation

We were unable to develop any quantitative evaluation criteria and we used only subjective measures to evaluate the effectiveness of the device. We monitored the CB broadcasts from vehicle operators in the area and their comments were overwhelmingly positive (a complete list of comments recorded is listed in the evaluation report). Typical comments from commercial operators via their CB radios are:

· “I think all states should get on the CB to warn you about this stuff.”
· “This is the first time I’ve ever heard anything like this. I wish every state would do it. It’d make things a lot easier.”

Driver Interviews at Rest Areas

Over the course of six days, truck drivers at interstate rest areas completed a total of 94 surveys. Of the drivers surveyed, 88 (94 percent) had a CB radio in their truck. Of those, 70 (80 percent) had their radios tuned to channel 19 during the preceding hours.

Of the 70 truckers that were listening to channel 19, 59 of them (84 percent) saw the paint crew on the interstate. This made a total of 59 truckers out of 94 (63 percent) that had their CB tuned to channel 19 as they passed the paint crew on the interstate. Table 1 shows how these 59 drivers answered when asked what first alerted them to the presence of the paint crew.

<table>
<thead>
<tr>
<th>Method</th>
<th>Number of Responses</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB Alert Message</td>
<td>24</td>
<td>(40%)</td>
</tr>
<tr>
<td>Lights on Trucks</td>
<td>14</td>
<td>(24%)</td>
</tr>
<tr>
<td>Signs</td>
<td>10</td>
<td>(17%)</td>
</tr>
<tr>
<td>Arrow Board</td>
<td>7</td>
<td>(12%)</td>
</tr>
<tr>
<td>Other Truck Drivers</td>
<td>4</td>
<td>(7%)</td>
</tr>
<tr>
<td>Total</td>
<td>59</td>
<td>(100%)</td>
</tr>
</tbody>
</table>

SAFETY WARNING SYSTEM

The Radio Association Defending Airwave Rights Inc. (RADAR) conceived and developed the concept of the Safety Warning System (SWS). This system consists of a transmitter and receiver (detector). MPH Industries Inc. manufactures SWS transmitters; and their device is shown in Figure 2. A number of other companies, including Bel-Tronics, Sanyo, Uniden, and Whistler also manufacture the SWS detectors.

FIGURE 2 Safety Warning System Transmitter

The transmitter can be mounted on the outside of a vehicle (e.g., inside the emergency lightbar) or placed in a stationary outdoor location (e.g., on the flashing arrow board trailer at a work zone). The SWS transmitter sends warning messages concerning road hazards to drivers of vehicles equipped with SWS detectors. Any K-band radar detector will sound a basic alarm when the SWS transmitter is sending a warning message; however, the ones capable of reading transmitted SWS messages will specifically display (in some cases, state) applicable messages.
Evaluation Case Study

The case study work zone consisted of a left lane closure with a crossover leading into two-way traffic. The SWS transmitter was mounted atop a stationary pole located 2,250 feet upstream of the lane closure taper. Traffic flow performance data were collected at 1,500 feet and 500 feet upstream of the taper using two traffic data collection trailers. One of the trailers is shown in Figure 3. The trailer includes a pneumatic mast to hoist video cameras 30 feet above the pavement’s surface where the cameras collect video of traffic operations. Videos are later reduced into traffic flow performance data through the use of image-processing technology.

We had hoped to observe a reduction in speed, a reduction in the standard deviation of speed, an increase in the number of vehicles in the 10-mph pace speed, and a reduction in the highest 15 percent of speeds. Unfortunately, no change in any of the parameters was observed.

SPEED MONITOR DISPLAY

This device detects the speed of vehicles using radar and displays the speeds of approaching vehicles using a variable message sign. Speed monitoring displays are not generally used to enforce speed limits and issue citations; rather the assumption is that motorists will drive slower once they see their excessive speed on the display. Further, the speed measuring radar will set off the radar alarms in vehicles equipped with radar detectors, resulting in driver assumption that enforcement personnel are located within the work zone, causing speeding motorists to slow.

The speed monitor display is shown in Figure 4 and consists of a large white box housing a K-band radar and two 18-inch LED characters, which are visible in direct sunlight from up to 1,000 ft away. The radar detects the approaching vehicles and shows their speeds on the LED display. The display box also has an Overspeed Option, which flashes motorists’ speeds when they exceed the speed limit. The speed threshold in this study was set to 55 mph, which was the posted regulatory speed limit of the work zone.

Data were collected two days prior to the installation of the SWS transmitter and for two days following the installation. During each day more than 2,500 data points were recorded. A number of traffic flow performance parameters were calculated from the data collected for traffic 1,500 and 500 feet upstream of the merge taper. These parameters included:

- the time mean speed
- the speed that 85 percent of the vehicles travel (the 85th percentile speed)
- the 10-mph speed interval containing the most observations (the 10-mph pace)
- the percentage of observations in the 10-mph pace
- the standard deviation of the time speed
- the percentage of observations complying with posted regulatory and advisory speed limits
- the time mean speed of the highest 15 percent of speeds

The speed monitor display used in this study included a solar power panel, which is mounted atop the box. This panel supplied power to the unit, and excess power was stored in a solar car-type battery housed in the box. The K-band radar used in the system broadcasts a directional radar beam over an approximate one-mile range.
In September 1999, the speed monitor display was deployed at a work zone on Interstate Highway 35. The purpose of this field test was to evaluate the impact of the speed display on reducing vehicles’ speed and increasing speed uniformity at work zones.

Case Study Evaluation

The case study work zone consisted of a left lane closure with a crossover leading into head-to-head traffic. The speed display was mounted atop a stationary pole located 2,250 feet upstream of the lane closure taper. Similar to SWS testing, traffic flow performance data were collected at 1,500 feet and 500 feet upstream of the taper using two traffic data collection trailers. In this case, traffic data were recorded for two days prior to the deployment of the devices and four days after installation under two modes (active radar only and active radar and display) for five hours each day. The active radar mode (mode one) was used to test just the impact of the radar signal. The active radar and display mode (mode two) was used to test the impact of the radar signal combined with the reaction of drivers observing their speed shown on the display board.

The speed data initially were grouped into one-before and two-after data sets (i.e., modes one and two) for each data collection site (i.e., 1,500 feet and 500 feet upstream of the taper). The speed data parameters were determined for passenger cars, non-passenger cars, and all vehicles for all six data sets (i.e., before and after data [under two modes] at 1,500 feet and 500 feet).

Results

We experienced a modest mean speed decrease when the speed monitor display was deployed. We also found an increase in vehicle percentages complying with the posted speed limit (i.e., 55 mph), an increase in vehicles traveling at the 10-mph pace, and a reduction in the 10-mph speed interval.

In order to determine whether the difference between the mean speed before and after deployment of the speed monitor display was statistically significant, t-tests were conducted at the 0.05 level of significance. The average speed decrease was statistically significant in any case. However, the device did seem to reduce the number and percentage of very high-speed vehicles, it increased the number of vehicles in the pace, and it reduced the pace speed. In other words, the device seemed to improve the speed of traffic in terms of the variability of speeds and the number of very high-speed vehicles but it does not provide a statistically-significant reduction in average speed. We felt, however, that the size of the characters on the unit we tested were smaller than those needed for vehicles traveling at freeway speeds. The device may have been more successful if the variable message sign was larger or if the current sign were used on an arterial street system.

ACKNOWLEDGMENT

The authors thank the Iowa Department of Transportation for its support for this project. The opinions and views in this paper are those of the authors and not necessarily of the sponsoring agencies.

CONCLUSIONS

Of the three devices tested, the Wizard CB Alert System provided the most promising results, although our measurements of effectiveness were largely subjective. We have recommended that the Iowa DOT consider using this device at other moving or static work zones. Neither the Safety Warning System (SWS) nor the Speed Monitor Display (SMD) resulted in a statistically-significant reduction in the average speed of vehicles approaching the work zone. SWS did, however, provide subjective evidence that it was improving the speed performance of vehicles approaching the work zone. Others have found disappointing results using similar devices, although our findings may in part be due to lack of vehicles equipped with SWS devices and the size of the letters in the variable message signs used by the SMD.
Modeling Traffic Flows Through a Modern Roundabout Based on Video Data

GREG LUTTRELL, EUGENE R. RUSSELL, AND MARGARET RYS

In 1997, the City of Manhattan installed the first modern roundabout in the state of Kansas. Field study of the operation of the roundabout has been conducted by a Kansas State University research team. Comparison to comparable traffic flows at stop controlled intersections have been made. Using the measured field conditions as a starting point, the SIDRA computer model was used to evaluate a range of traffic conditions. Field traffic conditions and measured delays were used to check output from the models and assist in model validation. SIDRA was used to replicate field conditions to test the projected roundabout operation at increasing traffic levels. The model results of the roundabout and other intersection traffic control are compared. The methodology should be valuable to any community desiring to consider roundabouts as a viable, cost-effective intersection traffic control device. Key words: roundabout, capacity, safety, intersection.

INTRODUCTION

This study examined the operation of a roundabout compared to other intersection control scenarios. Traffic counts were observed by way of a specially designed 360° view video camera. The camera was linked to video recording equipment. The collected traffic counts became inputs into the computer intersection and roundabout evaluation program: Signalized and Unsignalized Intersection Design and Research Aid, version 5.20b (SIDRA). SIDRA is based on gap acceptance theories adopted by the Australian Road Authority (1). Six of the SIDRA outputs were selected as measures of effectiveness (MOEs) for analysis and comparison of intersection operation; 1.) 95% queue, 2.) average delay, 3.) maximum approach delay, 4.) proportion stopped, 5.) maximum approach stopped, and 6.) degree of saturation. These MOEs were statistically analyzed to determine how the four intersection control scenarios compared with one another.

The Manhattan Roundabout

The Manhattan roundabout was the first modern roundabout built in the state of Kansas. It is located at the intersection of two collector roads adjacent to a residential area. For three years prior to 1997, the intersection had an average of three crashes per year. Citizens were concerned and wanted additional traffic control devices. The city engineer considered four-way stop control, but decided to construct a roundabout instead. The roundabout was completed in the fall of 1997.

The Manhattan roundabout controls a four-leg intersection. All legs are two lanes—one entering and one exiting,—parking is allowed on both sides of all approach roads. There is one circulating lane. All approaches are YIELD controlled, and have raised splitter islands and marked crosswalks (see FIGURE 1). The central island is 16.7 meters (54 feet) in diameter and the approach lane width ranges from 4.0 to 4.9 meters (13 to 16 feet).

Hourly Traffic Volumes

Jacquemart summarized existing US roundabouts in relation to entering traffic volumes. Entering traffic volumes ranged from a high of around 4,700 (Long Beach) to 300 (Los Vegas) (2). The Manhattan roundabout carries traffic at the bottom end of the range for existing roundabouts in the United States (see FIGURE 2). Twenty-two hourly entering volumes recorded at the Manhattan roundabout for study analysis ranged from 224 to 402 vehicles. The average hourly entering volume for the Manhattan roundabout was 310 vehicles.

Statistical testing indicated that the hourly samples were normally distributed. This was important in that the statistical analysis was based on an assumption of having normally distributed data. This analysis examined the operation of the roundabout intersection under four operational scenarios: 1.) two-way STOP control with single lane approaches (2S); 2.) four-way STOP control with single lane approaches (4S); 3.) four-way STOP control with multi-lane approaches, i.e., each
approach modeled with a separate left turn lane and shared through/right lane (4SL); and
4. roundabout intersection control (RA).

The traffic control prior to construction of the roundabout is represented by 2S. Alternative configurations suggested by the neighborhood residents are represented by 4S and 4SL. The city engineer opted to install a roundabout (RA).

**DATA COLLECTION**

Each vehicle must be tracked through the roundabout to determine what turn is made. Traditional traffic counting techniques were not considered feasible for this study, as resources were not adequate. The City of Manhattan obtained and installed a specially designed video camera and recording equipment for data collection. Intelligent Highway Systems, Inc. (3) supplied the camera, which provided a full 360° view of the intersection. Videotape records allowed the data to be collected and viewed for later examination.

The camera was installed on an existing street light pole at the intersection (see FIGURE 3). Video images were fed down the pole to a VCR assembly inside a weather tight cabinet (see FIGURE 4). The camera was mounted at a height of approximately 6.5 meters (20 feet) which provided ample sight of the entire intersection.

Once the videotapes were collected, they were viewed, and traffic counts recorded in 15-minute intervals. This allowed the heaviest traffic flows from each tape to be used for analysis.

**FIGURE 3 Omnidirectional camera mounted on power pole**

**SIDRA ANALYSIS**

The data were input into the computer program SIDRA to obtain values for the six MOEs selected for the analysis of the intersection under the four operational scenarios described above. (See TABLE 1). The roundabout geometric features required for SIDRA were based on measurements taken from the construction plans.

SIDRA evaluates the operation of an intersection using the maximum hourly flow as calculated by the equation: \( \text{Vol}_{\text{SIDRA}} = \frac{\text{Vol}_{\text{veh}}}{\text{Peak Hour Factor}} \). Therefore, the SIDRA analysis hour volumes are higher than the actual hourly volumes counted.
STATISTICAL ANALYSIS

The output from SIDRA was analyzed to determine how the operation of the roundabout compared to the three other intersection control scenarios. Statistical tests (4, 5) were performed using the Statistical Analysis Software, version 6.12 (SAS), on the Kansas State University Unix operating system.

Two base assumptions exist for the use of most statistical tests: normality and equal variances. These two data assumptions were tested prior to determining what specific statistical tests to use to evaluate the intersection operation (see TABLE 2). The first test of normality was an evaluation of the interquartile range divided by the standard deviation. A normal distribution was indicated if this ratio was within +/- 50% of the desired 1.3 value. The Shapiro-Wilk test was used as the second method for evaluating normality. This test is sensitive to small samples. Therefore, to lessen the possibility of a false rejection, a small alpha value of 0.01 was chosen. Equal variances between the four data sets were tested using the Levene’s test. This test is sensitive to normality assumptions; therefore, the null hypothesis was rejected only if the test p-value was less than 0.01 (value).

One of three different statistical paths was chosen to test for statistically significant differences of the six MOEs based on the results of the normality and equal variance tests, i.e., IIIA, IIIB, or IIIC (see TABLE 2).

Comparison of the four operational scenarios analyzed is presented below for each MOE.

### TABLE 1 Intersection Measures of Effectiveness

<table>
<thead>
<tr>
<th>Measure of Effectiveness</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>95% Queue</td>
<td>Total Length of the queue for all approaches at the 95% confidence level*</td>
</tr>
<tr>
<td>Average Delay</td>
<td>Average vehicle delay for all entering vehicles</td>
</tr>
<tr>
<td>Maximum Approach Delay</td>
<td>Average vehicle delay for the approach with the highest average vehicle delay</td>
</tr>
<tr>
<td>Proportion Stopped</td>
<td>Proportion of entering vehicles that are required to stop due to vehicles already in the intersection</td>
</tr>
<tr>
<td>Maximum Proportion Stopped</td>
<td>Proportion of entering vehicles that are required to stop due to vehicles already in the intersection on the approach with the highest proportion stopped value</td>
</tr>
<tr>
<td>Degree of Saturation</td>
<td>Amount of capacity that is consumed by the current traffic loading (referred to as v/c ratio)</td>
</tr>
</tbody>
</table>

*Queue lengths were based on a vehicle length of 8 meters (25 feet)

### TABLE 2 Statistical Test Summary - Overview

<table>
<thead>
<tr>
<th>Test</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>I. Normality</td>
<td></td>
</tr>
<tr>
<td>- IQR/S = 1.3</td>
<td>Interquartile divided by standard deviation</td>
</tr>
<tr>
<td>- Shapiro-Wilk P-value</td>
<td>$H_0$: 'have a normal distribution', $\alpha = 0.01$</td>
</tr>
<tr>
<td>II. Equal Variances</td>
<td></td>
</tr>
<tr>
<td>Levene’s test</td>
<td>$H_0: \sigma^2 = \sigma^2_{RA} = \sigma^2_{4S} = \sigma^2_{4SL}$, $\alpha = 0.01$</td>
</tr>
<tr>
<td>III.A. Normal w/ Equal Variances</td>
<td></td>
</tr>
<tr>
<td>Analysis of Variance F-test</td>
<td>$H_0: \mu_{2S} = \mu_{RA} = \mu_{4S} = \mu_{4SL}$, $\alpha = 0.05$</td>
</tr>
<tr>
<td>- Fail to reject – means considered equal, analysis stops</td>
<td></td>
</tr>
<tr>
<td>- Reject – perform multiple comparisons</td>
<td>Tukey’s and Duncan tests</td>
</tr>
<tr>
<td>III.B. Normal w/ Unequal Variances</td>
<td></td>
</tr>
<tr>
<td>Welch’s test</td>
<td>$H_0: \mu_{2S} = \mu_{RA} = \mu_{4S} = \mu_{4SL}$, $\alpha = 0.05$</td>
</tr>
<tr>
<td>- Fail to reject – means considered equal, analysis stops</td>
<td></td>
</tr>
<tr>
<td>- Reject – perform multiple comparisons</td>
<td>Fisher Least Significant Difference test</td>
</tr>
<tr>
<td>III.C. Not normal</td>
<td></td>
</tr>
<tr>
<td>Kruskal-Wallis test</td>
<td>$H_0$: ‘Population distributions are the same’, $\alpha = 0.05$</td>
</tr>
<tr>
<td>- Fail to reject – distributions considered equal, analysis stops</td>
<td></td>
</tr>
<tr>
<td>- Reject – Observe data plots to determine rank order</td>
<td></td>
</tr>
</tbody>
</table>

### 95% Queue

The 95% queue values ranged as shown in TABLE 3 and FIGURE 5. The data was found to be normally distributed with unequal variances. Data groupings were found to be $4S\neq 4SL=S\neq RA$. Based on the mean values, the roundabout produced the lowest level of 95% queue followed by the two-way STOP, four-way STOP and four-way STOP with turn lanes.

### TABLE 3 95% Queue

<table>
<thead>
<tr>
<th>Intersection Control</th>
<th>2-way STOP (2S)</th>
<th>Roundabout (RA)</th>
<th>4-way STOP (4S)</th>
<th>4-way STOP w/ Turn Lanes (4SL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>11 m (35 ft)</td>
<td>8 m (26 ft)</td>
<td>15 m (49 ft)</td>
<td>10 m (34 ft)</td>
</tr>
<tr>
<td>Maximum</td>
<td>24 m (78 ft)</td>
<td>17 m (54 ft)</td>
<td>34 m (110 ft)</td>
<td>28 m (92 ft)</td>
</tr>
<tr>
<td>Mean ($\mu$)</td>
<td>16 m (53 ft)</td>
<td>11 m (37 ft)</td>
<td>23 m (75 ft)</td>
<td>18 m (58 ft)</td>
</tr>
<tr>
<td>Standard Deviation ($\sigma$)</td>
<td>3.6 m (11.8 ft)</td>
<td>7.9 m (2.4 ft)</td>
<td>4.8 m (15.6 ft)</td>
<td>4.9 m (15.9 ft)</td>
</tr>
</tbody>
</table>
Average Delay

The average intersection delay values are shown in TABLE 4 and FIGURE 6. The average delay values were found to be not normally distributed. Using mean values and the non-parametric statistical test, the roundabout can be said to operate with the same average vehicle delay as the two-way STOP control and with less average vehicle delay than either of the four-way STOP scenarios $RA < 2S < 4S < 4SL$.

TABLE 4 Average Vehicle Delay

<table>
<thead>
<tr>
<th>Intersection Control</th>
<th>2-way STOP (2S)</th>
<th>Roundabout (RA)</th>
<th>4-way STOP (4S)</th>
<th>4-way STOP w/ Turn Lanes (4SL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>6.3</td>
<td>7.5</td>
<td>14.4</td>
<td>18.5</td>
</tr>
<tr>
<td>Maximum</td>
<td>10.2</td>
<td>8.1</td>
<td>23.1</td>
<td>21.7</td>
</tr>
<tr>
<td>Mean ($\mu$)</td>
<td>8.2</td>
<td>7.9</td>
<td>16.8</td>
<td>19.8</td>
</tr>
<tr>
<td>Standard Deviation ($\sigma$)</td>
<td>1.2</td>
<td>0.2</td>
<td>1.7</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Note: Average vehicle delay is measured in sec/veh.

Maximum Approach Delay

The maximum approach delay values are shown in TABLE 5 and FIGURE 9. These values were found to be not normally distributed. All distributions were found to be different. Based on the mean values, the roundabout was found to operate with the lowest maximum average vehicle delay, followed in order by the two-way STOP, four-way STOP and four-way STOP with turn lanes ($RA < 2S < 4S < 4SL$).

TABLE 5 Maximum Approach Average Vehicle Delay

<table>
<thead>
<tr>
<th>Intersection Control</th>
<th>2-way STOP (2S)</th>
<th>Roundabout (RA)</th>
<th>4-way STOP (4S)</th>
<th>4-way STOP w/ Turn Lanes (4SL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>11.3</td>
<td>8.4</td>
<td>19.7</td>
<td>24.2</td>
</tr>
<tr>
<td>Maximum</td>
<td>14.6</td>
<td>9.2</td>
<td>65.0</td>
<td>40.1</td>
</tr>
<tr>
<td>Mean ($\mu$)</td>
<td>12.9</td>
<td>8.8</td>
<td>27.9</td>
<td>31.5</td>
</tr>
<tr>
<td>Standard Deviation ($\sigma$)</td>
<td>1.0</td>
<td>0.2</td>
<td>9.1</td>
<td>4.7</td>
</tr>
</tbody>
</table>

Note: Maximum approach average vehicle daily measured in sec/veh.

Proportion Stopped

The proportion stopped is a measure provided in SIDRA and represents the proportion of vehicles stopped by other traffic already in the intersection. The values for proportion stopped can range from 0.0 to 1.0 (see TABLE 6 and FIGURE 8). The proportion stopped values were found to be normally distributed with equal variances. Tukey and Duncan’s multiple comparison tests both concluded that all four means could be considered statistically different and that the roundabout experienced the lowest value for proportion stopped, followed by the two-way STOP, four-way STOP with turn lanes and the four-way STOP ($RA < 2S < 4SL < 4S$).

TABLE 6 Proportion Stopped

<table>
<thead>
<tr>
<th>Intersection Control</th>
<th>2-way STOP (2S)</th>
<th>Roundabout (RA)</th>
<th>4-way STOP (4S)</th>
<th>4-way STOP w/ Turn Lanes (4SL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>0.17</td>
<td>0.13</td>
<td>0.78</td>
<td>0.69</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.31</td>
<td>0.23</td>
<td>0.87</td>
<td>0.79</td>
</tr>
<tr>
<td>Mean ($\mu$)</td>
<td>0.24</td>
<td>0.18</td>
<td>0.83</td>
<td>0.76</td>
</tr>
<tr>
<td>Standard Deviation ($\sigma$)</td>
<td>0.04</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
</tbody>
</table>
Maximum Proportion Stopped

The maximum proportion stopped values can range from 0.0 to 1.0 (see TABLE 7 and FIGURE 9). The maximum proportion stopped values were found to be normally distributed with unequal variances. The Fisher’s groupings found RA ≠ 2S ≠ 4SL ≠ 4S. Based on the mean values, the roundabout can be said to operate best for this MOE.

TABLE 7 Maximum Approach Proportion Stopped

<table>
<thead>
<tr>
<th>Intersection Control</th>
<th>2-way STOP (2S)</th>
<th>Roundabout (RA)</th>
<th>4-way STOP (4S)</th>
<th>4-way STOP w/ Turn Lanes (4SL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>0.21</td>
<td>0.21</td>
<td>0.89</td>
<td>0.82</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.44</td>
<td>0.32</td>
<td>0.99</td>
<td>0.91</td>
</tr>
<tr>
<td>Mean (µ)</td>
<td>0.31</td>
<td>0.25</td>
<td>0.94</td>
<td>0.87</td>
</tr>
<tr>
<td>Standard Deviation (σ)</td>
<td>0.06</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
</tbody>
</table>

Degree of Saturation

The degree of saturation (volume to capacity (v/c) ratio) values (see TABLE 8 and FIGURE 10) were found to be normally distributed with unequal variances. All four means were determined to be statistically different from one another. The roundabout operates at a lower degree of saturation value than the other three scenarios, followed by the two-way STOP, the four-way STOP and the four-way STOP with turn lanes (RA<2S<4S<4SL).

TABLE 8 Degree of Saturation

<table>
<thead>
<tr>
<th>Intersection Control</th>
<th>2-way STOP (2S)</th>
<th>Roundabout (RA)</th>
<th>4-way STOP (4S)</th>
<th>4-way STOP w/ Turn Lanes (4SL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>0.107</td>
<td>0.061</td>
<td>0.176</td>
<td>0.251</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.280</td>
<td>0.150</td>
<td>0.402</td>
<td>0.575</td>
</tr>
<tr>
<td>Mean (µ)</td>
<td>0.168</td>
<td>0.095</td>
<td>0.260</td>
<td>0.362</td>
</tr>
<tr>
<td>Standard Deviation (σ)</td>
<td>0.052</td>
<td>0.023</td>
<td>0.059</td>
<td>0.085</td>
</tr>
</tbody>
</table>

STUDY SUMMARY

This study evaluated the operation of an existing modern roundabout located in Manhattan, Kansas. The Manhattan roundabout was completed in the fall of 1997. The roundabout operates with approximately 4,600 daily and 310 peak hour entering vehicles. The operation of the roundabout was evaluated relative to three alternatives: two-way STOP, four-way STOP with, and four-way STOP without separate left turn lanes.

Intersection operation was evaluated using six measures of effectiveness (MOEs). Values for these MOEs were obtained from the computer program SIDRA. The results of the MOE evaluation (see TABLE 9) found that the roundabout operates better than the other three intersection control scenarios in all but one case—in that case, it is equal to the two-way STOP alternative.

This study evaluated the operation of the roundabout in Manhattan, Kansas versus three other intersection control scenarios. The conclusions may apply to other locals only if the overall conditions are similar to that found in this study.

Summary of Statistical Analysis for Intersection Scenarios

The purpose of analyzing the MOE data was to determine if and how the four intersection control scenarios differed operationally under similar levels of traffic flows. The summary of the statistical analyses of the four intersection scenarios studied are shown in TABLE 9.
TABLE 9 Summary of MOE Statistical Results

<table>
<thead>
<tr>
<th>Measure of Effectiveness</th>
<th>Statistical Result</th>
<th>Leading Traffic Control</th>
</tr>
</thead>
<tbody>
<tr>
<td>95% Queue Average Delay</td>
<td>RA &lt; 4SL = 2S &lt; 4S</td>
<td>Roundabout/2-WAY stop (tie)</td>
</tr>
<tr>
<td>Maximum Approach Delay</td>
<td>RA &lt; S2 &lt; 4S &lt; 4SL</td>
<td>Roundabout</td>
</tr>
<tr>
<td>Proportion Stopped</td>
<td>RA &lt; 2S &lt; 4SL &lt; 4S</td>
<td>Roundabout</td>
</tr>
<tr>
<td>Maximum Approach Stopped</td>
<td>RA &lt; 2S &lt; 4SL &lt; 4S</td>
<td>Roundabout</td>
</tr>
<tr>
<td>Degree of Saturation</td>
<td>RA &lt; 2S &lt; 4S &lt; 4S</td>
<td>Roundabout</td>
</tr>
</tbody>
</table>

*based on having the best performance on each MOE. All statistical testing yielded results at the 95% confidence level.

Safety

In the 2½ years since the roundabout has been in operation, there have been no crashes. This has been determined to be a significant reduction.

CONCLUSIONS

Under all conditions except one, the roundabout performed statistically better than the previous intersection control—two-way STOP. Under all measures of effectiveness, the roundabout was found to operate statistically better than the two, four-way STOP alternatives tested. In regard to safety, there have been no crashes in the approximately 2½ years since the roundabout was completed—a statistically significant reduction. The decision to build the roundabout was a good decision.

ACKNOWLEDGEMENTS

Funding for this study was supplied by the Mack Blackwell Transportation Center (MBTC) with matching funds from the Kansas Department of Transportation (KDOT) and an in-kind contribution and assistance, from the City of Manhattan. The authors are solely responsible for all statements and conclusions in this paper.

REFERENCES

Towards Semi-Automated Arterials:
Dynamic Traffic Signal Control with Time-Dependent Variable Speed

GHASSAN ABU-LEBDEH

This paper presents a framework for an advanced concept for traffic control on congested urban arterials. The main idea of semi-automated arterials is to utilize a dynamically optimized time-dependent variable speed as an additional signal control parameter. In a semi-automated arterial, speeds would be automatically optimized and set by a central computer. They would change between links and over time in response to changing traffic conditions. Drivers would follow the optimized speed as they enter a link. Once an optimal speed has been set for a link, it remains constant until the control cycle ends. Link speeds would be updated only at the end of every control cycle. The control cycle may change in length as system conditions evolve. The new control concept was tested on a congested arterial with multiple links. The arterial system was modeled as a discrete event time varying dynamical system with a control period spanning several cycles. System throughput was maximized subject to such critical operational measures as intersection blockage, queue spillbacks, and other relevant traffic operation measures. Genetic Algorithms (GAs) were used as an optimization tool. Results show that the semi-automated arterial concept will significantly improve traffic flow. The new control concept is suitable for on-line implementation in an ITS setting. Key words: signal control, variable speed, congestion.

INTRODUCTION

Urban traffic congestion is becoming a fact of life in many urban and suburban areas in the U.S. and elsewhere in the world. More than 70% of the U.S. urban peak-hour traffic is congested (1). Furthermore, traffic jams contribute substantially to air pollution. And travel demand continues to far outpace provision of roadway capacity. For example, for the Washington DC area, vehicle miles of travel (VMT) are expected to increase by 75% between 1990 and 2020; whereas lane miles of capacity are expected to increase by only 22%. Consequently, vehicle hours of delay are projected to increase by 480%, and 85% of regional travel is expected to occur on congested roadways (2). There is little reason to believe that congestion problems will go away by simply building new capacity. A different approach is needed to better manage and utilize existing transportation systems. In urban areas, traffic signal control plays a major role in the quality of traffic operations. As part of that, more advanced traffic signal hardware and control procedures are needed.

This paper presents a framework for an advanced signal control scheme. In this scheme, speed is introduced as control parameters as opposed to a soft constraint. Speed is optimized and allowed to vary over time and space. The optimized speeds are then integrated into a dynamic signal control algorithm.

BACKGROUND

Traditionally, traffic signal operations are optimized using the control variables of splits, offsets, and phasing. These parameters are usually designed to respond to traffic conditions, including desired speed, so that traffic progression is attained or delay is minimized. Either way, speed is used as an input on the assumption that it is fixed and that its value is “optimal.” But little examination of how speeds are normally set reveals that an optimal speed value should be a function of traffic conditions, which, in turn, is time-dependent. In the past, technical difficulties in reliably detecting traffic conditions may have been a serious hindrance in devising time-dependent changeable speeds based on real-time traffic information. Time has changed and with it comes a host of new technologies that are changing the way we do business, including traffic sensing and traffic signal operations design. It is becoming increasingly possible to reliably, but not perfectly, detect traffic conditions in real time. This, in turn, makes it feasible to vary speed based on field traffic information.

The concept of variable speed in traffic control is not new. It has been successfully used on freeways for quite some time particularly in Europe. It seems timely to start exploring the feasibility of applying a similar concept on signalized arterials. That is, vary speed dynamically on arterials and use it as an additional control parameter. This is what this paper is set out to explore.

This paper presents a framework for an algorithm that dynamically optimizes speeds on arterial links and uses them as additional control parameters. The optimized speeds are integrated, in parallel, into a dynamic signal control algorithm (3). Genetic Algorithms (GAs) were used to solve the optimization problem. The GAs’ adaptive, robust, directed search and flexible form of the objective function make them an ideal solution technique for dynamic control problems similar to the problem formulated in this paper.

NEW CONTROL SCHEME

The key feature of the control scheme of this paper is to dynamically vary speed and use it as a control variable. Speed is optimized in parallel with the other control variables based on evolving traffic conditions. The role of speed in this control scheme comes into play in adapting the values of offsets to traffic conditions on downstream links so that output is maximized and intersection blockage is prevented. Specifically, offsets between neighboring arterial approaches are set based on queue length and the available space on a link. Speed
determines the time needed to travel the available space on the link (i.e., the distance between the upstream intersection and the downstream queue). This time element is a component of the offset-determining algorithm. Hence, different speeds would result in different offsets for the same queue length and the same available space on a given link. In this framework, speed, in effect, is optimized to produce system-optimal performance (as opposed to link-optimal). Vehicle acceleration and deceleration rates are assumed constant.

The control optimization problem is structured as a discrete event time-varying dynamical system. The optimal arterial control formulation is:

Find the trajectory of control variables
- \( g_{ik}^a \) (green splits)
- \( \text{Speed}_{ik} \) (speeds)
- \( \text{Offsets}_{ik} \) (offsets)

That maximizes the objective function:

\[
\sum_{\text{cycle}} \sum_{\text{link}} \text{System output (Link - vehicles)} \times \text{Speed}
\]

Subject to the constraints on the state variables
- \( q_{ik+1} = q_{ik} + A \frac{V}{k} - D \frac{V}{k} \) (queue formation and dissipation models)

and the control variables
- \( g_{ik,\text{min}} \leq g_{ik} \leq g_{ik,\text{max}} \) (domain of green splits)
- \( \text{Speed}_{ik,\text{min}} \leq \text{Speed}_{ik} \leq \text{Speed}_{ik,\text{max}} \) (domain of speeds)

\( \text{Offsets}_{ik} = f(\text{queue length}_{ik}, \text{space}_{ik}, \text{speed}_{ik}) \)

Where \( q \) is the queue length, \( AV \) is volume of arriving traffic, and \( DV \) is volume of departing traffic. The cycle number is referred to as \( k \), and \( i \) is the link, or approach number.

The decision variables in this optimization problem are green splits (hence cycle length), offsets, and speeds. Speeds were allowed to vary between 15 to 40 ft/sec. Phasing was not optimized. The algorithm was applied to a congested system of seven intersections for a duration of ten cycles. Micro-Genetic Algorithms (micro-GAs) were used for optimization. Micro-GAs were used because of their ability to overcome combinatorial explosion and their flexibility in formulating the objective function. The results are discussed next.

**EVALUATION AND DISCUSSION OF RESULTS**

Several measures of effectiveness (MOE) were used to evaluate system performance with variable speed. The results were contrasted to system performance where a constant speed of 40 ft/sec is used. All inputs and parameter values were the same for the variable and constant speed schemes. Overall, using the scheme of variable speed appears to improve system performance. Specific results are discussed below. All results are presented on a per lane basis.

### Number of Stops

A vehicle is said to have stopped if it is unable to leave the link during the same cycle it enters. The number of stops provides a general measure of quality of progression, fuel consumption, and vehicle emissions. Figure 1 shows the number of stops per unit of output for each link for both variable and constant speed control schemes. The number of stops is lower under the variable speed scheme, and for the entire system there were 25% fewer stops. At the link level, the number of stops was generally lower under the variable speed scheme. The trend between individual links varied; two of the links experienced more stops under the variable speed scheme.

The lower number of stops with variable speeds is expected. The algorithm has a “system view” of traffic conditions, or, in other words, it can “see” beyond the immediate downstream link. This makes it possible to employ a speed value that is system-optimal as opposed to link-optimal which would be the case if speed selection is left to drivers. In real-world conditions, drivers can see only conditions on the link they are driving on. Hence, they select a speed they think optimal for that link only to find themselves forced to stop at intersections further downstream. Employing variable speed, in effect, overcomes this condition. That is, it takes the decision of selecting speed from drivers and gives it to the system “central controller,” which, in turn assigns speeds to system links so that the overall system performance is optimal.

### System Traffic Content

System content measures the volume of traffic present on system links during a given time window. Higher traffic content means less chance for progression and more chance of stopping. Figure 2 shows the system content at the beginning of each cycle. Traffic content is lower under the variable speed control scheme than under the constant speed scheme. The trend is clearer at the link level (Figure 3). Here, we see that the variable speed scheme was far more successful at reducing standing queues. This outcome follows from the fact that fewer vehicles are arriving prematurely at the respective approaches. Flexibility in speed makes this possible.
System Output

System output, measured in link-vehicles and normalized by arterial green time, is shown in Figure 4. The output is only slightly higher under the variable speed control scheme. The experimental setup could have contributed to this marginal difference: the optimized variable speeds were allowed to vary from 15 ft/sec to 40 ft/sec; whereas under the constant speed scheme, speed was set at 40 ft/sec. The calculated average speed based on all system links (not shown) for the variable speed case was lower than that of the constant speed case.

CONCLUSIONS

This paper presents a framework for a signal control algorithm wherein speed is used as a decision variable as opposed to a soft constraint—as is traditionally the case. The algorithm treats link speeds as functions of time, dynamically optimizes them, and then uses them as control parameters besides green splits and offsets. Speed is dynamically optimized based on evolving traffic conditions. Using speed as another control parameter adds a new dimension to the control problem and hence increases the spectrum of control choices. The algorithm was tested on a congested seven-intersection arterial system. The results were then contrasted to those with constant speed. Variable speed-based control was more efficient in many respects. It resulted in fewer stops, better conditions for progression, better queue management, and more system output. This implies improved traffic flow. There would be fewer premature arrivals at intersections and formation of queues—hence shorter time to clear queues—fewer stops and "stop-and-go" maneuvers, and less interactions and complications resulting from continuous stopping and starting shock waves. Reduced fuel consumption and harmful emissions are other desirable outcomes.

The new control concept is suitable for on-line implementation in an ITS setting. It can be extended to congested networks with closed loops. The value of this procedure is more when traffic on system links varies considerably either due to geometry or traffic generation factors. In the results presented above we assumed complete driver compliance with speed—an overly optimistic assumption. This and other implementation issues will have to be addressed before variable speed type control can be implemented in real-world conditions. It is recommended that this concept of control be further evaluated and validated in preparation for implementation.

REFERENCES

Simulation of Detector Locations on an Arterial Street Management System

GARY B. THOMAS AND JONATHAN E. UPHURCH

The research presented in this paper used computer simulation to investigate the relationship between detector location and the ability of a system to monitor traffic characteristics (flow, speed, occupancy) and from them estimate link travel characteristics (link speed, travel time, intersection delay). A 3 mi (4.8 km) section of roadway in the Phoenix metropolitan area was simulated using the program CORSIM. Four detector locations within each major link were analyzed. One detector location was downstream of a major intersection; the other three locations were upstream of a major intersection. Statistical techniques, in the form of regression analysis, were used to evaluate the various dependent and independent variables. Results of the analysis indicated the link travel characteristics are unique to each link on the network. Of the variables examined, there was no one singular relationship that can be used to predict link travel characteristics. Further, there was no particular detector location that proved to be superior to all other detector locations. Detectors located downstream of major intersections can use traffic flow to predict link travel time with reasonable accuracy. Detectors located upstream of major intersections can use spot speed or detector occupancy to predict link travel speed with reasonable accuracy. The predictive capability applies to recurring congestion but does not apply to incident situations. The spacing of detectors can be critical to the operation of a system. The research showed that detector data obtained on one link could not accurately predict link travel characteristics on an adjacent link. Key words: arterial street management, traveler information, traffic detection, simulation.

BACKGROUND

The Phoenix, Arizona metropolitan area is in the middle of an ITS model deployment project called AZTech. This public-private partnership will use ITS technologies to provide traveler information on several major corridors in the area. The corridors will be instrumented with detectors spaced at roughly 3.2 km (2 mi) intervals. The detectors will feed information back to a regional computer server that will process the data. The data will then be disseminated back to drivers via several different mediums to provide them information about current traffic conditions.

In the area of advanced traveler information systems (ATIS), the initial focus has been on freeways. Less work has been done in the area of arterial street management as it relates to providing travelers with real-time information on traffic conditions. Certain factors, such as signal timing, parking activity, transit stops, driveway access, and turning movements, make monitoring traffic flow conditions on arterials much more difficult.

PROBLEM STATEMENT

Given the budgetary limitations of many local and state agencies, outfitting arterials with vehicle detection must be done in the most cost effective manner. The primary goal of this paper is to answer the following question: What is the relationship of detector location to link travel characteristics on an arterial street network?

STUDY AREA

The study area chosen to investigate the problem statement is a 3 mi (4.8 km) section of Southern Avenue located in Tempe and Mesa, Arizona (Figure 1). This section is one of the eight corridors selected in the AZTech ITS project and has an average daily traffic (ADT) over 42,000 vehicles per day.

The westbound direction has three through lanes throughout. The eastbound direction has two through lanes to the west of Dobson Road and three through lanes to the east of Dobson Road. The portion of the corridor in Mesa has a raised median. The Tempe portion has a two-way left-turn lane in the center of the roadway. Separate right-turn lanes exist on some approaches at major intersections. The speed limit varies from 40 to 45 mph (64 to 72 kph).

There are a total of twelve signalized intersections in the study area operating on either a 94-second or a 110-second cycle length. All of the signals have some level of detection at the intersection. For permissive-protected left turns, both Tempe and Mesa use the third-car actuation technique.

LITERATURE REVIEW

A literature review was performed to determine what research has been done previously in this area. Much of the research on arterial street detection has been geared towards optimizing traffic signal operations or incident detection. Very little has been done in the area of traveler information on arterial streets. A full literature review can be found in Optimal Detector Location on Arterial Streets for Advanced Traveler Information Systems (1). Sisiopiku et al. have done the most significant research in the area similar to this paper (2). This research was possibly the first of its kind involving the correlation of system detectors and travel time through simulation.
Among some of the conclusions of their study were the following:
1. Travel time is independent of both flow and occupancy under conditions of low traffic demand.
2. As percentage occupancy increases, the correlation between travel time and occupancy becomes more significant.
3. Simulation and field data indicate a strong correlation between flow and occupancy for certain ranges of values.

They noted that the observed relationships are complex and that substantial research is needed to investigate the relationship completely. The authors suggested that future research will further detail the models.

The research presented in this paper builds upon the research by Sisiopiku. The primary change is that detector location is varied in order to determine if there is an optimal location to place detectors in order to make accurate predictions.

DATA COLLECTION

Roadway and Signal Information

Roadway information was gathered from a number of sources. Traffic signal construction plans and aerial photographs were obtained from the cities for most of the study area. The aerial photographs provided turn lane storage lengths. A geographic information system (GIS) base map was obtained to determine the distance between signals to a degree of accuracy superior to aerial photographs or field measurements (< 3 m). Both jurisdictions provided signal timing sheets for all of the traffic signals in the study area. The timing sheets provided cycle lengths, phase split settings, yellow and red clearance intervals, minimum green times, pedestrian clearance intervals, and cycle offsets. All of these data elements are used to define the actuated controllers in the CORSIM program.

Traffic Information

Fifteen-minute morning peak turning movement counts were made at all of the signalized intersections in the study area. These counts were used as input to the simulation network. Mid-link traffic volumes were also collected at three locations as part of the AZTech project. These counts were used to calibrate the network.

The City of Mesa provided additional traffic volume information in the form of approach counts for the intersections of Southern/Dobson and Southern/Alma School. Since these intersections represent major inputs into the traffic network, the approach volume counts were used to determine volume inputs at the respective nodes. This research examined a fifteen-minute interval, and it was assumed that the turning movement percentages are the same throughout the time interval.

Heavily used transit routes can greatly affect the operation of the network. Although transit activity is low along Southern Avenue, the routes were coded into the network.

NETWORK CALIBRATION

Three methods of calibration were examined. Although other more precise methods of calibration exist – measured observations and the two-fluid method (3) – the simulation model was calibrated by observation due to limited resources in collecting data in the first case and lack of source coding in the second case.

The observation method involves visually comparing the graphical and tabular output of CORSIM with the actual conditions in the field. Observations can be made at intersections relating to cycle failures and queue lengths. This method is not as accurate as either of the first two methods. However, the time and expense to collect field observations are much lower. The goal of this research was to determine optimal detector location on an arterial. This research could have been performed on a fictitious section of roadway and

FIGURE 1 Study area
still achieved the desired goals. In that case, calibration would not have been an issue. However, to improve acceptance of the research results and to maximize the application to the AZTech project, the research simulated one of the AZTech corridors.

Observations were made at the four major intersections. The primary observed value was the average queue length in the left turn pockets and the through lanes. The simulation of the base volumes reflected similar queue lengths at the major intersections. Therefore, it was concluded that the simulation model is a reasonable approximation of actual conditions.

A comparison was also made between the simulated traffic counts and the actual traffic counts observed on the links. With the exception of link 5, all of the comparisons are well within an acceptable range (< 6 percent difference).

NETWORK DETECTION

The most recent version of CORSIM (Version 4.2) allows the user to place surveillance detectors on the network links. The simulated detectors measure three data items: traffic volume, mean spot speed, and occupancy. Each of the six links was assigned a number (1 – 6) for analysis purposes.

Network detection was placed in four locations on each 1 mi (1.6 km) segment of roadway. Detectors were placed in all lanes. As shown in Figure 2, detectors were placed in three locations approaching an intersection: 900 ft (275 m), 600 ft (183 m), and 300 ft (92 m) from the stop bar and labeled “B,” “C,” and “D,” respectively.

The fourth detector (A) was placed 600 ft (183 m) downstream (and labeled position A) from the major intersection. Varying the location of the downstream detector did not have a big impact on the output variables when the distance was greater than 400 ft (122 m). If detectors are placed less than 400 ft (122 m), the results could be misleading because vehicles are still accelerating close to the intersection.

SIMULATION RUNS

Two factors were varied for this experiment: entry traffic volumes and detector location. There were six levels of entry traffic volumes: the base case scenario, base +20%, base +40%, base +60%, base +80%, and base +100%.

The existing peak hour volume in the peak direction on Southern Avenue is about 1,480 vehicles per hour, a volume that results in Level of Service D, or E, or F at the major intersections. The entry level volumes used in the simulation range from about 68 percent to 135 percent of the 1480 vph volume. The “base + 100 %” volume, therefore, forces high levels of congestion in the simulation. The actual volume/capacity (V/C) ratios produced by the simulation ranged as high as 1.7.

The experimental design resulted in a total of six separately coded networks, each with detectors placed at the four locations (Figure 3). This resulted in 24 separate traffic simulation scenarios.

Once all of the simulations were run, the relevant data for each volume scenario were reduced onto a single spreadsheet. These data included the detector information, link speed and travel time, and intersection delay. After this process was complete, there were twelve spreadsheets containing the relevant data (two for each volume level: one containing detector output data, the other containing link travel information).

The various measures of effectiveness (MOEs) were averaged over the eight simulation runs. Then all of the averaged MOEs were combined into a single spreadsheet, imported into the statistical program Minitab®. Preliminary analysis was performed on the data by plotting graphs of the following data: Link Travel Time,

![FIGURE 2 Surveillance detector locations](image-url)
Link Travel Speed, and Next Approach Stopped Delay vs. (1) Detector Volume, (2) Detector Occupancy (sum and average), (3) Average Detector Speed, and (4) V/C Ratio at the Next Intersection.

Only the through vehicles on the links were included in the data analysis. It was felt that including vehicles that traversed only a portion of the 1 mi (1.6 km) link (and turned left or right at intermediate nodes) or including vehicles that turn left or right at the major intersection nodes may distort the results. The research by Sisiopiku also did not include left- and right-turning vehicles in the analysis.

CONCLUSIONS

The amount of data generated by the simulation runs was considerable. A full analysis of every correlation is given in the main dissertation document (1). Only a few critical observations are shown in this paper.

The first conclusion that became evident was that the relationship between detector output variables and link travel characteristics is very link specific. Attempts to “normalize” the links (by dividing detector output by the number of lanes or downstream capacity) did not provide any more meaningful results. Therefore, further analysis was performed on specific links rather than the aggregation of all six links. Additionally, only two of the six links had any variability in link travel characteristics such as travel time, approach delay, and travel speed. Hence, only the relationships on these two links were analyzed using the regression tests. There was not a clearly “optimal” detector location for all links in all cases. The relationship between detector output variables and link travel characteristics is unique on each link and thus calibration is necessary for every link.

Table 1 is just one example of the numerous regression analyses performed on the data. Correlations of coefficients were calculated along with the P-value for both linear and quadratic relationships. The P-value represents the smallest level of significance that would lead to rejection of the null hypothesis. Finally, the residuals were examined. A residual is the difference between an observation and the corresponding estimated value from the regression model. A “yes” means that the residuals are normally and independently distributed with constant variance, abbreviated NID(0, (2), which is preferred.

In all regression analyses that were conducted, quadratic equations resulted in better statistics (high R-squared value, smaller P-values, and a “yes” for residuals) than did linear equations. The Table 1 data are examples. Table 1 demonstrates that detector Position A is the best location when detector volume is used to predict link travel time.

Table 2 lists the statistics for the best detector location for each combination of detector output variable and predicted link travel characteristic. For example, Table 2 demonstrated that Position A is best when detector volume is used to predict link travel time. These data are listed in the first two rows of Table 2. The remaining rows list the best detector location for other combinations of detector output variable and link travel characteristics. Two “best locations” are shown for occupancy predicting delay because the two locations are very competitive.

After complete analysis of the data, several conclusions were drawn about the relationships between detector location and link travel characteristics. These conclusions are supported by the data in Table 2.

Detector position A is a good predictor of link travel time, link travel speed, and approach delay when detector occupancy or detector speed are used as the independent variable. Detector position A is not a good predictor of travel time, link travel speed, or approach delay if detector speed is used as the independent variable.

Detector position D is a good predictor of approach delay when detector occupancy or detector speed is used as the independent variable. These conclusions are based on V/C ratios of up to 1.7.
TABLE 1  Regression Results for Link Travel Time vs. Detector Volume

<table>
<thead>
<tr>
<th>Link</th>
<th>Position</th>
<th>Equation Type</th>
<th>R-Squared</th>
<th>P-value</th>
<th>Residuals?</th>
</tr>
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<tbody>
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<td></td>
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<td></td>
<td>B</td>
<td>Linear</td>
<td>86.4</td>
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<td>Yes</td>
</tr>
<tr>
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<td></td>
<td>Quadratic</td>
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<td>C</td>
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</tr>
<tr>
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</tr>
<tr>
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<td>D</td>
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</tr>
<tr>
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<td></td>
<td>Quadratic</td>
<td>76.8</td>
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</tr>
<tr>
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<td>A</td>
<td>Linear</td>
<td>89.4</td>
<td>0.004</td>
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</tr>
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<td></td>
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<td>0.000</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>Linear</td>
<td>89.5</td>
<td>0.004</td>
<td>No</td>
</tr>
<tr>
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<td></td>
<td>Quadratic</td>
<td>98.5</td>
<td>0.002</td>
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</tr>
<tr>
<td></td>
<td>C</td>
<td>Linear</td>
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</tr>
<tr>
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<td></td>
<td>Quadratic</td>
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<td>0.004</td>
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</tr>
<tr>
<td></td>
<td>D</td>
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<tr>
<td></td>
<td></td>
<td>Quadratic</td>
<td>88.9</td>
<td>0.004</td>
<td>No</td>
</tr>
</tbody>
</table>

Given the high degree of correlations found with detector position A, it was concluded that this position is more than adequate for modeling (predicting) link travel characteristics for recurring congestion based on traffic volumes. Detector positions C and D also showed very promising results for modeling link travel characteristics for recurring congestion based on average detector speed. What this research shows is that significant thought and/or research must be given to locating system detectors for use in traveler information systems. There isn’t a “one answer fits all” solution. Two factors that must be taken into consideration are locations of minor intersection signals and the locations of uncontrolled driveways with heavy traffic volumes that may skew vehicle speeds.

Calibration for each detectorized link will be necessary to obtain reliable information. Separate calibration may also be needed for each timing plan. There are far more variables that affect travel characteristics on arterial streets than on freeway segments.

Using similar analyses, the research also attempted to answer the question of how much detection is necessary on the network to provide accurate estimations of link travel characteristics. Is detection needed on every mile link? Or are detectors on links capable of estimating link travel characteristics on adjacent links?

When this hypothesis was tested, the results were inconclusive. For one of the links, there was a reasonably high correlation (although not as high as when the detection was on the link in question). However, for the other, the correlation was not as high. It is doubtful that detectors on one link would be able to provide consistent data on adjacent links.

REFERENCES


TABLE 2  Best Detector Location For Predicting Link Travel Characteristic

<table>
<thead>
<tr>
<th>Detector Output Variable</th>
<th>Predicted Link Travel Characteristic</th>
<th>Link</th>
<th>Best Detector Location</th>
<th>R-Squared</th>
<th>P-Value</th>
<th>Residuals?</th>
</tr>
</thead>
<tbody>
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<td>Volume</td>
<td>Link Travel Time</td>
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<td>A</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>A</td>
<td>99.5</td>
<td>0.000</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>Link Travel Speed</td>
<td>1</td>
<td>A</td>
<td>97.8</td>
<td>0.003</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>A</td>
<td>99.3</td>
<td>0.001</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>Approoch Delay</td>
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<td>96.9</td>
<td>0.005</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>A</td>
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<td>0.001</td>
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</tr>
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<td>Sum of Occupancies</td>
<td>Link Travel Time</td>
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<td>0.001</td>
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<tr>
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<td></td>
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<td>A</td>
<td>99.7</td>
<td>0.000</td>
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</tr>
<tr>
<td></td>
<td>Link Travel Speed</td>
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<td>A</td>
<td>96.3</td>
<td>0.007</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
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<td>99.4</td>
<td>0.001</td>
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</tr>
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<td></td>
<td>Approach Delay</td>
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<td>A</td>
<td>96.4</td>
<td>0.007</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
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<td>99.0</td>
<td>0.001</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>D</td>
<td>98.4</td>
<td>0.002</td>
<td>Yes</td>
</tr>
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<td></td>
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<td>D</td>
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<td>Average of Occupancies</td>
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</tr>
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<td></td>
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<td>Speed</td>
<td>Link Travel Time</td>
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</tr>
<tr>
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<td>0.001</td>
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<tr>
<td></td>
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<td>4</td>
<td>C</td>
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<td>Yes</td>
</tr>
</tbody>
</table>

Note: In all cases the regression results are for quadratic equations.
Cross-Tensioned Concrete Pavement: An Alternative Modern PCCP Design

JEFFREY HANCOCK AND MUSTAQUE HOSSAIN

The rapid deterioration of Portland Cement Concrete Pavement (PCCP) is most often due to intrusion of water into the pavement and foundation layers through transverse joints and cracks. To minimize this problem, transverse joints are sealed with a flexible sealant that allows the joint to open and close while keeping water out. However, water does eventually get in and deterioration begins due to freezing and thawing, erosion of the subbase, etc. One possible way to solve this problem is to eliminate all transverse joints and to control the cracking tendency of the pavement by applying an external force in the form of post-tensioning. The advancement of post-tensioning products and procedures over the last twenty years has made this a fairly simple and inexpensive procedure. A reduction in slab thickness and elimination of sawing and sealing transverse joints can offset the cost of the post-tensioning hardware and process. The limited success of post-tensioning PCCP in the past was due to the design of longitudinal post-tensioning within the slabs. Longitudinally post-tensioning PCCP requires the construction of gap slabs where the actual post-tensioning work takes place. The gap slabs tend to deteriorate very fast. The need for gap slabs can be eliminated by cross-tensioning PCCP. This procedure may not only eliminate the need for any transverse joints, but will also resist the tendency of the pavement to crack in any direction. Key words: pavement, PCCP, gap slab, cross-tensioning, post-tensioning.

INTRODUCTION

The rapid deterioration of Portland Cement Concrete Pavement (PCCP) is most often caused by intrusion of water into the pavement layers. The water usually infiltrates through transverse joints and cracks. During construction process and routine maintenance, the transverse joints and cracks are typically sealed with a flexible joint sealant. The sealant keeps the majority of water out, however, some infiltration still happens. Concrete pavement deterioration then occurs due to freezing and thawing, erosion of subbase, etc.

Elimination of transverse joints and cracks is one solution to this common problem of PCCP. Pre-stressing PCCP could eliminate joints and cracks. By applying an external force in the form of post-tensioning, theoretically all joints in the pavement can be eliminated, and no cracking should happen. Extended pavement life may also be expected due to pre-stressing of PCCPs. The potential elimination of all transverse joints will provide a smooth and comfortable travelling surface, lower maintenance cost, and increased load-carrying capacity of the pavements.

Pre-stressing is a broad term that covers many different types of applied external forces to PCCP. The types include post-stressed, pre-tensioned, and post-tensioned. Post-stressing is accomplished by applying an external force to the ends of the slabs and is done without wires, strands, or cables. Pre-tensioning is done before the concrete is cast. Once cast, strands, cables, or wires are used to apply compressive forces. The force is transferred to the PCCP through surface friction. Post-tensioning, on the other hand, is applied after the concrete has cured. The post-tensioning force is applied through unbonded strands to the ends of the pavement.

The aforementioned procedures are generally referred to as “pre-stressing” (f) in generic term.

Post-tensioning of PCCP is not a new concept. It has been tried all over the world. The first construction of a post-tensioned pavement was at the Orly Airport, Paris, in 1946. The Europeans have long since advocated the use of post-tensioning in airport pavements. Europe has also taken the lead in the application of post-tensioning to highway pavements. During the 40s, 50s, and 60s, the Europeans constructed thirty post-tensioned pavements. During the same time, the United States constructed only six airport pavements that were post-tensioned. Finally, in the early 70s three highway pavements and an access road at Dulles International Airport were constructed. Since the 70s only one post-tensioned highway, near Phoenix, has been built by the Arizona Department of Transportation (ADOT) (2).

THEORY

Post-tensioned pavement design is unlike that of any other PCCPs. Conventional PCCP design is based on the modulus of rupture of concrete and does not take advantage of the high compressive strength of concrete. Post-tensioned concrete pavement increases the stress range during flexure of concrete. Resulting advantages are the following: (1) absence of cracks from the road surface, (2) reduced cost through a reduction in slab thickness and elimination of joint construction and maintenance, and (3) increase in load carrying capacity (3).

The fundamental formula for design of pre-stressed concrete pavements is (3):

\[ f_t + f_p \geq f_{\Delta t} + f_F + f_L \]

where,

- \( f_t \) = allowable concrete flexural stress
- \( f_p \) = modulus of rupture/factor of safety;
- \( f_{\Delta t} \) = compressive stress in concrete due to post-tensioning;
- \( f_F \) = curling stress due to difference in temperature between top and bottom surfaces of the concrete slab;
- \( f_L \) = load carrying capacity of the pavement.

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creep does level off after 300 to 400 days of load application.

The allowable concrete flexural stress may be taken as high as 80 to 100 percent of the modulus of rupture of concrete. Introduction of the compressive stress in the concrete pavement changes the failure criterion from a bottom tension crack to a top circular crack. The failure load is at least twice the load that produces the first bottom crack. This allows the designer to choose a factor of safety between 1 and 1.25 for $f_s$ (3).

Allowable stresses in the post-tensioning strands, $f_s$, after all losses, should not exceed 80 percent of the ultimate strength, $f$ of the post-tensioning steel. A typical post-tensioning monostand will be a 0.5 in (12.5 mm) diameter strand, made with six wires twisted around one wire. The ultimate strength of post-tensioning strands is 270 ksi (1,862 MPa) (4).

The curling stresses in concrete can be calculated from (3):

$$ f_m = \frac{\alpha E_c \Delta t}{2(1-\nu)} $$

where,

- $\alpha$ = coefficient of thermal expansion (6X10^-6 in/ in/°F, 3.5X10^-6 mm/mm/°C);
- $\Delta t = \text{temperature gradient (3°F/in, 0.63°C/mm)}$;
- $h = \text{slab thickness in inches or millimeters}$;
- $E_c = \text{static modulus of elasticity of concrete in psi or N/mm}^2$; and
- $\nu = \text{Poisson ratio for concrete (0.15)}$.

Curling stresses in post-tensioned PCCP could be caused by the warping of a slab due to creep or due to differences in the temperature or moisture content in the zones adjacent to its opposite faces. Usually concrete curls when one face is warmer or cooler than the other face. The warmer face of the concrete wants to expand while the cooler face will not. This causes the slab to curl either up or down depending on which is face is warmer. If the warmer face of the concrete is on top then the slab tends to curl down on the ends, however, if the opposite is true, the slab will tend to curl up on the ends (5). These changes in stress condition in the concrete slab can have adverse effects on the stresses within the strands. For example, suppose we have a slab where the stressed post-tensioning strands are in the lower half of the slab. When the lower face of the slab is cooler than the top face, the slab will try to contract on the bottom and expand at the top. The contraction at the bottom will cause some relaxation in the strands, hence the strands will loose some of their stresses. As a result, cracks may develop (3).

Fluctuations in stresses due to seasons can also cause adverse stress effects in concrete pavement. In Belgium, an 11,000 ft (3,353 m) pavement slab was post-stressed with screw jacks. Engineers tried for four years to maintain the stress in the concrete by frequently adjusting the screw jacks. The results of their attempt showed fluctuations in prestress that ranged from nearly 0 to 1,900 psi (13.1 MPa). If the total slab length were smaller, the adverse stresses caused by temperature differentials would not have fluctuated so drastically (2).

Creep also has adverse effects on stresses. Creep is the change in length of a pavement slab under a constant stress. The sustained post-tensioning force in the pavement results in creep of the concrete. Creep has similar effect on the prestressing force as curling, however, creep does not fluctuate as much as the curling stress. Higher compressive strengths of concrete produce less creep. Furthermore, creep does level off after 300 to 400 days of load application.

This allows the designer to design taking the creep effects of concrete into account.

The friction between the subgrade and the PCCP also produces a stress that is most significant in the middle portion of the slab. For a slab that is less than 700 ft (220 m) in length, the frictional stress ($f_s$) between the sub-grade and PCCP can be estimated as (3):

$$ \max f_s = \frac{c \gamma L}{2 \times 144} $$

where,

- $c = \text{coefficient of sub-grade friction}$,
- $L = \text{total longitudinal length between ends of strands}$, and
- $\gamma = \text{unit weight of concrete (145 lbs./ cubic ft or 2323 kg/cubic meter)}$.

If the reactive force applied by the subgrade friction is too much, it could effect the stresses applied by the strands. Consider, for example, that a slab is supported on the subgrade at two different points between the ends of a post-tensioning strand. When the strand is pulled to tension, a compressive force is exerted on the concrete slab. However, the region between the two pins experiences no compressive force. This is the problem that subgrade friction imposes on post-tensioned PCCP. The problem becomes quite significant as the slab becomes longer. For this reason, the frictional stress between the slab and the subgrade must be considered in design (6).

The coefficient of sub-grade friction is substantial with a value of 0.2 to 1.5 for slabs resting on a sand or granular subbase. For design purposes, the coefficient of subgrade friction can be taken as 0.5 to 0.8 (6). Most historical designs of post-tensioned pavement incorporate two layers of 6 mil (0.15 mm) polyethylene sheeting which have a coefficient of friction of less than 0.2. However, in the ADOT post-tensioned PCCP project, it was discovered that almost no difference exists in friction between the slab and the subgrade on either 1 or 2 layers of polyethylene sheeting (7). In either case, it is still recommended to use a conservative value of 0.5 for the coefficient of friction in design. This gives the designer reasonable assurance that the value can be achieved in the field (6).

Conventional PCCP design consider traffic load as the primary factor affecting the required pavement thickness. For post-tensioned PCCP, traffic loads induce tensile stresses at the bottom of the pavement. This stress can be calculated by the Westergaard’s formula. The edge loading of PCCP is always the controlling criteria. The edge is where the stress in the slab will always be the greatest under a tire load. The stresses in the pavement induced by successive traffic loads can lead to elastic deformation of the slab to the point where the maximum moment beneath the loaded area exceeds the sum of the flexural strength of the concrete and the induced force. At this point, a crack forms a hinge under the load, repeated load applications cause a moment in the slab some distance away from the loaded area. If traffic repetitions continue, tensile cracks may form at the top of the pavement. When loading is increased beyond this point, the load will eventually punch through the slab. For this reason, it is extremely important to consider traffic load in post-tensioned PCCP (6).

Westergaard’s formula for stresses due to edge loading of PCCP is given as (5):

$$ f_L = \frac{0.803 E c}{h} \left[ 4 \log \left( \frac{l}{a} \right) + 0.66 \frac{a}{l} - 0.034 \right] $$

where,

- $f_L = \text{maximum edge stress under load}$;
- $l = \text{radius of relative stiffness}$;
tensioned in the longitudinal direction. The only difference is the pavement is done in much the same way as that of a PCCP post-nonjointed, uncracked concrete. The procedure for designing such a pavement is done in much the same way as that of a PCCP post-tensioned in the longitudinal direction. The only difference is the tensile strength of concrete should be neglected in all axial calculations. The tensile strength of concrete is highly variable and is usually considered to be only 10 to 15 percent of the compressive strength.

The thickness of concrete used in post-tensioned PCCP is less than conventional PCCP thickness. Pavements as thin as 3.5 in. (90 mm) have been developed for use with longitudinally post-tensioned pavements. However, problems associated with coverage and uniform paving make this an unfavorable choice on most highway pavements. It is recommended by ACI that a pavement thickness of at least 65% of the thickness of an alternative plain concrete pavement be used. This allows for appropriate coverage and variations in construction. For most cases, a pavement thickness of 6 in. (150 mm) is sufficient for coverage and construction tolerances.

Special consideration needs to be given to the stressing operation. The strands cannot be tensioned to design values until the concrete has gained sufficient strength. ADOT tensioned its longitudinally post-tensioned PCCP in three stages. Since cracking occurs from

\[
\begin{align*}
E & = \text{modulus of elasticity of concrete} \\
& (\text{assume} \ 4,000,000 \text{ psi}); \\
k & = \text{modulus of subgrade reaction} \\
& (\text{assume} \ 100 \text{ pci}); \\
P & = \text{concentrated load, lbs.; and} \\
a & = \text{contact radius} \\
& = \text{width of the pavement.}
\end{align*}
\]

For a typical maximum load of 22, 000 lbs (97,860 N) per axle on duals, P can be estimated as 5,500 lbs (24.5 kN) and the tire pressure can be estimated as 100 psi (0.69 Pa).

Deflections also occur in the slab. Excessive deflections in PCCP are most often caused by softer foundation support. Post-tensioned pavement, however, can span the softer spots in the subgrade. This happens due to an increase in flexural strength of the concrete due to the post-tensioning force, which results in controlled deflection of the post-tensioned PCCP under repeated loads.

**TYPICAL DESIGN**

In most post-tensioning PCCP projects, the post-tensioning had been done in the longitudinal direction of the PCCP. This does have its advantages and disadvantages. One of the advantages of this type of design is the simplified procedure of placing the strands in the pavement. Most slipform pavers can easily be adapted to accommodate the insertion of the post-tensioning strands. ADOT used this approach when it post-tensioned part of the Superstition Freeway near Phoenix. In longitudinal post-tensioning, a direct force perpendicular to the weak plane of the concrete, the transverse plane, is also applied. This reduces joint spacing and cracks in the concrete.

Longitudinal post-tensioning does have its disadvantages. First, slabs can only be designed to be so long before post-tension losses, such as, those induced by the frictional stresses between the subgrade and the PCCP, become so great that they cannot be overcome unless more strands are added. For this reason, most post-tensioned slabs tensioned in the longitudinal direction are limited to less than 500 ft. (152 m) in length. Second, longitudinally post-tensioning PCCP requires the work areas, known as gap slabs, be constructed between two post-tensioned pavements. These areas are where the actual tensioning of the strands takes place and the hardware is placed. Contractors must have enough space in the gap slab to apply the tensioning force without bumping into the slab behind them. Gap slabs range in length from 6 to 10 feet (1.80 to 3.00 m). These gap slabs have been found to deteriorate more rapidly, affecting the ride quality. The elimination of gap slabs should increase the riding comfort on post-tensioned PCCP. Eliminating gap slabs also eliminates two joints where water infiltration is likely to occur. Many different joints have been developed to control the inflow of water at the transverse joints on either side of the gap slab. Most are either too difficult to install or are not at all a cost-effective solution.

**PROPOSED DESIGN**

Cross-tensioning PCCP may give the designer an option to construct nonjointed, uncracked concrete. The procedure for designing such a pavement is done in much the same way as that of a PCCP post-tensioned in the longitudinal direction. The only difference is the interpretation of the frictional losses between the PCCP and the subgrade. In longitudinal post-tensioning design, the designer considers the pavement slab as having two ends that result in transverse joints. The length of the pavement between the two joints is the slab length that is subjected to the frictional force between the PCCP and the subgrade. In cross-tensioning, the designer post-tensions the pavement in diagonal directions from both sides. By cross-tensioning, the PCCP cannot expand in any direction. This allows for the design of a nonjointed pavement of virtually infinite length.

The angle at which the cross-tensioning should be applied varies between 30 and 45 degrees to the longitudinal direction of the pavement. This allows the majority of the stress force in the strands to act against the weak plane of the concrete, the transverse plane. When the 30 to 45-degree specifications also give the concrete less capability of cracking in the longitudinal direction, as did the ADOT longitudinally post-tensioned PCCP. It should be noted the smaller angle of post-tensioning results in less steel strands.

Strand location in the slab is most important. Historically, the strands of post-tensioning steel have been placed 0.5 in. (12.5 mm) below the mid-depth of the slab. At this depth, the strands are able to carry very well the loads induced on them by truck traffic. Lowering the strands to 0.5 in. (12.5 mm) below mid-depth causes the slab to have an induced negative moment (3). When a truck passes over the pavement, the induced negative moment is cancelled by the moment generated by the axle load. Placing the strands at or above mid-depth will only add to the load already present due to the axle load. This should never be done.

Strand placement is also very significant at the edge of the pavement. It is important that the strands tensioned in different directions, on the same side of the pavement and very near each other, be placed so they each apply a compressive force between them. This requires that the strands be crossed very close to the pavement edge. For example, assume a pavement is stressed with the 45-degree post-tensioned strands. Consider now that the strands are laid out such that two strands, perpendicular to each other, are very close, but not crossing at the pavement edge. This will cause a tensile stress in the concrete between the two strand ends. If the tensile stress is too great, the concrete may develop a transverse crack. Now consider the same two strands as crossing very near the pavement edge. The only stress that will exist between the two strands in this case is a compressive stress. It is well known that concrete is very weak in tension. In fact, the American Concrete Institute (ACI) code requires that the tensile strength of concrete should be neglected in all axial calculations. The tensile strength of concrete is highly variable and is usually considered to be only 10 to 15 percent of the compressive strength.
volume change, thermal gradient, and subgrade restraint, it is imperative to apply the first stage tensioning to the cables at the earliest practical time, usually within 24 hours. Some fine cracks may form before initial jacking; however, they should close upon application of the jacking force. The second jacking should be done within 24 hours of the previous one. Lastly, the final jacking should be done when the concrete strength has reached at least 3,000 psi (21 MPa), independent of time. It is imperative not to jack the strands beyond the strength of the concrete. Jacking forces should be determined considering the size and thickness of the bearing plate end anchors and minimum concrete strength necessary to withstand the applied force. It may be necessary to increase the bearing plate size to distribute the high stresses at the slab edges (7).

CONCLUSIONS AND RECOMMENDATIONS

Post-tensioned PCCP of the cross-tensioned variety may be a suitable solution to the problems associated with conventional PCCP. Post-tensioned PCCP can reduce pavement thickness, reduce pavement deflection, allow for unjointed design, and increase the load-carrying capacity of PCCP. As with any new design, this needs to be proven. It is, however, possible that cross-tensioned PCCP could be everlasting pavement for the 21st Century.

Further studies and actual field tests of cross-tensioned PCCP should be done before any attempt is made to apply the theory to an actual PCCP construction. The most extensive research should be done to check the stress condition in the areas where the strands cross each other near the pavement edge. At this location, little is known about the behavior of the concrete. Studies should also be done on the behavior of the concrete at any location where strands cross each other. Furthermore, researchers should study the effects of applying a skew force to the strands. This will cause some side force on the strand near the pavement edge. Additionally, the construction of cross-tensioned pavements will need to be reviewed. Some new and innovative construction techniques may need to be developed for installation of the crossed strands.

REFERENCES

Matching Load Transfer to Traffic Needs

J.K. Cable and L.I. Wosoba

Current pavement design in Iowa calls for the inclusion of load transfer dowels in transverse joints in both state and local pavements. The dowels have been included to protect the pavement against faulting of the joints and other forms of distress resulting from erosion of the soils from beneath the joints. Faulting has been found to be present mostly at the outer edges of the driving lane. Iowa Highway Research Board Project TR-420 is directed at the evaluation of placing alternative numbers of dowels in the transverse joints of the pavement. A rural and an urban pavement were selected for the test sites on county highways near Creston, Iowa. The sites include subsections containing zero dowels in the transfer joint, three or four dowels in the outer wheel path only, and a full basket of dowels across the joint. This paper will discuss the results of deflection testing in both wheel paths in both pavement directions on the rural and urban sections. Fault measurements, joint opening widths, and visual distress surveys have been conducted twice per year on each of the projects. The construction projects are now one year old and we can begin to evaluate the response to load in each case. Key words: load transfer, pavements, dowels, whitetopping, portland cement concrete pavements.

STUDY OBJECTIVES

The research conducted under Iowa Highway Research Board Project TR-420 contained the following objectives aimed at answering the dowel location and number question:

- Evaluation of dowel location and number performance
- Evaluation of construction and installation procedures

TEST SITE LAYOUT AND VARIABLES

Two sites in Union County, near Creston, were chosen for this research. The first was located at the southeast corner of Creston and extended south from US 34 along Union County P33 (project STP-5-88(25)—5E-88) some 6.4 miles. It is referred to as the Rural test site. In this case, the pavement was constructed on an existing granular base.

The second or Urban project was located along the north and east boundaries of Creston on Union County H33 and P33 (project L-P-298—73-88). This project is approximately 1.5 miles in length. This project was constructed on an existing flexible base constructed over time to an average depth of six inches using cold mix asphaltic materials.

The variables selected for this research involved the number of dowels to be placed in the transverse joints. The options included omission of all dowels, installation of three or four dowel combinations in the outer wheel path only, and the default value of dowel across the entire transverse joint. Dowel spacing in each of the study areas was maintained at 12 inches. The joints in each project were constructed on a 6:1 skew as shown in Figure 1. In the case of the Rural project, the research team identified 20 joints each in succession containing no dowels, three dowels, four dowels, and a full dowel basket across the joint. This pattern was employed in both lanes. For the Urban project, the same plan was implemented on 10 joints in each direction with the exception of the section with no dowels. In this case, one lane contains no dowels and one lane contains full dowel baskets. Table 1 and 2 indicate the location of the various test sections on each project.

TEST SITE CONSTRUCTION AND DATA COLLECTION

The projects were paved in August of 1998 by the Fred Carlson Co. Full width paving was employed on both projects. In the case of the Rural project, the base was trimmed immediately in front of the paving machine, and the concrete was placed on the granular base. The concrete was placed directly on the existing flexible pavement in the Urban project. In both cases, the dowel basket assemblies were placed immediately in front of the paving machine.
Data being collected on each of the projects includes biannual visual distress surveys, joint faulting and opening measurements, and deflection testing of each joint by direction and both wheelpaths. Faulting is measured using a Georgia Fault Meter, and the joint openings are measured using a digital caliper. Deflection testing is accomplished by use of the Iowa DOT Roadrater. The distress surveys are conducted in accordance with the methods described in the Strategic Highway Research Program (SHRP) Pavement Distress Manual (1). Distress surveys were conducted on the Urban Project prior to construction. After construction, each of the data collection activities were conducted on both projects prior to opening to traffic.

**TEST RESULTS TO DATE**

The evaluation period for TR-420 is expected to last for five years. The initial results of the data collection have not resulted in conclusive answers to the research questions. The deflection measurements have been analyzed for the first full year and to date do not show any significant differences between test sections. The same can be said about the area bounded by the deflection basin, considering all the sensors on the Roadrater. Joint opening widths and faulting measurements are also inconclusive at this time. Each of the measurements was somewhat erratic at the time of construction and has stabilized over the first year of operation. There are no significant visual distresses other than minor spalling along the edges, as a result of shoulder construction equipment, and one transverse crack at a rural intersection as a result of pavement jointing techniques employed.

**CONCLUSIONS**

The conclusions that can be drawn at this time are as follows:
- Conventional pavements can be constructed employing partial dowel baskets with no change in the paving machine or labor requirements.
- To date the reduction in dowels has caused no reduction in performance (load transfer, deflection, joint opening or faulting).

**REFERENCES**

Application of Advanced ITS Interfacing That Improves Maintenance Operational Effectiveness and Winter Safety in Rural Areas

Leland Smithson, Bill McCall, and Dennis A. Kroeger

In 1995, the state departments of transportation of Iowa, Michigan, and Minnesota formed a consortium to define and develop the next-generation highway maintenance vehicle that would utilize the latest maintenance operational technologies and interface with ITS. Focus groups revealed that while all maintenance operations could benefit from creating this new generation vehicle, ice and snow operations were the most complex and would benefit greatly from improvements in state-of-the-art vehicle navigation systems, onboard computer applications, and enhanced safety systems. This advanced technology highway maintenance vehicle functions as both operational truck and a mobile data-gathering platform. Sensors mounted on the vehicle record air and roadway surface temperature, roadway surface condition, and roadway surface friction characteristics. This information is Global Positioning Systems (GPS) correlated and used in maintenance operational decision making. The information will eventually be interfaced with the ITS technology in the Traffic Management and Information Service Provider Centers Subsystems of the National ITS Architecture. The advanced technology highway maintenance vehicle performs an important role in the U.S. Federal Highway Administration’s “Weather Information for Surface Transportation ITS Field Operational Test” being conducted by the FORETELL consortium.

BACKGROUND

The mission of a department of transportation is to provide its customers reliable transportation facilities that perform to their level of service expectations and to accomplish this in the most efficient and effective manner possible. This mission is particularly challenging to snow belt states during the perils of a winter season. Just-in-time goods deliveries, a key ingredient in any state’s economic vitality, places an ever-increasing importance on reliable year-round transportation. These increasing transportation demands are coming at a time when most states are being asked to downsize their maintenance operations work force. The application of advanced snow and ice control technologies and their integration with ITS offer excellent potential for increasing operational efficiency and effectiveness as well as improving winter mobility and driver safety.

In recognition of the potential that exists when utilizing advanced methods and ITS technologies for highway maintenance activities, a four-phase study, shown in Figure 1, was initiated to define the desired vehicle and equipment capabilities for the next generation highway vehicle, develop and evaluate prototype vehicles, conduct benefit/cost analysis, and produce maintenance vehicles for fleet applications. The initial focus is on maintenance operations that are under public observation the most. Agency operations and ITS surveys have shown that safety and winter mobility rank high in customers concerns and expectations. Winter snow and ice control operations therefore are receiving first consideration for technology applications in developing the next generation highway maintenance vehicle.

Foundation Statements:
1. “The solutions must be selected and recommended based on a benefit/cost analysis and a reasonably short time to implementation.”
2. “The application of solutions must be described in terms that related to improving service to customers.”

FIGURE 1 Four-phase study flowchart
FOUR-PHASE RESEARCH IN PROCESS

Phase I

The objective of Phase I was to develop the functionality of the concept vehicle will provide and to enlist private sector partners to provide the functionality. This phase began with a literature review of materials related to winter highway maintenance activities. One hundred and five articles were collected which pertained to state-of-the-art equipment, technologies, and research related to winter highway maintenance activities.

The ideal capabilities of a winter maintenance vehicle were identified through focus group activities. Five focus groups were formed. The focus groups included representation from equipment operators and managers, mechanics, resident and central maintenance office engineers, area supervisors, law enforcement agencies, and emergency responders. Focus group meetings were held in the three consortium states generating more than 600 ideas. These ideas were later combined and organized into a list of 181 desired capabilities for the highway maintenance concept vehicle. The final prototype design for the three prototype vehicles provided the following desired capabilities that resulted from the focus group activities:

- Sense roadway surface temperatures
- Sense roadway surface friction conditions
- Record and download vehicle activities
- Improve fuel economy
- Provide adequate horsepower for the vehicle
- Carry and distribute multiple types of materials
- Provide removable salt/salt brine dispensing system
- Provide back-up sensors/monitors
- Provide back-up sensors/monitors
- Private sector equipment and technology providers were introduced to the study and asked to join in the effort. These private partners committed to providing equipment and expertise for the duration of the study. Phase I is complete and a more detailed discussion can be found in Concept Highway Maintenance Vehicle, Final Report Phase One, dated April 1997, Iowa State University, Ames, Iowa. The report is also on the Iowa State University’s Center for Transportation Research and Education web site at <http://www.ctre.iastate.edu/Research/conceptv/conceptv.htm>.

Phase II

The objectives of Phase II were to build three prototype concept vehicles, integrating the subsystems into a working system, conduct proof of concept, and perform for field evaluations of three prototype vehicles in Phase III. The mission of a department of transportation is to provide its customers reliable transportation facilities that perform to their level of service expectations and to accomplish this in the most efficient and effective manner possible. This mission is particularly challenging to snow belt states during the perils of a winter season. Just-in-time goods deliveries, a key ingredient in any state’s economic vitality, places an ever-increasing importance on reliable year-round transportation. These increasing transportation demands are coming at a time when most states are being asked to downsize their maintenance operations workforce. The application of advanced snow and ice control technologies and their integration with ITS offer excellent potential for increasing operational efficiency and effectiveness as well as improving winter mobility and driver safety. Proof of concept was conducted for each of the functional areas integrated into the prototype vehicles. Proof of concept for Phase II was defined as conducting “end-to-end” processing, observing the success of the “end-to-end” processing, and observing if the data was reasonable. Proof of concept is not a rigorous statistically valid field test. A data collection and observation plan was developed to conduct proof of concept while operating the prototype vehicles during the winter of 1998-1999.

In addition, telephone interviews were conducted with the prototype vehicle operators to ascertain equipment performance. The interviews and documentation of equipment performance led to guidelines for the desired equipment capabilities for the Phase III prototype vehicle. Phase II is complete and the final report, Concept Highway Maintenance Vehicle, Final Report Phase II, is on the Iowa State University’s Center for Transportation Research and Education web site at <http://www.ctre.iastate.edu/Research/conceptv/conceptv.htm>.

Phase III

The general objectives of Phase III to be achieved in 1999-2000 are to perform proof of concept on newly discovered technologies, establish the functionality of each technology to be implemented, conduct a benefit/cost analysis for each technology, estimate the time to implementation, conduct field evaluation, produce data flow and decision process maps to integrate the concept vehicle functionality in management and ITS systems and develop draft vehicle specifications for each consortium state.

Phase III will answer these questions:

- Which technologies should be implemented?
- What are the benefit/costs of each technology?
- What is the expected time to implementation?

Sensing roadway surface conditions is being attempted by Norsemeter of Norway using a device called Saltar. The Saltar design is an outgrowth of the evaluation done in Phase II and has been tested at Wallops Island, Virginia and North Bay, Ontario. Both tests are sponsored by NASA and attended by several manufacturers of surface friction measuring devices. The Saltar device did function as expected. The report will be placed on the concept vehicle web page.

Benefit–cost analysis is currently being conducted on the pavement surface temperature measuring technology. Benefits will be based on estimating the difference between materials distributed knowing the pavement surface temperature at the vehicle’s location and materials distributed based on the pavement temperature measured at a remote road weather information system (RWIS) site. Data are being collected based on interviews with field staff and collected from databases generated by the vehicle and the RWIS site. Analysis is currently underway.

Phase III also includes conducting proof of concept evaluation on a pavement surface freezing point sensing system. The system, supplied by Enator of Sweden, is currently undergoing benching testing conducted by CTRE.

Phase IV

The objectives of Phase IV are to:

- Equip ten vehicles per state with selected advanced technologies.
- Conduct field evaluation.
INTERFACING WITH ITS

National ITS Architecture

The Iowa Department of Transportation envisions the concept vehicle functionality fitting into the National ITS Architecture Subsystem and Communications architecture very smoothly. Figure 2 illustrates the placement of the functionality.

Road and Weather Model Interface

As part of the "Weather Information for Surface Transportation ITS Field Operational Test," the Iowa prototype maintenance vehicle provided air and pavement temperatures to the FORETELL Consortium to assist in the calibration of a new road and weather forecast model. The vehicle operates as a mobile environmental sensor station gathering real-time pavement thermal profiles and air temperature data for input to the FORETELL microscale models. This interface is

FIGURE 2 National ITS architecture subsystem and communications vision for the Iowa DOT highway maintenance concept vehicle
depicted in Figure 3. It is envisioned that the ten advanced technology maintenance vehicles in Phase IV of this research will serve as FORETELL's mobile platforms using NTCIP 'ESS' protocol standards to radio air temperatures, wind speeds, pavement data and maintenance operations reports in real time to FORETELL ITS service centers. These ITS service centers will provide the interface between ITS, and ITS users, allowing progressive deployment of weather, roadway, and other ITS applications throughout the service center area.

CONCLUSIONS

The four-phase research project to develop a new generation, advanced technology highway maintenance vehicle began in 1995. The vision was to develop a concept vehicle that would support equipment operators and fleet managers in making more informed and cost-effective decisions based on using emerging ITS technology. The approach was to bring technology applications from other industries to the concept vehicle. The customer was brought into the planning process at the very beginning and is one of the reasons the project has been successful in field-testing. Each of the three consortium states has built and operated an advanced technology highway maintenance vehicle in its daily maintenance operations for three years. The advanced technology applications have withstood the severity of snow and ice control operations for two winters with only minor problems. Each vehicle and its advanced concept technologies have passed proof of concept tests. Each technology is now being evaluated to make sure what benefits have been realized and calculate their respective benefit/cost ratio. Emerging technologies are also being tested on the concept vehicle. First generation concept technologies are being redesigned to improve their reliability and reduce complexity and cost. For example, a roadway friction-measuring device has been redesigned to make it smaller, less complex, and more durable, and the cost has been reduced by 65%. Reduced cost is especially important because each state will need several hundred friction measuring units to adequately meet the need of rural ITS to accurately determine and predict the winter condition of road surfaces and its impact on braking and driving traction.

Field operators and managers feel the new technology has made their efforts more efficient and effective. The information these vehicles provide to the ITS community is an incidental benefit to the main snow and ice control mission and is used by both the department of transportation in their operations management and the ITS service centers.

As new technologies emerge, they will be evaluated and tested using the model developed for this research project.

FIGURE 3 Work in progress, Phase III concept vehicle, Iowa network diagram
Vehicle Safety: Transit Buses and Garbage Trucks

SUE McNEIL

This paper presents the results from two projects undertaken at Carnegie Mellon University. The first explores the automation of solid waste collection vehicles. The second is developing side collision warning systems for buses and is an ongoing project. The application of highway automation technologies, particularly vehicle based sensors, software, hardware, and control, to solid waste collection vehicles is timely. Such vehicle automation is likely to be beneficial to workers, drivers, equipment owners and operators, and the general public as it does not replace operators but focuses on improved safety. The end result is fewer accidents. Interviews with driver and equipment manufacturers and anecdotal evidence are used to explore the opportunities, benefits, and costs related to automation, particularly for automated pick-up. Research and development related to side collision warning systems has been directed at light vehicles and long-haul trucks. In this paper, we present evidence that supports our hypothesis that the side collision warning systems for transit buses are very different, as they must focus on detecting pedestrians. Data analysis, driver interviews, a review of relevant literature, and an evaluation of existing systems are presented. Based on the results of this preliminary research, a plan for developing a performance specification for a side collision warning systems for buses is presented. The relevance of this technology to solid waste collection vehicles is also explored. Preliminary economic analysis suggests that side collisions warning systems and automated pick-up are worthwhile investments. Key words: vehicle safety, side collision warning systems, transit buses.

INTRODUCTION

Both the solid waste collection industry and the transit industry have been through an era of unprecedented change brought on by increasing growth of urban areas, continuing urban sprawl, changes in land use, environmental regulation and economic pressures. In both cases, the process, pickup, and transportation of passengers or solid waste is inherently dangerous as drivers maneuver unwieldy vehicles in constrained spaces with both stationary and mobile objects, pedestrians, and other vehicles. In the case of solid waste collection vehicles, the vehicle itself is equipped with devices such as compactors and lifters that add risk to operators, objects, and the general public. In the case of transit buses, the driver has to be particularly cognizant of pedestrians, as they run for a bus they think they have missed and dart out to cross in front of the bus.

New technologies offer opportunities to make these processes inherently safer for both drivers and the general public. This paper presents the results from two projects undertaken at Carnegie Mellon University. The first explores the automation of solid waste collection vehicles. The second is developing side collision warning systems (SCWS) for buses. The paper reviews some of the background research in this area and describes the types of safety improvements under consideration. Finally, the paper presents a preliminary cost-benefit analysis.

BACKGROUND

Several previous and ongoing studies provide evidence of opportunities for safety improvements for buses and solid waste collection vehicles. These studies provide a foundation for this paper and are briefly described in this section.

From 1995 to 1997 the National Automated Highway Systems Consortium (NAHSC) worked with the US Department of Transportation to conceptualize the automated highway (1) for hands-off, feet-off, computer-controlled driving to improve safety and throughput. The project culminated in a proof of technical feasibility in August 1997 in San Diego California. “Demo 97” involved several technologies (2) that focus on safety improvements. These technologies are particularly interesting in this context as they also address many of the safety issues of concern to solid waste collection and transit bus drivers related to pedestrians and children in the vicinity of a moving vehicle and to obstacles. Another important part of the NAHSC was the exploration of costs and benefits and the issues related to operation and maintenance. A spreadsheet tool was developed and modified to quantify safety, fuel, emissions, and other benefits of automation (3). An important part of this work was the recognition that different stakeholders have different perspectives and that “one size does not fit all.” These concepts also hold true for transit buses and solid waste collection vehicles.

While funding for the NAHSC was withdrawn for various reasons (1), the US DOT has developed a new program, the Intelligent Vehicle Initiative (IVI). The IVI program is a part of the Intelligent Transportation Systems (ITS) Program and again focuses on safety and efficiency of motor vehicle operations. Specific activities include vision enhancement, vehicle stability warning, driver condition warnings, automated transactions, obstacle and pedestrian detection, tight maneuver/precision docking, and fully-automated control in specific situations. For many of these activities, there are also significant parallels between solid waste collection vehicles and transit buses as they both operate in constrained environments and are concerned with pedestrians (4).

One area of particular interest to buses and solid waste collection vehicles is side collision warning systems. Much of the research and development work related to side collision warning systems to date has focused on light vehicles and long-haul trucks. To begin to address the particular needs of transit, an evaluation of commercially available side collision warning systems was conducted using ITS...
America data and Internet searches to identify vendors (5). Sensors include infrared, radar, and sonar. Two to ten sensors are required to provide coverage of the side of a bus.

Safety improvements for solid waste collection vehicles have largely focused on solid waste pickup rather than the driving task. Specifically, the use of a one-person operation known as a sideloader has received considerable attention. Using this automated method, the driver teleoperates an arm mounted between the cab and the hopper to grab, lift, empty, and return specially designed carts to the curb. Automated residential solid waste collection was developed in the 70s and has evolved to service around 300 million people, although it is not widely accepted (6). Semi-automated collection refers to the use of traditional rear loaders with hydraulic lifters, so the workers are not physically lifting bins. Experiences in the United States, Australia, and France and the range of equipment offered by various manufacturers are described in (7).

While there is widespread interest in improvements in vehicle safety in the transit and solid waste collection industry, this paper focuses on two particular improvements. The first is side collision warning systems for transit buses. The second is automated solid waste collection. We also discuss the relevance of the work on side collision warning systems for transit buses to solid waste collection vehicles.

**SIDE COLLISION WARNING SYSTEMS FOR TRANSIT BUSES**

In August 1998, the Pennsylvania Department of Transportation sponsored “Demo 98” at State College. Carnegie Mellon University demonstrated a commercially available side collision warning system fitted to a Port Authority Transit (PAT) bus. The bus has four sensors installed along the right side of the bus. The commercial sonar sensors, manufactured by EchoVisionÔ, were selected on the basis of availability and coverage. The sensors activate when the four-way hazard lights or the right-turn indicator are on. If an object is identified in the “field of view” of any one of the sensors, an audible warning signal is generated. A rear-looking system was also installed on the bus.

While only four evaluation sheets were received because of the limited opportunity for drivers to use the bus, these comments were insightful and sent a clear message. The questionnaire asked the drivers to identify when and where they drove the bus, to supply the weather conditions, and to provide comments and make suggestions. The biggest concern was that the system, as configured, was too sensitive, and the buzzer went off too frequently and too loudly. A second concern was that the sensors, as configured, only covered the right side of the bus, leaving the left side uncovered. (These responses give an inconsistent impression; the drivers did not completely like the system and yet still want it on both sides of the vehicle). It is clear that the drivers have to be helped by the system rather than annoyed, therefore system sensitivity and driver interface are crucial (8).

Subsequent interviews with Port Authority Transit drivers in Pittsburgh tell us something about the nature of crashes involving buses. The theme in these conversations is a major concern with preventing pedestrian accidents. Although there are relatively few pedestrian accidents, they are the most likely to cause serious injury. They are also among the hardest to prevent. Pedestrians are significantly harder to see than vehicles since they are smaller, move unpredictably, and can be found in areas close to the bus where the driver has limited visibility. To enable us to go beyond the anecdotal evidence, PAT has provided us with their database of liability claims. This database includes comprehensive data for claims and crashes since 1997. For the period of January 1997 to May 1999, the database includes over four thousand records. Incidents involving pedestrians are largely clustered in Downtown Pittsburgh and Oakland—an area with universities and hospitals and heavy pedestrian traffic.

Based on interviews, accident analysis, and experiences with the existing commercial system, the greatest challenges lie in developing a side collision warning system that reliably and effectively detects pedestrians, vehicles, and fixed objects.

**AUTOMATED PICKUP OF SOLID WASTE**

The solid waste collection and hauling industry is a complex $36 billion industry handling over 200 million tons of solid waste annually in the US (9). It has also undergone rapid change in the past decade as more municipalities are privatizing waste collection services, large firms acquire smaller firms, large companies merge (10), and competitiveness becomes a basic business philosophy in the industry. In addition, the regulatory environment has changed as the industry begins to better understand the environmental implications of waste disposal, public awareness of these issues has increased, and health and safety have become important factors in the working environment. At the same time, vocal groups of consumers demand recycling options, and the public at large wants to have as little interaction with their garbage as possible (11). Grigg (12) describes the problem succinctly as follows: “Collection is a distributed, difficult-to-control problem involving the cooperation of citizens and a high labor content.”

Vehicle operators, public and private service providers, and equipment manufacturers emphasized the importance of safety for workers and the general public in responses to interview questions focusing on experience with solid waste collection automation. Anecdotal evidence suggests that incidences involving pedestrians and children are of particular concern (7). The Fatal Accident Reporting System (FARS) indicates that in 1998 of the 37,081 fatal crashes that occurred in the United States, 118 involved garbage trucks (19).

The typical solid waste collection operation involves a driver and one or two runners. Automated vehicles require only a driver who rarely leaves the cab. Automation of solid waste pickup reduces the exposure of the driver and runners to other vehicles and injuries due to lifting heavy trash bags and bins and from dangerous objects in the trash. It requires purchase of specialized carts (wheeled bins) and trucks. Typical features on the vehicles include dual-drive, low-entry cabs, and reversing cameras. Most importantly, the introduction of automation requires an extensive consumer education program including what can be picked-up and how the cart must be positioned.
AN ENHANCED COLLISION WARNING SYSTEM

Many of the existing and potential safety systems for transit buses and solid waste collection vehicles act independently. In this study, we focus on side collision warning systems for transit buses and solid waste collection vehicles and automated pick-up of solid waste. Both transit bus and solid waste collection vehicle drivers must multitask. The bus driver, particularly in central city areas has to be alert to the actions of passengers, pedestrians, other drivers, and the presence of fixed objects as he or she maneuvers in constrained spaces. The solid waste collection vehicle driver is often in an “off-side” driving situation, and the vehicle operates in constrained environments with poles, parked vehicles, children, and other drivers believing that the truck will avoid them. An ongoing study of side collision warning systems for transit buses (8) points to the many elements that are common concerns of drivers of solid waste collection vehicles. In both cases, pedestrians are of significant concern to the drivers. Past studies have focused on cars and long haul trucks where this is not the significant issue. The ideal side collision warning system will be simple to operate, inexpensive, provide both visual and audio warnings, provide a high level of reliability in terms of detection, inform the driver of hazardous situations with sufficient time to take corrective action, and minimize false warnings and nuisance alarms (13).

In our study, we identify some important things to consider in order to design a system appropriate for the application. These important aspects are the following:

1. What should be detected—Pedestrians are the most important and the hardest to see. Vehicles to be detected include bicycles, cars, trucks, and motorcycles. Stationary obstacles should also be detected.
2. Where should they be detected—Detection should occur on both sides of the vehicle.
3. How should they be detected—Potential sensors include stereo vision, infrared cameras, infrared proximity sensors, sonar, radar, and ladar.

Based on our research to date and past studies (13), some general performance goals have been identified (8). Preliminary analysis of available data and driver interviews has placed additional emphasis on pedestrians. As the project proceeds we will use an iterative process to develop and evaluate performance specifications based on analysis of available crash data, past experiences, investigation of state-of-the-art technology, and sensor development. We are also working towards developing a system that is specific to transit buses. We will continue to explore its applicability to solid waste collection vehicles.

COSTS, BENEFITS AND OPPORTUNITIES

Side Collision Warning Systems for Buses

Even a preliminary analysis of the costs and benefits of side collision warning systems is premature at this point. However, an analysis of the break-even cost of a side collision warning system is useful as it provides an indication of the whether or not such a system may be feasible. The crude analysis presented here for transit buses is based on many assumptions. These include:

- The cost of the SCWS consists of an initial cost, and it has a life of three years.
- The only benefits derived from the SCWS are in the form of accidents saved and are measured in terms of the social costs of accidents in 1992 dollars, which are assumed to be $880,000 for a fatality, $29,500 for a injury, and $6,500 for a property damage (14).
- The discount rate is 6%.

The magnitude of the saving is highly dependent on the effectiveness of the system. Performance specifications for SCWS typically require 97% detection of potentially dangerous situations (9). However, once a situation has been detected, the driver must recognize and respond. More important than the reliability of detection is the time available for the driver to respond. Eaton manufactures a commercially available SCWS for trucks. The system claims to “reduce preventable accidents by 73%” (20). In this analysis, we distinguish between preventing crashes and reducing their severity. Table 1 summarizes the values assumed for savings from prevention and reduction in severity for each of the various types of crashes and the effectiveness in reducing crashes. These values are then used with the approximate annual number of crashes for PAT to produce total savings for each type of incident. Therefore, the crash savings to society—because we have used a social cost of crashes—are almost $1.5 million. PAT has approximately 900 buses. If each bus was fitted with a SCWS, and the system had a life of 3 years, the system would have to cost under $4,350 to break even. We believe that it is feasible to develop, manufacture, and install a system for under this amount. Therefore, with relatively modest reductions in incidents, significant benefits can be gained.

### Table 1 Hypothesized Savings by Incident Type for Side Collision Warning Systems

<table>
<thead>
<tr>
<th>Type of Incident</th>
<th>Savings ($)</th>
<th>Effectiveness Annual</th>
<th>Total Savings</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Prevented</td>
<td>Reduced</td>
<td>Number of Crashes</td>
</tr>
<tr>
<td></td>
<td>Prevented</td>
<td>Reduced</td>
<td></td>
</tr>
<tr>
<td>Pedestrian</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fatality</td>
<td>880,000</td>
<td>850,500</td>
<td>0.1</td>
</tr>
<tr>
<td>Injury</td>
<td>29,500</td>
<td>23,000</td>
<td>0.1</td>
</tr>
<tr>
<td>Vehicle collision</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fatality</td>
<td>880,000</td>
<td>850,500</td>
<td>0.1</td>
</tr>
<tr>
<td>Injury</td>
<td>29,500</td>
<td>14,750</td>
<td>0.1</td>
</tr>
<tr>
<td>PDO</td>
<td>650</td>
<td>3250</td>
<td>0.1</td>
</tr>
<tr>
<td>Fixed object</td>
<td>650</td>
<td>3250</td>
<td>0.1</td>
</tr>
</tbody>
</table>
| Total            | 650        | 3250                  | 0.1           | 0.2        | $19,644

| Total            | $1,465,022 |

Automated Pickup of Solid Waste

The potential benefits of automation of solid waste collection are documented in vendor literature (7). Different stakeholder groups can gain different benefits as indicated below.

Service Provider

Both public and private sector service providers achieve higher productivity rates, which means reduced labor costs and fewer trucks. There are also fewer and reduced value of workers com-
pensation claims, fewer driver and operator injuries, and fewer incidents involving property damage to vehicles and other property. Drivers also report reduced driver stress, and providers note increased driver pride in their equipment. Automated pick-up is neither age nor sex discriminatory.

Customer

For the customer, automation means consolidation of refuse containers and makes it easier for the resident to transport solid waste to curbs. It is also easier for residents to dispose of bulky—but not oversized—items and greatly improves the aesthetics. Automation eliminates animal intrusion and allows the introduction of variable rate structure based on volume, which reduces collection costs for the majority of households.

Society

For society as whole, fewer trucks mean reduced emissions and energy use, and faster pickup means less congestion other vehicles.

Much of the analysis of automated solid waste pickup has focused on productivity improvements due to fewer trucks being able to service more households. For example, Heil (an equipment manufacturer) provides some sample calculations and demonstrates. According to Heil, for a community with 10,700 households, the cost for using a rear loader is $4.66 per home per month, and the cost of using an automated system is only $3.11 per home per month (7). These computations of savings due to productivity improvements are highly dependent on several assumptions, particularly the productivity rates. However, under most reasonable assumptions, it can be shown that the investment in automated equipment can be justified on the basis of productivity improvements alone. This result means that the benefits exceed the costs for the stakeholder group—the service providers that bear the cost of automation. Therefore, benefits that are more difficult to quantify—such as health and safety benefits—will only make automation more desirable. However, it is worth reviewing some of these benefits.

- Studies of the impacts of the introduction of wheeled carts in South Australia in the period 1983/84 to 1987/88 indicated significant reductions in injuries as laborers were not handling the solid waste or lifting the bins (15). Reductions between 36% and 75% are cited. More specifically, the number of workers compensation claims, the number of days lost, and the average cost of each claim were reduced by 32%, 25%, and 48% respectively.
- A more rigorous study of work injuries was conducted using data from California. A statistical analysis of injuries among 176 refuse workers over a six-year period during which automated and semi-automated collection was phased in showed a significant reduction in the hazard rate when the process is mechanized (16).
- Other reports provide anecdotal evidence of the current practice and changes. For example, experiences in Europe, California, Illinois, and Milwaukee are reported where programs are focused on workers safety and relied on some level of automation to improve efficiency (17).
- Reduced emissions and energy use are important quantifiable ben-

FUTURE OPPORTUNITIES

There are significant opportunities for the use of existing technology to become more widespread, and to implement new technologies that are currently available or under development. Some of these technologies appear to offer significant economic benefits. In the solid waste industry, new and emerging technologies include tagged bins, automated weighing of bins and billing, side and rear-collision warning systems, adaptive cruise control, truck weigh control, automatic call to sweep vehicle, clearance checking, and identification and removal of illegal and hazardous substances (18). Similarly, in the transit industry future improvements include side and rear collision warning systems, adaptive cruise control, passenger information systems, and precision docking.

REFERENCES

6. Strangier, M. Godzila, Litter Pig, and Trash Hog just part of history of waste industry, new and emerging technologies include tagged bins, automatic weighing of bins and billing, side and rear-collision warning systems, adaptive cruise control, truck weigh control, automatic call to sweep vehicle, clearance checking, and identification and removal of illegal and hazardous substances. Similarly, in the transit industry future improvements include side and rear collision warning systems, adaptive cruise control, passenger information systems, and precision docking.

ACKNOWLEDGMENTS

Partial support for this work was provided by the Federal Transit Administration, Pennsylvania Department of Transportation, the Port Authority of Pittsburgh, and the Environmental Industries Association Research and Education Foundation. The inputs and ideas of Chuck Thorpe, Christoph Mertz, and David Duggins of the Robotics Institute at Carnegie Mellon have been particularly helpful. Christoph also assisted with the data analysis.


ITS Institutional Issues: A Maintenance/Operations Perspective

LELAND SMITHSON, HAU TO, AND CLARE E. BLAND

As advances in technology find their way into the maintenance activities of departments of transportation, traditional methods are being challenged with innovation. The struggle results in finding a balance between applying past methods that have shown results and using new advanced technology to optimize resources. The issues that are prevalent implementing new and innovative technology in maintenance operations are very similar to deploying intelligent transportation systems (ITS). This paper details these challenges and presents resolutions for addressing these concerns. The paper is based on the findings of a project sponsored by the multinational Aurora consortium and exchange of information among members. One significant issue is that technology-based methods of resolving operations and maintenance problems may meet resistance from staff familiar with conventional, proven methods of performing their duties. Another issue is that deployment of new technology involves considerable investment in training to ensure proper results. The inability to effectively use a system as a result of lack of training creates frustration and loss of faith in the system. Furthermore, the cost of equipment is another issue. With funding for projects already spread thinly across various interests, the competition for funds remains fierce. Public agencies are seeking partnerships with the private sector to share costs for implementing and operating traditionally “public” responsibilities. The Aurora-sponsored project found that, particularly with Road Weather Information Systems (RWIS), the proprietary nature of new technologies tends to hold public agencies to using equipment from a single vendor. This is a concern because, ultimately, the benefits of deploying RWIS are most visible when the technology is implemented as a statewide network. The same will be true with ITS deployments. This issue can be resolved with standards and protocols, but they are slow in emerging. Although there are many challenges, recognizing their existence is the first step in resolving these issues.

INTRODUCTION

The Aurora consortium is leading the charge to maintain a joint program for cooperative research, evaluation, and deployment of advanced technologies for detailed road monitoring and forecasting for improved surface transportation with emphasis on efficient highway maintenance and effective real-time information outreach to travelers. The Consortium provides a forum of exchange for Road Weather Information Systems (RWIS), operations, and maintenance practices among transportation agencies within the United States and worldwide at their quarterly meetings.

By bringing together the leaders in snow and ice control movement, many lessons can be learned from their experiences in implementing RWIS. As a result of expressed frustration in dealing with institutional issues, a project was suggested (funded by Aurora) to research, compile, and share the concerns related to deploying RWIS.

The increased interest in RWIS is a result of efforts to optimize resources. Like many intelligent transportation initiatives, maintenance engineers have sought new technologies for years to further enhance present snow and ice control techniques. While the positive effects of road weather information systems have allowed maintenance personnel to better predict weather events and then to manage these activities, fundamental issues seem to arise with deploying these technologies. In regards to ITS, similar parallels can be drawn. This paper will discuss some of the issues determined from a study funded by the Aurora consortium and show the parallels to implementing Intelligent Transportation Systems (ITS).

TECHNOLOGY AS IT APPLIES TO MAINTENANCE ACTIVITIES

Traditional means of snow and ice control consist of reactive approaches to maintenance activities. When a storm occurs, oftentimes maintenance crews wait for accumulation before plowing and applying salt and sand. With over $2 billion spent on snow and ice control each year within the United States, any reduction or savings from practices that optimize resources would mean alleviating a burden on maintenance budgets. As such, proactive approaches such as anticing and RWIS practices are becoming increasingly more popular forcing maintenance practices to be reexamined. These methods allow maintenance personnel to anticipate storms, preapply chemicals to prevent adhesion of precipitation to road surfaces, and then to better schedule staff to handle the effects of the storms, resulting in better optimization of resources.

The deployment of new technologies for use within the surface transportation community raises many issues that need to be addressed. A study conducted by the Aurora consortium reviewed previous research into RWIS issues. The types of issues that were identified could be categorized into funding, staffing, partnership, and standards issues. The primary sources of archived institutional issues were derived from a set of four documents. They included:
1. Road and Weather Information Systems (RWIS) Feasibility Workshop Summary of Findings (1);
2. Proceedings of the FHWA Surface Transportation Weather Information Workshop (2);
3. Road Weather Information Systems Volume 1: Research Report 1993 (3); and
4. Missouri Weather Collection and Dissemination Study (4).

In the case of the first two publications, the information concerning institutional issues was identified from comments given by the workshop participants. The information on institutional barriers obtained from the Road Weather Information Systems Research Report were the results of actual interviews conducted with snow and ice personnel and from a review of field tests conducted during the winter of 1990. The interviews encompassed personnel from every level of the state agency (3).

In addition to the literature review, Aurora members were interviewed to further support the findings. The results of the literature search and interviews can be found in the report, Review of the Institutional Issues Relating to Road Weather Information Systems (5).

Funding

From the documented research, the issues associated with funding included finding sources and competition for funding (1,2,4). Obtaining funding is the initial step in developing an RWIS program. Although the initial backing for purchase of equipment was important, funding for maintenance and operation of the technology in addition to funding for training of personnel were other considerations. As systems are implemented, funding for upgrading equipment to prevent the technology from becoming obsolete was more apparent. These were challenges for maintenance departments as they competed for funds against such contenders as traditional road repair and rehabilitation projects.

Traditional projects appeared to have more precedence since they provide the most clear-cut results from the road-using citizen perspective. However, obtaining funding may be easier if the benefits of RWIS were shown to be worth the investment. For example, numerous RWIS programs exist across the United States from which to derive proven success stories. These may be used along with published benefit/cost studies to educate higher level positions within transportation organizations about the effectiveness of RWIS. By leveraging the lessons learned and experiences of those with existing systems, the implementation of RWIS may be more cost-effective.

The strong correlation between RWIS and ITS is the use of technology to resolve traditional transportation related concerns. As a result, funding for implementation of ITS projects faces the same funding issues of determining initial funding sources and competing for funding with a multitude of other traditional road preservation and operational projects. Ultimately, whether RWIS or ITS related, the documented research noted that implementation benefits must be quantifiable whenever possible and well-documented in order to justify funding (1,2,4).

Staffing

Once the initial hurdle of acquiring funds has been overcome and RWIS has been successfully installed, the next issue faced was staff acceptance. Successful implementation involved the willingness of maintenance personnel to use the new system and the practices involved. However, the negative mindset—including feelings of cynicism, lack of knowledge of what RWIS can accomplish for agencies, and lack of appreciation for the benefits of RWIS—of staff using RWIS was identified as a difficulty faced by agencies implementing RWIS. The mindset issue was further nourished by the perceived ineffective dissemination of weather information and the lack of understanding of who the customer was and what it was they needed. There appeared to be a distinct need for personnel to overcome technical change and their attitudes towards RWIS and technology (1, 2).

As an example, RWIS use is successful within the Swedish National Road Administration where the system has been in place for many years. However, the system has not always been popular with their staff. When the program was first initiated, their staff saw “an enemy in the typewriter.” There were concerns with trying to use new techniques to interpret and read the data (5). Previous methods for determining winter maintenance included applying chemicals whenever staff felt it was necessary. Personal judgment was preferred over relying on data from sensor systems (5). How could this mindset change to the point where RWIS has become institutionalized to the extent that maintenance personnel are now fully at ease with these technologies? The answer has been considerable investment into initial and continual training.

Sufficient training was an area that seemed to be lacking for the Aurora agencies surveyed within the United States. Many personnel were opting to use “tried and true” methods over RWIS practices. This may have been attributed to the inability to effectively use RWIS resulting in frustration and a lack of trust in the system. Again, looking to the successful program in Sweden, less emphasis on education may result in RWIS being perceived as “another extremely expensive thermometer” rather than a tool that may aid in better decision making (5).

Staff acceptance of new technology could also be quite common in ITS implementation. As documented from the RWIS institutional issues, overcoming the reluctance to accept new technical systems by personnel was a huge hurdle (3). As more investments in advanced methods are implemented, bridging the gap between traditional methods and using ITS and RWIS technologies will become easier as staff become more familiar with and can witness benefits firsthand.

Partnerships and System Ownership

Investment in new technology can be expensive and result in less than ideal expectations. Partnerships were an alternative to bearing the burden of the full cost of a program. From the documented research, partnership issues overlapped other issues such as funding and ownership (1). It was noted that there were concerns over public agencies’ commitments for long-term financial obligations within partnerships. While the public sector was concerned over private sector monopolization of data, the private sector had concerns over giving the information away for free (1,2). Other barriers of concern involved the private sector profit-driven market such as issues over maintaining competition and lack of public interest as a primary concern for private vendors.

Although there was strong interest from the public sector with pursuing private sector partners, there have not been many experiences from which to learn. Public/private partnerships to implement RWIS have been attempted only in Minnesota within the United States and have proved unsuccessful. The lack of success was due to concerns over liability issues and assumption of risk (5). Currently, most RWIS programs exist on a contractor-based premise. In Virginia, for example, RWIS was maintained and operated under a con-
tract with the vendor. The Virginia Department of Transportation (VDOT) owned both the systems and the resulting data. The vendor owned the proprietary program that collects and processes data at the remote processing units and transmits these to the central processing units. Interestingly, maintenance of 150 miles of interstate was privatized within Virginia, and the organization maintaining this roadway had requested RWIS information. VDOT allowed maintenance contractors access to their station data for free, however, these contractors paid a fee to the RWIS vendor for additional access privileges (5).

ITS implementation faces similar partnership issues as documented for RWIS. The difficulties of partnering stem from conflicting bottom-line objectives such as the providing a public service versus the private profit margin. Beyond the contractor agreements, successful partnership models are few and unclear.

Standards

RWIS was most useful with widespread coverage such as when it is deployed as a statewide network. The most pressing issues documented concerned compatibility of systems from different vendors (1). The problem for most agencies when a network was deployed was that they were held to using equipment from the same vendor. While the RWIS market is currently growing and allowing more options in terms of vendors, this does not resolve the issue of compatibility among systems. Compatible communication infrastructures as well as fully compatible hardware and sensors were essential to successfully sharing information between various jurisdictions and creating a useable statewide network of technology. Products developed by different vendors need to communicate with one another, and the only way this may be achieved is through the standardization of protocols. The proprietary development of products was frustrating to public agencies where purchasing policies encourage tendering of generic products and discourage the purchase of single source products. The Ontario Ministry of Transport encouraged suppliers to develop a compatible product to technologies currently in use by approaching RWIS suppliers and expressing their concern with incompatibility of communications systems (5).

Standards and compatibility issues are also consistent with ITS. For example, traffic operations centers collect various traffic data from field systems. Similar to a network of RWIS stations, the ability for different components to exchange and accept information is critical, but is sometimes difficult due to the incompatibility of systems. While standards (such as center-to-center or roadway-to-center communications) are being developed for RWIS and ITS, they are still in the developmental stages. Despite their eventual emergence, the issue of vendors not being held to incorporating standards into their designs remains significant. Nevertheless, standards provide a common framework that will allow for compatibility in the future.

HOW DO DEPLOYMENT OF OPERATIONS AND MAINTENANCE TECHNOLOGIES RELATE TO ITS?

The term ITS often conjures up images of technologies that only assist traffic and transportation engineers in resolving congestion or manage traffic flow. However, there are many facets to ITS among those being the use of technologies for maintenance activities. As already discussed, a host of issues occur with the deployment of “new” technologies that, although documented for RWIS, may be applicable to ITS. While the objectives of traffic/transportation and maintenance engineers may be different, they both have the same underlying goal, making the road a safe environment for drivers. Maintenance personnel may focus on obtaining the best “driveable” surface and providing valuable pavement condition information to users, while traffic and transportation engineers work toward removing incidents from the roadway that may attribute to secondary accidents and better informing travelers of hazards, for example. Whether they are for traffic, transportation, or maintenance engineers, technologies are being sought to assist in better performing their jobs.

CONCLUSIONS

Generally speaking, integration of new technology involves allocation of funds that may be difficult to procure. In addition to initial costs, funding for yearly operations and maintenance in addition to future upgrades also require consideration. Innovative financing methods as well as partnerships with the private sector are attractive options for funding new technology. However, there are currently few examples of successful public/private partnerships. As more partnerships are forged, more can be learned from what creates or prevents successful partnerships.

Transition to new technology requires personnel to adapt to a changing environment in addition to learning new skills. Educating users is an important step in successful applications of technologies, with emphasis on continued training. Although the technical feasibility of equipment is a concern, for technologies such as RWIS and other proven ITS technologies, the issue of standards and protocols is more apparent. Agencies want technologies that are compatible (that can communicate with each other), easily adaptable and upgradable as improved systems continually emerge.

Institutional issues remain some of the biggest concerns to tackle as the internal environment of each agency is unique. Nevertheless, being aware of the types of issues that may arise when deploying a technology-based (maintenance or ITS) project and learning from the lessons of previous endeavors will provide agencies with the insight to face these challenges head on.

REFERENCES

Hours of Service Preferences: A Case Study of a Midwest Carrier’s Drivers

FENG YONG HUANG AND CLYDE KENNETH WALTER

The length of the working day had been declining for over a century and was a topic of concern when early motor carrier laws were written. Unchanged since the late 1930s, hours of service regulations continue to be debated for possible revision. This report is based on a survey of truck drivers that measured their hours of service preferences, experiences, and backgrounds. Their responses suggest that experienced drivers are more likely to prefer the current regulatory situation, while newer drivers favored changed working hour limits. Discussion includes historical background as well as implications for policy makers, managers, and future researchers.

INTRODUCTION AND BACKGROUND

Hours of service (HOS) regulations for truck drivers are over 60 years old but are facing revisions pending further deliberation in Congress. Major arguments for changing the rules are based on the effects of driver fatigue and working conditions. Fatigue has obvious safety connotations but can detract from overall working conditions and job satisfaction (1), factors that contribute to the high turnover rates of drivers. This paper presents the results of a survey of the drivers for a concerned Midwest carrier. Their indicated preferences and problems with HOS regulations, while not necessarily representative of all truckers, can add insight to the continuing debate for future legislative and management changes.

Work Hours: Pressures to Reduce

The eight-hour workday has precedents well before the twentieth century, with examples of English coal miners and ploughmen working shorter shifts in the 1700s (2). A 1918 report noted a shift in the U.S. working day from 10 to 8 hours in slaughtering and meatpacking, machine trades, garment production, shipyards, coal mining, and railroads. Reasons centered on the health hazards of fatigue, the “general loss of moral restraint and increase of intemperance,” and a lack of leisure time and energy (3). During the 1930s, trainmen argued for a 6-hour workday, citing the current high unemployment and the ability to realize “a more cultural existence for workers”(4).

Trucking Hours of Service

In 1933, the average trucker earned $24 per week, but the hours ranged from 50 to 99. According to a 1934 trucking industry code of fair competition, drivers could work up to 108 hours in any two-week period, with overtime pay at one and one-third their normal salary for over 48 hours a week. Federal legislation of trucking came in 1935, when Congress passed the Motor Carrier Act, giving the Interstate Commerce Commission (ICC) the power to regulate the industry (5). A National Safety Council survey showed fatigue as exceeding all other causes of accidents and more likely to occur to truck drivers than to other drivers. It noted that most of the well-run truck fleets had already adopted safety measures voluntarily, so the effects of legislation would “reduce competition from and chance of collision with trucks whose drivers are working dangerously long hours” (6).

In an early confrontation, the Teamsters Union wanted an 8-hour workday, but lost to an American Trucking Associations proposal for 60-hour weeks and the use of sleeper cabs. The regulation, effective December 1, 1938, allowed interstate truckers to work 10 hours per day before a mandatory eight-hour rest. A year later, the ICC allowed an added 5 hours for loading or unloading (7). Other provisions were a limit of 70 hours of driving in 8 days and a requirement for a daily logbook. The regulations became Title 49, Code of Federal Regulations, part 395 (as amended March 1, 1939) and applies to interstate truck drivers engaged in for-hire service (8). Two exceptions—emergencies and adverse driving conditions—allow up to two additional hours to complete the run or to reach a safe place (9).

HOS continues to receive attention, as motor carriers have experienced a shortage of drivers, leading to high turnover rates and the attendant costs of hiring and training new drivers. Drivers cite compensation, loneliness, unpredictable work schedules, poor working conditions, and lack of advancement opportunities as reasons for leaving (10). A survey of 67 truckload carrier executives cited “outdated hours-of-service rules” as one of the biggest safety issues (11).

MOTOR CARRIER CASE STUDY

Data Collection

A participating Midwest truckload carrier agreed to distribute and collect a survey of its 100 drivers about their preferences for proposed alternatives for HOS regulations. The questionnaire was a single sheet, folded once to form a four-page booklet. The title page identified the survey and the university affiliation of the researchers.
It asked for three minutes of time and return of the completed survey to a clearly labeled box. Most questions printed inside were structured for easy response, either checking a multiple-choice list or entering a number. Fifty-one surveys were returned in at least partially complete form.

**Driver Characteristics**

Drivers indicated their age category (20-29, 30-39, etc. and 60 and over) and filled in their years of experience. The median age group was 40 to 49, with 22 drivers under 40 and 26 who were 40 and above, providing two groupings for comparisons. Experience varied from 2 to 50 years, with a median of 10 and a mean of 14.5.

The hypothesis that the mean differences of the experiences of the younger vs. older groups were equal was tested, using the t-test, and rejected (t = 5.65; d.f. = 48; level of significance = .005). Younger drivers averaged 6.6 years of experience, compared with 20.6 years for those 40 and over. While expected, this difference also validates the data, i.e., responses appeared to be sincere. The standard deviation of years of experience—4.0 for the younger drivers and 12.3 for the older—shows that the occupation of driving was entered at various stages of life for the older drivers, possibly after earlier agricultural or manufacturing employment.

The results confirmed that truck driving is a high-turnover occupation. The median number of employer changes was 3; the mean was 3.6; and the range was zero to 15 times. The prime reason was income, cited by 72% of the respondents, which supports Schultz’s statement that compensation is the reason drivers leave their jobs (12). Other reasons for switching employers were schedule (36%) and equipment (34%).

**Preferred Hours of Service Alternatives**

Five HOS alternatives were provided, taking their form from limits imposed in other countries or those being proposed for the U.S. Two types of changes were included: one places drivers on a 24-hour clock (as opposed to the current 10 hours driving plus 8 hours rest cycle), and the other increases the minimum rest period by 50% (13). The expanded rest period is consistent with the National Transportation Safety Board recommendation “to enable drivers to obtain at least 8 continuous hours of sleep” after driving. The Board concluded that drivers involved in fatigue-related accidents averaged 2.5 hours sleep less than drivers in non-fatigue-related accidents (14), because the current 8-hour minimum off-duty time does not allow for other personal needs.

**10 Hours Driving and 10 to 12 Hours Off-Duty**

The first selection was the current daily driving restriction but with the additional off-duty time, as required in the above proposal for long-haul drivers. As Figure 1 shows, this proposal was picked by only 8% of the drivers.

**FIGURE 1 Preferences for changed hours of service regulations**

**12 Hours On-Duty and 12 Hours Off-Duty**

The second alternative goes a step beyond simply increasing the rest periods and places drivers on a 24-hour work-rest cycle. This 12 on-12 off format was selected by 27% of respondents. It would increase flexibility of scheduling by permitting up to two additional hours driving per shift and addresses the issue of inadequate sleep by including 50% more rest hours than current practices.

**13 Hours Driving per Day**

Canadian HOS rules generally match the U.S. rules except for a higher driving-time limitation of 13 hours per day. This proposal, which would provide some uniformity for North American truckers, received the least support (6%).

**14 Hours Driving per Day**

Perhaps the allure of increased earning power of 14 hours on the road, based on railroad engineer maximums, reduced the potential popularity of the Canadian 13 hours limit; 21% of the sample preferred the higher limit. The combined support for 13 and 14-hours driving per day matched the preference for the 12 and 12 plan. Any of these three alternatives would place drivers on the 24-hour work-rest cycle more typical in the work place than the 18-hour minimum cycle currently provided by the trucking HOS laws.

**No Change from Current Regulations**

The top-ranked alternative, from 38% of the drivers responding, was keeping the current restrictions. Among the several messages in these responses may be a preference for the familiar—the rules the drivers observe daily—compared to a change, even if the change is simple and provides flexibility plus safety. Second, among the four alternatives for change, the 12 and 12 had the most support. This proposal would place drivers on a 24-hour cycle, allowing drivers a more normal living schedule. The 14 hours of driving limit also had
some support, presumably because it would allow drivers the opportunity to increase their income through longer hours than currently.

**Problems Caused by Hours of Service Laws**

A clear majority of drivers (58%) occasionally have problems they attribute to HOS laws, as seen in Figure 2. The drivers who said they frequently had problems were just about balanced out by those who never have problems, 20% and 22% respectively. Tight schedules and bad weather were the main problems, with 58% of drivers including them in responses. Scheduling becomes a problem if carriers use the legal maximums as the routine hours expected, with little or no room for delays. Weather problems would lower average speeds and require longer time for completing tightly scheduled trips. Income problems, with 56% of respondents indicating, could be attributed to any restriction on hours driven that may limit an employee’s income. The fourth-ranked problem (51%) was traffic, which, like scheduling and weather, is a cause for delay and lower average speeds. Equipment and “other” each received 21%. Other problems included over-sleeping and lack of regular sleep, waiting for loads and other adverse customer influence, layovers, and inconsistent enforcement of states’ speed laws.

**Comparisons Between Groups**

**Change vs. Current Regulations**

Since the greatest portion of drivers preferred the current regulations to the changes listed, the responses were first grouped into change, including all responses to the four proposed alternatives, vs. current preferences. These two groups were compared with the remaining variables of problem frequency, types of problems, employer changes, reasons for change, experience (0-10 and over 10 yr.), and age (20-39 and 40-up). Table 1 lists the results of the chi-square calculations, showing that two null hypotheses would be rejected at the .05 level of significance:

**TABLE 1 Change vs. No Change Comparisons (for Grouped Data)**

<table>
<thead>
<tr>
<th>Variable Cross-tabulated</th>
<th>d.f.</th>
<th>Chi-sq.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency of problems (never, occasionally, frequently)</td>
<td>2</td>
<td>6.06**</td>
</tr>
<tr>
<td>Problems caused</td>
<td>5</td>
<td>1.08</td>
</tr>
<tr>
<td>Employer changes (0-3 vs. 4-up)</td>
<td>1</td>
<td>0.39</td>
</tr>
<tr>
<td>Reasons for change</td>
<td>3</td>
<td>1.69</td>
</tr>
<tr>
<td>Experience (0-10 vs. over 10 yr.)</td>
<td>1</td>
<td>4.62**</td>
</tr>
<tr>
<td>Age (20-39 vs. 40-up)</td>
<td>1</td>
<td>0.82</td>
</tr>
</tbody>
</table>

Differences significant at:

*** .01 level  
**  .05 level  
*  .1 level

Drivers’ preference for changing vs. retaining current regulations do not vary by:

1) the frequency of hours of service problems they have experienced; and

2) the number of years of driving experience.

The second null hypothesis, with driving experience expressed as a continuous variable, was also rejected (at the .05 level) using a t-test.

Figure 3 shows that 67% of those drivers favoring changed regulations have occasionally experienced HOS problems, compared with 39% of those who would retain the current law. Drivers reporting frequent problems were evenly split (23 and 22% for change vs. current, respectively). Only 10% of the drivers favoring changed laws had never had problems with them, while 39% of those in favor of the status quo had not encounter hours-related problems. In other words, drivers who had more extensive backgrounds relative to HOS were more strongly supporting the concept of changed regulations than were drivers for whom the current laws had not been restrictive.
should not be surprising since the experienced drivers must have found their working conditions, including the legal environment, generally acceptable or else they would not have amassed their years of experience. Less experienced drivers would be expected to be more open to considering alternatives that might improve conditions in their more recently chosen occupations.

Any mandated changes in the current regulations would need to be clearly explained to those who make their living through truck driving. Rationale may be found in the studies of fatigue, which show the necessity for reasonable uninterrupted sleep, and through comparisons with work hours, including overtime policies, in other industries. Conversely, legislators may consider two-tier regulations, with older limits being “grandfathered in” for current drivers who want to retain them (possibly subject to demonstrated safety records), while more restrictive limits apply to newly hired drivers. Lawmakers may also consider overtime provisions, which would provide income enhancements for drivers and also encourage carriers to schedule drivers more in line with their other employees.

Management Implications

Policymakers and carrier managers alike need to balance their concerns with safety and the issues that concern the drivers—those individuals most directly affected by HOS regulations. Rodriguez and Griffin stated that drivers need to be treated “as an equal with other employees” (15). One effective step toward meeting that need may be the collection of drivers’ viewpoints about the legal limits and then scheduling their working hours with these in mind. Managers need to be aware of drivers’ concerns about tight schedules, bad weather, and opportunities for income. Even though the drivers surveyed were generally comfortable with the current hours of service regulations, carrier management must remember that the law established merely the maximums. It was not intended to set day-to-day normal working hours for one occupation that are noticeably longer than for others.

Research Implications

This project was a demonstration of information collected at the worker level rather than a survey of managers. While it was a case study of one firm only, it benefitted from a sample size of 50% of the total drivers, largely through the cooperation of that firm. Other researchers, including those providing background for legislators, may benefit from employing similar surveying and data collection techniques. If and when HOS regulations are revised, their acceptance may be improved if they are based to some degree on input from the drivers, as well as from experts on fatigue and safety and carrier managements concerned with driver turnover.

CONCLUSIONS

Policy Implications

Background material showed that working hours are a centuries-old controversy. Legislation has limited working hours where public or worker safety has been deemed in peril. The results of this case study of Midwest drivers showed their first preference was for the regulations as they already know them. This outcome should be significant for policy-influencers because it portends some resistance to changes that may be based on sound safety and social reasons. Experienced drivers were most strong in their preference for not changing the current regulations, and they would be a necessary “core” group for a successful transition to any new limits. Their ability to increase their earnings through their driving performance appears to outweigh the desire for shorter hours found in other industries. Less experienced drivers will be more likely to embrace new rules.

REFERENCES

Pneumatic Capsule Pipeline—Basic Concept, Practical Considerations, and Current Research

HENRY LIU

Pneumatic capsule pipeline (PCP) uses air blown through a pipeline to propel capsules (wheeled vehicles carrying cargoes) through the pipeline. It is a modern and large version of the century-old technology of “tube transport” used rather widely and successfully in the first half of the 20th century in major European and U.S. cities for transporting mail, parcels, telegraphs, documents, cash, and other lightweight materials. Modern PCP systems, such as those used in Japan for transporting limestone to a cement plant, use large wheeled capsules moving heavy cargoes through pipes of 3-ft diameter, approximately. Each capsule can carry almost two tons of cargo. The system is driven by blowers located near the beginning of the pipeline, and it is highly automated (by computers and programmable logic controllers). The system is being used very successfully in Japan, with a high reliability record. Yet, only limited use exists today due to its high unit freight transportation cost in $/ton as compared to that by truck. The unit cost is high due to low system throughput (freight capacity). Major improvement in throughput can be made by replacing the pumping mechanism from blowers (which are used currently), to electromagnetic pumps (for the future systems), and using off-line loading/unloading. Research in such improvements of PCP is currently underway at the Capsule Pipeline Research Center at the University of Missouri-Columbia. Key words: capsule pipeline, PCP, pneumatic capsule pipeline, tube freight, underground freight transport.

INTRODUCTION

Capsule pipeline is the transport of freight by capsules moving through a pipeline propelled by fluid—either gas or liquid. When the fluid involved is liquid (usually water), it is called hydraulic capsule pipeline (HCP), and when the fluid is gas (usually air), it is called pneumatic capsule pipeline (PCP). Both HCP and PCP have distinct characteristics and respective advantages and disadvantages. Therefore, they are best suited for different applications—having their different niches.

Because the fluid used in HCP (water) is much denser than the fluid used in PCP (air), the capsules in HCP can be suspended by the fluid and hence they don’t need wheels to move through the pipe. In contrast, PCP capsules need wheels in order to carry heavy cargo. On the other hand, PCP capsules move much faster in the pipe than HCP capsules—15 to 20 m/s for PCP and only 2 to 3 m/s for HCP. Due to these and other differences, HCP is more suitable for transporting low-cost bulk materials such as coal and other minerals, grain and other agricultural products, and solid wastes. Speed of transportation is not a crucial factor in transporting these commodities. In contrast, PCP is more suitable for transporting higher valued products that must be delivered speedily, such as mail and parcels, fruits and vegetables, and many other commercial products. Details about HCP and PCP and their expected applications can be found in a 1998 publication of the Task Committee on Freight Pipelines, American Society of Civil Engineers (1).

The advantages of using PCP to transport freight are not difficult to see. When PCP is used to transport a significant portion of freight in the future, it will reduce the number of trucks on highways and streets, resulting in reduced traffic jams, accidents, air and noise pollution, and damage to pavement and bridges, that would otherwise be caused by trucks. Consequently, PCP can make a significant contribution to the U.S. highway system in the future.

The purpose of this paper is to discuss the basic concept, practical considerations, and current research on PCP, especially with respect to its potential future use as a means of underground freight transport between major cities.

BASIC CONCEPT

All contemporary PCP systems, such as those used in the former Soviet Union for transporting rock (2) and those used in Japan for transporting limestone and other products (3), use wheeled capsules rolling through a pipeline. For capsule stability inside a circular conduit (pipe), it is not permissible to mount wheels on the bottom of capsules. Instead, two sets of wheels are mounted on the two ends of each capsule as shown in Figure 1. Each wheel set consists of a minimum of five small wheels placed in symmetric position around the pipe interior in contact with the pipe wall. The wheels are connected to the central axis of the capsule and free to rotate around the pipe in a manner similar to gimbals. In so doing, the capsule will always remain in a stable equilibrium position without spilling the cargo. For PCPs that use square or rectangular conduits, as used in Japan for tunnel construction (4), the gimbals-type wheels are no longer needed. Such PCPs use capsules with wheels mounted on capsule bottom, as in the case of ordinary vehicles that run on flat horizontal surface. Most current systems use rubber tires to minimize noise and prevent wear of the pipeline.

The PCP capsules normally have a diameter approximately 85% of the pipe diameter, and a capsule length about five times the capsule diameter. This means the capsule length-to-diameter ratio, called the
“aspect ratio,” is about 5. Both ends of the capsule are mounted with an end disk made of steel having a rubber rim. The end disks minimize air leakage through the capsule, increase capsule drag, and make the capsule to move almost as fast as the air moving in the spacing between capsules. In ordinary commercial PCPs, the end-disk diameter is about 97% of the pipe inner diameter. This creates a drag coefficient in the neighborhood of 1,000, and a capsule velocity within 1 m/s of that of the fluid (air) velocity.

Normally, five or more capsules are linked together to form a capsule train moving through the pipe. Double pipe is used—one for delivering the cargo, and the other for bringing empty capsules back to the intake. The capsule train is first loaded with cargo at the loading station. Then, air is directed behind the capsules to push them through the pipeline. As the capsule train reaches the end of the pipe, the capsule bottom opens up, and the cargo is dropped by gravity. Then the air is directed behind the empty capsules to push them back to the pipe inlet. The entire system is shown in Figure 2.

The current commercial systems of PCP, as used in Japan, have demonstrated their high reliability—over 98% of availability when needed. The system is highly automated, and the operation cost is low. In spite of that, the system has not been widely used even in Japan due to the following reasons.

The current PCP systems have very low linefill rate (3%). This means only 3% of the pipe length is occupied by capsules; 97% is empty. As a result of this low linefill rate, large diameter pipe is required to transport relatively small quantity of cargo—the throughput is low! Note that unit freight transportation cost in $/ton is directly proportional to the total system cost and inversely proportional to the linefill rate or throughput. Therefore, a five-fold increase in linefill from 3% to 15% would cut the unit cost by half even if the total system cost is increased 2.5 times due to increased throughput—more capsules, more handling facility, and higher operation costs.
There are two reasons for the low linefill and low throughput. The first is the use of blowers (fans) which block the passage of the capsules. Capsules must stop before they reach any blower, and then must be routed through a bypass line in order to proceed down the pipeline without going through the blower. This greatly impedes the capsule traffic, and limits the linefill rate. Another reason for the low linefill is in-line loading/unloading. Instead of taking the capsules out of the pipe for loading/unloading, the current system has the capsules loaded and unloaded while they are inside the pipe. Flow is stopped while loading/unloading is taking place. This again greatly impedes the traffic and reduces the linefill rate.

Therefore, in order to improve the current PCP system and increase its linefill, two things must be done. First, a non-intrusive type of pump must be used in lieu of blowers which impede capsule motion. Secondly, loading and unloading of capsules must be done outside the pipeline—i.e., off-line loading/unloading. With such improvements, it is believed that a fivefold increase in the linefill of PCP from the current 3%, to 15%, is possible.

The most promising non-intrusive pump for PCP is electromagnetic capsule pump such as linear induction motor (LIM), the same technology used for advanced roller coaster systems and magnetic propulsion and levitation of high-speed train (5). As capsules of metallic wall enter the LIM, an electromagnetic thrust is generated on the capsules to accelerate and to push the fluid forward. The capsule wall will be made of steel with an outer layer of aluminum. An “eddy current” is induced in the aluminum wall which generates the electromagnetic force (thrust) needed to push the capsules forward. By having the inner diameter of the LIM made slightly smaller than the inner diameter of the pipe, the capsules going through the LIM behave like a piston pump, causing the fluid in the pipe to be pushed forward (6). See Figure 3 for such a pump.

Note that LIMs can be used not only for pumping capsules, but also for capsule injection/ejection, climbing steep slopes, capsule speed control, and capsule breaking. They need not be placed along the entire pipeline. On the contrary, they need only to be placed at strategic locations along a PCP. Furthermore, single-sided LIMs can be used to control the direction of motion of capsules at pipeline branching points (see Figure 4). By turning on the LIM on one side of the branch instead of the other, a lateral electromagnetic force is generated to direct the capsule into the desired branch. This means that the LIM can also be used for system control purpose.

All PCP systems must be automatically controlled. This requires the detection of capsules at many strategic locations along the pipe. Capsules can be detected and identified by placing bar codes on them and using a special bar-code scanner to detect and identify them.

For transporting bulk materials such as minerals or solid wastes, cylindrical capsules in cylindrical pipe are preferred. In contrast, for transporting packaged commodities, capsules of square or rectangular cross sections traveling inside a square or rectangular conduit will be the choice. A 1.5 m wide rectangular section will allow cargo to be transported on pallets, and a 2 m wide rectangular section will allow large crates to be transported. Rectangular conduits can be used for PCP due to the relatively low internal pressure in the system—usually not more than one or two atmospheric pressure. Such conduits can be made of reinforced concrete.

**CURRENT RESEARCH**

Only limited research is currently taking place in PCP to improve the system. Four current or recent research projects are known. One is a project completed recently at the University of Minnesota, sponsored by the Minnesota Department of Transportation (7). This study investigated the possible use of large conduits or tunnels of diameter greater than 3 m placed under a highway. The capsules are self-propelled vehicles carrying their own electric motors. Such a system is more of the nature of underground electric trains rather than PCP.
though many commonalties exist between such a system and PCP. The greatest advantage of this system is it can carry large size cargoes—including standard containers. The drawback of the system is high cost.

A study is taking place currently in Florida for transporting phosphate by using PCP (8). The PCP is to be driven by linear synchronous motors (LSM) instead of linear induction motors (LIM). LSM may have slightly higher efficiency than LIM, but it is difficult to control because the capsule speed and the motor speed must be synchronized. Furthermore, synchronous motors do not self-start. The motor has zero thrust when capsules are in standstill. The rotor (capsules in this case) must be brought to synchronous speed by some other means (a starter) before the LSM can start to work.

A third study is being conducted at Texas Transportation Institute (TTI). In the TEA-21 legislation, TTI was authorized (and funded with $1.125 million) to conduct a feasibility study of using PCP to transport freight in Texas in order to reduce truck traffic (9). The scope of the study does not include research to improve the current system.

Finally, in 1998, the Mid-America Transportation Center, in collaboration with the Sumitomo Metal Industries, Ltd. in Japan, sponsored a PCP research project at the Capsule Pipeline Research Center (CPRC), University of Missouri-Columbia. The project is focused on studying the use of LIM for PCP. The pertinent electromagnetic and fluid mechanics equations have been derived (10, 11). Currently, preparation is underway to test a special LIM for PCP donated by Force Engineering, Inc. in England (see Figure 5). The test data will be used to verify the equations derived so that these equations can be used with confidence for designing future PCP systems powered by LIMs.

Other much needed research before the improved PCP can be developed and used smoothly in commercial applications includes a study to construct PCPs along highways easement, testing the use of modern bar code scanning systems to detect, identify, and control PCPs, designing a new system for efficient off-line loading/unloading of capsules, and investigation of the economics and market of the new PCP system.

CONCLUSION

It can be concluded that the use of PCP for underground freight transportation has many environmental and safety benefits including reduction of traffic jam, accidents, air and noise pollution and damage to roads and bridges—all caused by the reduced use of truck for freight transport resulting from using PCP.

ACKNOWLEDGEMENT

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REFERENCES


Crash Injury Severity of Older Drivers in Iowa

Aemal J. Khattak, Michael D. Pawlovich, and Reginald R. Souleyrette

According to the US National Highway Traffic Safety Administration, 175,000 older individuals were injured in traffic crashes in 1997, representing 14 percent of all traffic fatalities and 13 percent of all vehicle occupant fatalities in that year. Compared with the fatality rate for drivers aged 25-69 years, the rate for drivers 70 years or older is nine times higher. With the aging of the United States population, the safety of older drivers is becoming more crucial every day. Using 1990 through 1997 reported crash data from the State of Iowa, the authors investigated causal factors that may contribute to the severity of injuries inflicted on older drivers (age ≥ 65 years) involved in single vehicle crashes. The authors used the ordered probit modeling technique and investigated factors from vehicle, roadway, and driver characteristics that can potentially contribute to injury severity. Some environmental, temporal, and policy factors were also investigated. The objective was to isolate factors leading to more severe injuries among older drivers so that transportation agencies may focus on those to improve safety for older drivers in particular. Analysis results indicate that advancing driver age and lack of occupant protection inside the vehicle significantly contributed to the severity of older drivers' injuries. Crashes that involved an overturned vehicle or a vehicle striking a fixed object were more injurious to older drivers. Older driver injuries were more severe in farm vehicles as compared to injuries in other types of vehicles. Additionally, injuries to older drivers were more severe when crashes occurred on curves and in rural areas. The authors discuss some of the improvements that may be of interest to transportation agencies desiring to improve the safety of older drivers. Key words: injury severity, traffic safety, older drivers, ordered probit.

LITERATURE REVIEW

The literature reviewed in this study indicates the vulnerability of older drivers involved in traffic crashes. Previous research utilizing Iowa crash data indicates that both age and gender are good predictors of injury severity in multiple-vehicle broadside and angle collisions (6) and head-on collisions (7). Additionally, older drivers experience a higher frequency of crashes per unit distance traveled. The two studies indicate that older drivers are more likely to suffer severe or fatal injuries in a crash, and that changes in driving ability due to reductions in visual capacity, peripheral vision, and cognitive process (e.g., attention and reaction time) are important factors in increased crash involvement. Physiological changes such as bone density loss, age-related diseases, and reduction in resiliency to injury are identified as contributory factors to injury severity of older drivers.

A report by the Iowa Department of Transportation's Safety Management System Task Force on Speed Limits indicates that deterioration of driving skills with advancing age of drivers is an important attribute, and speed differential is a problem for older drivers (5). A Nebraska study that examined issues related to safety of older drivers (8) indicates deficiencies in driving knowledge and deficits in physical, perceptual, and cognitive abilities as issues related to the safety of older drivers. Researchers using data from Hawaii determined that very young and very old drivers are most likely to be at fault in crashes (9). Richardson et al. (10) report that older drivers have higher incidences of involvement in rear-end, sideswipe, and broadside crashes. A study by Kim et al. (11) mentions the possibility that resiliency after injury or other factors affecting injury might affect the odds of older drivers being injured in a crash. Another study examining run-off-road crashes reports that age is not a significant factor in such crashes (12).

INTRODUCTION

According to the National Highway Traffic Safety Administration (NHTSA) of the United States Department of Transportation, 175,000 individuals 70 years of age or older were injured in traffic crashes in 1997. This represents 14 percent of all traffic fatalities, and 13 percent of all vehicle occupant fatalities in 1997 (1). Calculated on the basis of estimated annual travel, the fatality rate of older drivers is the highest among different age-based driver groups and is nine times as high as that of drivers between the ages of 25 and 69 years. Furthermore, according to the Federal Highway Administration (FHWA), the number of older persons who are licensed to drive continues to increase (2). The United States population is undergoing a major demographic transformation that is resulting in a larger proportion of older individuals in the population (3, 4, 5). With the aging population and increase in older licensed driver numbers, the safety of older drivers is becoming more crucial every day.

DATA COMPILATION

A number of factors can potentially affect injury severity in a single vehicle crash. The authors conceptualized those factors under several major categories: policy actions, roadway characteristics, driver attributes, vehicle characteristics, crash characteristics, and environ-
mental and temporal characteristics (see Figure 1). The presence of so many factors under these categories complicates the relationships between these factors and injury severity. To discern meaningful relationships among older driver injury severity and the influencing factors, the authors restricted the available data (1990 through 1997) in several ways. First, the reported crash data were limited to single vehicle crashes involving older drivers (age ≥ 65 years) in which an injury was reported. Injuries on crash record forms in Iowa are rated on the KABCO scale of severity: K (Killed), A (incapacitating), B (evident), C (possible), and PD (property damage only/other). The authors limited the crash data to the categories represented by KABC, i.e., the property damage crashes were taken out. Second, single vehicle crashes involving pedestrians were removed from the data due to the involvement of multiple persons (driver and pedestrian) in the crash and the possibility that the reported injuries may be those of the pedestrian and not of the driver. The resulting file contained 1,984 observations representing single vehicle crashes in Iowa involving injury to older drivers.

The maximum speed limit on some rural expressways in Iowa was raised from 88 km per hour (55 mph) to 105 km per hour (65 mph) in 1996 after the US Congress repealed the National Maximum Speed Limits in November 1995. Single vehicle crashes involving older drivers that occurred before and after 1996 on those particular routes were identified to study the impact of increased speed limit on injury severity of older drivers.

Figure 2 presents older driver gender and age distributions across the four levels of injury severity under investigation. Overall, 590 C-type, 948 B-type, 361 A-type, and 85 K-type, single vehicle crashes involving older drivers were reported during the study period. Except in the C-type category, male drivers were over-involved in all injury severity levels. The numbers of male and female drivers reporting C-type injuries were equal in the data file. Many more male and female drivers in the 65 to 75 years of age category reported injuries (of all levels of severity) when compared to drivers in the 76 to 85 years category. Similarly, many more male and female drivers in the 76 to 85 years category reported injuries (of all levels of severity) when compared to drivers in the 85+ years category.

Figure 3 presents the distribution of different types of vehicles and the damage sustained by the vehicle during the reported crash.

Passenger cars were involved in most of the crashes followed by pickup trucks. These two vehicle categories reported the heaviest damage to vehicles as well.

Figure 4 presents the distribution of crashes across type of roadway and as a function of type of environment (rural and urban). Most...
crashes involving older drivers occurred on county roads and state/US routes. Most crashes occurred in the rural environment, which is not surprising given the rural setting of Iowa.

MODELING INJURY SEVERITY

The dependent variable in this study is the severity of injury sustained by older drivers involved in single vehicle crashes. The crash severity scale is ordinal (K-type representing most severe and C-type representing least severe injuries). The appropriate models to use for severity scale is ordinal (K-type representing most severe and C-type representing least severe injuries). The appropriate models to use for severity scale is ordinal (K-type representing most severe and C-type representing least severe injuries). The appropriate models to use for severity scale is ordinal (K-type representing most severe and C-type representing least severe injuries). The appropriate models to use for severity scale is ordinal (K-type representing most severe and C-type representing least severe injuries).

According to Greene (16), the ordered probit model has the following form:

\[ y^* = \beta' x + \varepsilon \]

where:
- \( y^* \) is the dependent variable (injury severity in this case),
- \( \beta' \) is the vector of estimated parameters,
- \( x \) are the explanatory variables, and
- \( \varepsilon \) is the normally distributed error term.

The parameter estimates (\( \beta' \)) represent the effect of explanatory variables on the underlying injury scale. Based upon this specification, the probability of the dependent variable falling in any ordered category is:

\[
\Pr(y=n) = \Phi(\mu_n - \beta'x) - \Phi(\mu_{n-1} - \beta'x)
\]

where \( n = 1, 2, 3, 4 \).

Computation of marginal effects is meaningful for the ordered probit model because the estimated parameter coefficients do not represent the magnitudes of variables \( x \) on the intermediate categories of the dependent variable. A measure of the model goodness of fit (\( \rho^2 \)) can be calculated as:

\[
\rho^2 = 1 - \left[ \frac{\ln L_0}{\ln L_\text{b}} \right]
\]

where \( \ln L_0 \) is the log likelihood at convergence and \( \ln L_\text{b} \) is the restricted log likelihood. The \( \rho^2 \) measure is bound by zero and one. Values of \( \rho^2 \) closer to one indicate better fit of the model.

MODELING RESULTS

The authors estimated an ordered probit model with injury severity of older drivers as the dependent variable. Independent variables from all the categories of factors that may affect injury severity were tried in the model (see Figure 1). Table 1 presents the modeling results. The rho-squared (\( \rho^2 \)) term for the ordered probit model, a goodness-of-fit measure, indicates a reasonable fit between the model and the data. A positive estimated coefficient in the model implies increased severity with increase in the value of the explanatory variable. Independent variables from each category that were tried in the model specification as well as the ones that the model indicated as significantly contributing to injury severity are discussed below. The marginal values provide the impacts that a unit change in the individual independent variables have on different levels of injury severity when all other variables are held at their means.

The model indicates a positive estimated coefficient for driver age, which is statistically significant at the 95 percent confidence level (a \( z \)-statistic of 1.96 or higher indicates significance at the 95 percent confidence level). This indicates that advancing age increases the propensity of more severe injury, as expected by the authors. The positive sign of the estimated coefficient for gender (coded as male = 1, female = 0) in the model indicates that male older drivers experience more severe injuries. The absence of occupant restraint systems inside was investigated by including an indicator variable in the model specification. As expected, the model indicated strong statistical evidence that older drivers in vehicles without any restraint systems incurred more severe injuries. Although not statistically significant, further probing indicated that older drivers under the influence
of alcohol tend to experience more severe injuries as compared to older drivers not under the influence.

Several variables from the roadway category that may contribute to the severity of injuries to older drivers in single vehicle crashes were examined. The model indicated that crashes occurring at curves in level terrain were more injurious compared to crashes occurring at other locations. The roadway network in Iowa consists mostly of straight highway sections. It is possible that older drivers may not fully compensate in terms of vehicle speed for curves on highways resulting in more severe injuries at curves on level terrain when compared to other locations. Speed limit was included in the model specification to evaluate its effect on the injury severity of older drivers. As expected, the estimated coefficient for speed limit is positive. However, the variable is not statistically significant. Differences in severity of injury across highway sections and intersections were explored (by using an indicator variable), but the model did not indicate the presence of any statistical evidence. The indicator variable was then excluded from the model specification to improve model fit.

The type of vehicle may also considerably influence severity of driver injury. Several types of vehicles (passenger car, pickup truck, etc.) were tried in the model specification. Interestingly, farm vehicle crashes resulted in significantly higher injuries compared to other types of vehicles. This finding is important because only 17 percent of farm vehicles in the data set had incurred heavy damage, yet the injuries sustained in farm vehicles were more severe. Farm vehicle traffic in Iowa significantly increases during the spring and fall, and although usually slow moving, they are unique and significantly different in design than regular vehicular traffic on roadways. Several types of crashes were explored to study their effects on severity of injuries to older drivers. As expected, the model indicated that overturned vehicles resulted in more severe injuries to older drivers compared to other types of crashes. Also, crashes in which vehicles struck a fixed object resulted in more severe injuries. However,

<table>
<thead>
<tr>
<th>Category</th>
<th>Independent Variable</th>
<th>Estimated Coefficient</th>
<th>z-statistic</th>
<th>Marginal Effects</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
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<td>C-type</td>
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<tr>
<td>Driver characteristics</td>
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<tr>
<td>Age (in years)</td>
<td>0.011</td>
<td>2.763</td>
<td>-0.0038</td>
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<tr>
<td>Gender (male = 1, female = 0)</td>
<td>0.220</td>
<td>4.240</td>
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<td>0.0106</td>
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<td>Non-use of seat belt (not used = 1, used = 0)</td>
<td>0.497</td>
<td>8.062</td>
<td>-0.1691</td>
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<tr>
<td>Drunk (under influence =1, otherwise = 0)</td>
<td>0.133</td>
<td>1.034</td>
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<tr>
<td>Roadway characteristics</td>
<td></td>
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<tr>
<td>Speed limit (in km per hour)</td>
<td>0.002</td>
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<td>Curves in level terrain (curve in level terrain =1, otherwise = 0)</td>
<td>0.218</td>
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<td>Farm vehicle (farm vehicle = 1, otherwise = 0)</td>
<td>0.989</td>
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<tr>
<td>Crash Characteristics</td>
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<td>Vehicle overturned (overturned = 1, otherwise = 0)</td>
<td>0.186</td>
<td>1.915</td>
<td>-0.0634</td>
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<td>Fixed object struck (hit fixed object = 1, otherwise = 0)</td>
<td>0.158</td>
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<td>Animal related crash (animal involved = 1, otherwise = 0)</td>
<td>-0.246</td>
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<tr>
<td>Environment</td>
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<tr>
<td>Clear weather (clear weather = 1, otherwise = 0)</td>
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<td>2.116</td>
<td>-0.0359</td>
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<tr>
<td>Rural environment (rural = 1, urban = 0)</td>
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<td>2.469</td>
<td>-0.0547</td>
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<tr>
<td>Model-specific attributes</td>
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<tr>
<td>Constant</td>
<td>-0.899</td>
<td>-2.843</td>
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<tr>
<td>$\mu_2$</td>
<td>1.356</td>
<td>36.778</td>
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<tr>
<td>$\mu_3$</td>
<td>2.403</td>
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Model Summary Statistics
- No. of observations: 1984
- Degrees of freedom: 12
- Log likelihood: -2002.326
- Restricted log likelihood: -2297.544
- $R^2$: 0.128
crashes involving animals were not as injurious as other types of crashes, also as expected by the authors. The authors investigated environmental factors such as weather conditions at the time of crash. The model indicated that injuries to older drivers tended to be more severe if the weather was clear. This may be so because older drivers tend to adjust their driving behavior, e.g., slow down, during adverse weather conditions. The effects of rural and urban environment were investigated by using an indicator variable for rural locations. The model indicated the presence of statistical evidence that rural crashes tend to be more severe compared to urban crashes. Other variables such as different light conditions and crash year (1990–1997) were investigated but not found to be statistically significant and hence not included in the model specification.

Finally, the authors tried to evaluate the effects of speed limit changes on some Iowa divided multilane highways during 1996 on the injury severity of older drivers. Indicator variables were created for crashes that occurred before and after 1996 on those specific highways where speed limits were changed. The model did not indicate statistically significant differences in the injury severity of older drivers before and after the increase in speed limit. The two indicator variables were excluded from the model specification to reduce the “noise” in the model. Although, there is evidence in the literature that increased speed limits result in more injurious crashes, it appears that the increase in speed limits on selected highways in Iowa did not increase injury severity of older drivers. It is clear, however, that injury severity of drivers in other age groups and/or multiple vehicle crashes were affected by the increased speed limits, as the Iowa Department of Transportation has documented an overall increase in fatalities, injuries, and total crashes following the speed limit change (17). This issue requires more in-depth research and collection of additional crash data after 1996 to verify a statistically significant trend.

SUMMARY AND DISCUSSION

The model indicated that several explanatory variables from various categories were important in predicting injury severity of older drivers involved in single-vehicle crashes. Advanced driver age, gender (male), absence of occupant restraint systems in vehicles, and use of alcohol contributed to severity of injuries. Among roadway characteristics, crashes occurring on curves in level terrain were more injurious. Injuries in farm vehicles were more severe compared to injuries in other vehicles. Crashes that resulted in overturned vehicles or crashes in which vehicles struck fixed objects were more injurious to older drivers. Animal-related crashes were less injurious than other types of crashes. Crashes occurring in a rural environment and crashes occurring under clear weather were also more injurious to older drivers. The modeling effort did not indicate evidence that older driver injury severity in single-vehicle crashes increased on selected Iowa highways due to an increase in speed limit.

The findings have several important implications for the safety of older drivers. Advancing age increases the propensity of more severe injury, and older drivers need to be aware of this when they are considering a reduction in driving. Transportation agencies may consider installing curve warning signs or using rumble strips on long sections of highways that are followed by curves to alert older drivers to oncoming curves in the road geometry. Older driver injuries in crashes involving farm vehicles need further investigation to assess causal factors behind the high level of injury severity. Transportation agencies may focus their attention on reducing crashes that involve overturned vehicles and crashes that involve vehicles hitting fixed objects. The effects of policy actions, such as an increase in speed limits, need further investigation.

This study did not explore variations in injury severity within subgroups of older drivers. Further, it did not compare crash injury severity of older drivers to that of younger drivers. Future studies may focus on investigation of subgroups of older drivers and comparison to injuries sustained by younger drivers.

ACKNOWLEDGEMENTS

The authors thank the Iowa Department of Transportation for providing the data used in this study. The opinions and views expressed in this paper are those of the authors and not necessarily of the sponsoring agencies.

REFERENCES


Safety Management in Iowa — After the Mandates

THOMAS M. WELCH AND MARY STAHLHUT

In 1991, the federal Intermodal Surface Transportation Efficiency Act mandated that each state form a safety management system work plan by October of 1994 to address surface modes of transportation. Iowa complied, and the Iowa Highway Safety Management System (SMS) structure was developed. When the mandate was withdrawn in 1997, Iowa continued the success of the SMS initiative through the commitment and collaboration of the diverse SMS membership and the strength of the comprehensive Iowa SMS vision. In November 1999, the Iowa SMS Coordinating Committee received one of the first FHWA Partnership in Excellence Awards for its multi-disciplinary efforts with state and local governments, private industry, FHWA, and other federal agencies. The Iowa SMS has sustained leadership in a number of areas and has reinvented itself to address current issues and facilitate appropriate solutions through cooperation and collaboration. This paper contains a summary of the Iowa SMS history, current SMS activities including the Iowa Strategic Highway Safety Plan, the “toolbox” approach to matching remedies with issues, and the Iowa SMS vision for Iowa’s future roadway safety. Key words: safety management system, multi-disciplinary plan

THE IOWA INITIATIVE

The Iowa Safety Management System (SMS) Coordination Committee was formed and began regular monthly meetings in February 1995. At that time a federal mandate was in effect requiring states to implement safety management systems.

The Iowa Department of Transportation (Iowa DOT) was designated as the focal point, and the Iowa DOT Office of Transportation Safety Director was identified as the coordination committee chair. This organization was retained in Iowa after the federal mandate was dropped in January of 1997.

Since the earliest planning stages of the Iowa SMS, the Iowa Department of Public Safety’s Governor’s Traffic Safety Bureau (GTSB) has partnered with the Iowa DOT’s Office of Transportation Safety to develop and sustain the Iowa SMS.

During its early years, the SMS coordination committee established communication and cooperation among its inter-disciplinary members, identified highway related safety problems, defined areas that could be improved, and established task forces to address these problems (see Table 1).

SMS members collaborate to develop and maintain a multi-disciplinary approach that provides a “toolbox” of strategies and ideas that may be selected and applied to transportation safety issues. Some of the tools involve hard engineering solutions. By contrast, some tools are less tangible and leverage education, aptitude, awareness, attitudes, and other human factors to resolve highway safety issues.

### TABLE 1 Iowa SMS Coordinating Committee Members and Plan Authors

<table>
<thead>
<tr>
<th>Authors</th>
</tr>
</thead>
<tbody>
<tr>
<td>American Association of Retired Persons (AARP) 55 ALIVE Program</td>
</tr>
<tr>
<td>American Public Works Association municipalities representative</td>
</tr>
<tr>
<td>American Automobile Association (AAA) Minnesota/ Iowa Cedar Rapids Police Department</td>
</tr>
<tr>
<td>Highway Safety Engineering Consultant</td>
</tr>
<tr>
<td>Federal Highway Administration, Iowa Division</td>
</tr>
<tr>
<td>Governor’s Traffic Safety Bureau, (GTSB) Department of Public Safety</td>
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<tr>
<td>Iowa County Engineer’s Association Emmett County</td>
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<tr>
<td>Iowa Department of Education</td>
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<td>Iowa Department of Elder Affairs</td>
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<tr>
<td>Iowa Department of Public Health</td>
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<td>Iowa Interstate Railroad</td>
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<tr>
<td>Iowa Motor Truck Association</td>
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<tr>
<td>Iowa State Sheriffs and Deputies Association – Story County Sheriff</td>
</tr>
<tr>
<td>Iowa Northland Regional Council of Governments</td>
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<tr>
<td>Iowa State Patrol, Department of Public Safety</td>
</tr>
<tr>
<td>Iowa Department of Transportation: Transportation Safety Local Systems Transportation Data Driver Services Motor Vehicle Enforcement Maintenance Operations Design Program Management Data Services Traffic Engineering Iowa State University Farm vehicle safety Iowa Fire Service Institute Safety Circuit Rider Center for Transportation Research and Education (CTRE) Iowa Traffic Control &amp; Safety Association National Highway Transportation Safety Administration, Region VII Retired Transportation Safety professionals State Farm Insurance</td>
</tr>
</tbody>
</table>

### IOWA SMS CONCEPTS

Some of the concepts employed by the Iowa SMS Coordinating Committee and plan authors are the following:

1. Highway safety is the shared responsibility of federal, state, and local levels of government. Each branch is organized and positioned to make substantial, unique contributions to the entire effort. The SMS approach helps leverage funding and other resources for the best results across the state.
2. Multidisciplinary safety management procedures such as those used by the Iowa SMS insure that comprehensive program development and delivery attains the goals of saving lives, reducing injuries, and putting public funds to their best possible use.

3. While SMS can play a key role in a number of initiatives, there is no intent for SMS to manage or control initiatives in Iowa’s transportation safety community.

4. SMS can serve as a catalyst or source of “synergy” that helps coordinate and maximize transportation safety efforts statewide.

5. SMS is not intended to sustain existing programs or provide ongoing funding for new programs.

6. Cost-effectiveness is critical to the success of this enterprise, and should be clearly established in the demonstration or pilot phases of the various emphasis areas. With credible documentation and history to demonstrate cost effectiveness, budget requests to fund the various strategies will be more realistic, more likely to succeed, and result in better public policy.

MISSION, VISION, AND GOALS

The mission of the Iowa SMS is to reduce human suffering and economic losses resulting from crashes on Iowa’s roadways through the identification of causes, resources, and safety implications of policy decisions.

The vision of the Iowa SMS is a state whose citizens enjoy the safest highway system possible. This system is achieved when service providers and citizens communicate needs, coordinate efforts, and cooperate across political and professional boundaries in the pursuit of the mission. The Iowa SMS is the tool for achieving and sustaining this vision and must:

· accommodate Iowa’s specific social, economic, and geographic conditions;
· foster an interdisciplinary approach to problem solving;
· provide a forum for leadership in safety management; and
· achieve continuous improvement.

The goal of the Iowa SMS is to reduce the number and severity of crashes on Iowa’s roadways by promoting systematic processes to identify, implement, and evaluate all opportunities for improvement relating to:

· highway planning, design, construction, maintenance, and operations;
· traffic and transportation law, law enforcement, and adjudication;
· emergency response, trauma patient care, and the educational activities of the health care community related to highway safety;
· other safety programs relating to vehicles, cargo, and people (included are special users groups such as older drivers, pedestrians, bicyclists, motorists, commercial motor carriers, and hazardous material carriers);
· integration with railroads and with public transportation (included are railroad-highway grade crossings and relevant data such as number of trains, crossing warning devices, FRA numbers for linkage with motor-vehicle crash files, etc.);
· information systems to accomplish the above-mentioned tasks and for prioritizing problems and effectively utilizing resources.

TASK FORCES AND PROJECTS

The membership of each task force is developed as issues and their related experts, funding, resources, policy makers, and stakeholders are identified.

The Access Management Task Force launched a statewide study of the effectiveness of access management techniques and a program to educate project decision makers and business owners on the benefits and economic impacts of access management.

The Speed Limit Task Force began its work in 1996. The task force continues to produce annual comprehensive summaries of speed-related crash data and risk assessment related to speed limits for use by state legislators.

The Emergency Medical Services Task Force was formed when an SMS member identified highway construction as a disruption to dispatch patterns. The results of their study help facilitate planning, communication, and understanding between local emergency providers and the central highway authority.

Emergency Response Information System (ERIS) is underway to add fire, rescue, and EMS response areas to Iowa’s safety data GIS. It will enhance communication among hundreds of urban and rural response agencies and with other segments of Iowa’s SMS.

The Statewide Traffic Records Advisory Committee (STRAC) was organized in June 1994 and is now an SMS standing subcommittee. It conducts strategic planning including the requirements for Section 411 funding, and it collaborates with Iowa’s National Model for transportation safety data, supported in part by the FHWA. (The State of Iowa was recently awarded the National Partnership for Reinventing Government Hammer Award for major re-invention in collecting, transmitting, and managing highway safety data in Iowa.)

The Red Light Running Task Force is studying the use of automated enforcement of traffic signal violations in Iowa. Local traffic safety organizations and law enforcement agencies are key members of this group.

Multi-disciplinary local traffic safety groups operate under the umbrella of SMS in several Iowa communities. Further development of these groups is an important goal for SMS leadership. This grassroots network of local problem identification and problem solving can be enhanced with SMS resources, and effective solutions can be replicated in other Iowa communities, thereby multiplying the success of local initiatives.

SMS and the Iowa Traffic Control and Safety Association help support education efforts and conferences for members of both groups and other Iowa transportation safety individuals.

The Pavement Marking Visibility for Older Drivers study is underway to assess the effectiveness of improved highway markings for older drivers.

The Older Drivers Task Force formed with representatives from a number of public and private entities and stakeholder groups to address sustaining competent driving, highway engineering accommodations, and related transportation concerns for Iowa’s aging population. Entering the year 2000, Iowa has a
growing number older drivers and an increasing need for policy and strategies to safely accommodate their transportation needs.

DEVELOPMENT OF THE IOWA STRATEGIC HIGHWAY SAFETY PLAN

SMS Committee members, transportation safety experts, and interested parties collaborated to write the August 1999 draft Iowa Strategic Highway Safety Plan identifying transportation safety issues with possible strategies and solutions. The plan will be used to help identify and focus resources on specific highway safety issues and collaborative strategies used in resolving them. The plan is conceived as a “living document” that will continue to change and evolve as a central source for highway safety discussion and strategy development for all of Iowa’s roadways. It serves as a “tool box” of strategies to apply in resolving specific issues facing Iowa’s transportation safety community.

The plan addresses traditional concerns for infrastructure as well as driver, occupant, vehicle, and post-crash responsibilities. Moreover, many initiatives within this strategic plan are built upon safety programs already in existence while others suggest new programs. Another important highway safety initiative, Intelligent Transportation Systems (ITS) will generally address a different set of issues, although both may investigate some of these issues and correlate their results.

The American Association of State Highway Transportation Officials’ Strategic Highway Safety Plan, dated September 1997, was used as a model for developing the Iowa plan. The AASHTO Plan contained six major topics with a total of 22 key emphasis areas. The original AASHTO plan can be viewed at http://safetyplan.tamu.edu.

In the Iowa Plan, some emphasis areas were deleted or combined, and five were added. The Iowa Plan contains now 25 key emphasis areas, with a list of proposals for each section (see Tables 2 and 3).

REVIEW OF THE IOWA STRATEGIC HIGHWAY SAFETY PLAN

This DRAFT plan has been distributed directly to over 700 entities and individuals in Iowa who are personally or professionally involved in transportation safety. Public response has been promoted through media coverage and a University of Northern Iowa public opinion survey will help assess Iowans’ highway safety priorities and their sensitivity to proposed strategies. Transportation safety peers in other states have also been invited to review the plan and comment on its content. The complete plan with an electronic comment form is available at www.state.ia.us/government/dot/safetyplan.

<table>
<thead>
<tr>
<th>TABLE 2</th>
<th>DRAFT Iowa Strategic Highway Safety Plan Table of Contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drivers</td>
<td>1. Instituting Graduated Licensing for Young Drivers</td>
</tr>
<tr>
<td></td>
<td>2. Ensuring Drivers are Fully Licensed and Competent</td>
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<tr>
<td></td>
<td>3. Sustaining Proficiency in Older Drivers</td>
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<td></td>
<td>4. Curbing Aggressive Driving</td>
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<tr>
<td></td>
<td>5. Reducing Impaired Driving</td>
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<td></td>
<td>6. Keeping Drivers Alert</td>
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<td></td>
<td>7. Increasing Driver Safety Awareness</td>
</tr>
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<td>8. Increasing Safety Belt Usage and Improving Air Bag Effectiveness</td>
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<td>Special Users</td>
<td>9. Making Walking and Street Crossing Safer</td>
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<td>10. Ensuring Safer Bicycle Travel</td>
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<td>11. Ensuring School Bus Safety</td>
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<td>12. Improving Motorcycle Safety and Increasing Motorcycle Awareness</td>
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<td>13. Making Truck Travel Safer</td>
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<td>14. Reducing Farm Vehicle Crashes</td>
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<td>15. Reducing Train-Vehicle Crashes</td>
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<td>16. Reducing Deer-Vehicle Crashes</td>
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<td>17. Implementing Road Safety Audits</td>
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<td>18. Accommodating Older Drivers</td>
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<td>19. Keeping Vehicles on the Roadway and Minimizing the Consequences of Leaving the Road</td>
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<td>20. Improving the Design and Operation of Highway Intersections</td>
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<td>21. Reducing Head-On and Across-Median Crashes</td>
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<td>22. Designing Safer Work Zones</td>
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<td>Emergency Response</td>
<td>23. Enhancing Emergency Response Capabilities to Increase Survivability</td>
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<td>Management</td>
<td>24. Improving Information and Decision Support Systems</td>
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<td>25. Creating More Effective Processes and Safety Management Systems</td>
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<thead>
<tr>
<th>TABLE 3</th>
<th>Sample of Section Proposals</th>
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<tr>
<td>Chapter 6 Keeping Drivers Alert</td>
<td>A. Assemble a synthesis of current research on cell phone usage and crash involvement.</td>
</tr>
<tr>
<td></td>
<td>B. Prepare a drowsy driving PSA and brochure</td>
</tr>
<tr>
<td></td>
<td>C. Ensure that driver fatigue elements are included in the new Iowa crash report form.</td>
</tr>
<tr>
<td></td>
<td>D. Ensure that ample information is in the Iowa Officer Reporting Guide</td>
</tr>
<tr>
<td></td>
<td>E. Ensure that driver alertness is emphasized in officer training at the Iowa Law Enforcement Academy.</td>
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<tr>
<td></td>
<td>F. Support engineering efforts to increase shoulder width, paved shoulders, and the use of rumble strips</td>
</tr>
<tr>
<td></td>
<td>G. Educate the public about the importance of shoulder rumble strips</td>
</tr>
<tr>
<td></td>
<td>H. Support legislation for headlight use during low visibility</td>
</tr>
<tr>
<td></td>
<td>I. Monitor and support ITS</td>
</tr>
<tr>
<td></td>
<td>J. Support the recommendations of the Rest Area Task Force’s Rest Area Needs Study</td>
</tr>
<tr>
<td></td>
<td>K. Increase the availability of passenger car rest areas</td>
</tr>
<tr>
<td></td>
<td>L. Monitor and support the Commercial Driver Hours of Service regulations</td>
</tr>
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</table>
From this base of information, ideas, and responses, the Iowa SMS Coordination Committee believes that an ongoing, comprehensive approach will marshal Iowa’s best strategies and leverage results gained from multi-disciplinary collaboration. This integrated approach will produce measurable results to significantly reduce deaths, injuries, health care costs, and other losses on our highways.

IMPLEMENTATION STRATEGIES

The strategies developed for the key emphasis areas are designed to address each area’s major problems or to advance effective practices by means that are both cost effective and acceptable to a significant majority of Iowans. Some strategies will apply existing federal programs to Iowa, supplement, or apply such programs more effectively. Other strategies involve enhancement of programs developed within the state. Still other strategies will require pilot projects to demonstrate benefits of implementing new strategies.

The SMS coordinating committee will periodically review the proposals made and determine when to initiate or fund them. Consideration may include: which proposals best address current critical needs; which have the best chance for success; which can leverage other funding sources; or which contribute the most safety benefit for the funding identified.

SMS members acknowledge that it can be difficult to compare strategies when the outcomes range from hard engineering changes to driver behaviors, training, education, and the quality of human life. SMS members expect that most decisions will result from a range of tangible factors and conditions rather than competing on a direct value comparison.

CONCLUSION

Perhaps the most rewarding aspect of Iowa’s Safety Management System, is strengthened cooperation and collaboration among its many many groups in helping to fill Iowa’s transportation safety “toolbox” with the most innovative and effective solutions available.

Iowa’s Safety Management System for highway safety has continued to succeed through the strength of its diverse membership and its vision for continually improving Iowa’s highway safety. Using a multi-discipline approach, SMS is able to identify issues, comprehensively test and evaluate strategies, and ultimately apply tangible solutions that leverage the most return from available resources.
Sign Vandalism—An Estimate of the National Cost

DUANE SMITH AND TIM SIMODYNES

Some of the most noticeable vandalism is that which is inflicted on our transportation system signage. This vandalism is not only costly to remove or repair, but it can be dangerous for the users. The Iowa Department of Transportation (Iowa DOT) and Iowa County Engineers Association (ICEA) initiated a grassroots effort to assess the national impact of sign vandalism. This paper presents one part of that effort, which was to estimate the annual national cost of repairing and replacing vandalized signs. The Iowa DOT entered into an agreement with the Center for Transportation Research and Education (CTRE), a center of Iowa State University, to conduct a national survey on sign vandalism. The survey was used to provide the raw data needed to develop the estimate. The surveys were distributed to city, county, and state transportation agencies. The objective of the survey was to identify sign vandalism rates that could be used to estimate the annual national cost of sign vandalism. The correlation between attribute pairs was measured using the simple linear regression. The analysis showed that acceptable rates existed for the number of signs per lane mile, the percent of signs that are vandalized, and the average cost per sign replacement. The rates were then used with the total lane miles of roads nationally to estimate the annual national cost at $274 million per year. Key words: cost, sign(s), survey, vandalism.

INTRODUCTION

Vandalism in general causes untold amounts of damage annually. Vandalism is seen in every aspect of our lives—on the transit bus, on railcars, in public rooms, and in other public places. One of the most noticeable areas is the vandalism that is inflicted on our transportation system signage. Everyone has known of signs that have been knocked down, stolen, written on with graffiti, shot, or damaged in some other way. This paper summarizes an effort to use a national survey to estimate the annual national cost of repairing and replacing vandalized signs.

Sign vandalism can include destruction of the sign or supports, mutilation of the sign face, or theft of the sign and/or supports. Past studies have been conducted and several articles written on the subject of sign vandalism. These efforts have estimated the national cost to be as much as $50 million annually (1). One study estimated as much as $1.5 million at the county level in Iowa alone (2). This vandalism is not only costly to remove or repair, but it can be dangerous for the roadway users who depend on proper signage for traffic control and guidance (3, 4). One of the first steps to combating the problem of sign vandalism is to determine the severity of the problem by quantifying the annual cost of repairing and replacing vandalized signs.

Background

The Iowa Department of Transportation (Iowa DOT) and the Iowa County Engineers Association (ICEA) initiated a three-step effort to define the sign vandalism problem, identify corresponding solutions or initiatives, and set a course of action for the future. This paper focuses on the first step and specifically on estimating the national cost of sign vandalism. A full report on the entire sign vandalism project is expected in early 2000.

Objectives

This portion of the sign vandalism project was concerned with defining the sign vandalism problem. The work presented here has two objectives:

· Identify some general rates related to signs and sign vandalism that could be used to estimate the extent of sign vandalism at various jurisdictional levels.

· Use the identified rates to estimate the annual national cost of sign vandalism.

METHODOLOGY

In order to assess the national cost impact of sign vandalism, a national survey was designed, distributed, and evaluated. The results of that survey were then used to estimate a national cost. The Center for Transportation Research and Education (CTRE), a center of Iowa State University, provided staff to conduct the survey. The survey went to city, county, and state transportation agencies and asked for raw data related to signs and sign maintenance. The data were used to develop general rates, which, in turn, were used to estimate the cost of sign vandalism. Although the survey was not designed to be statistically robust, it did serve the purpose of identifying some useful signing and sign vandalism rates.

Survey Content

The survey was designed as a one-page, self-addressed, postage-paid questionnaire with ten questions relating to signs and sign vandalism. Three of the questions related to the types of vandalism experienced and the type of sign management program used. The other seven questions asked for the following information for the respondent’s jurisdiction:

· number of lane miles

· number of signs in place
- total replacement cost of signs in place
- number of signs vandalized annually
- annual cost to repair of replace vandalized signs
- population represented
- number of sign maintenance employees

Survey Distribution

The survey was distributed to as large an audience as possible as shown in Figure 1. The Local Technical Assistance Program (LTAP) national communication network was the backbone for survey distribution and identifying state contacts. There is an LTAP center in each state along with six tribal centers across the nation. The mission of LTAP is to provide technical assistance to local governments. With this close association to local governments, LTAP centers were used to identify local government officials in each state who would respond to the survey. The Iowa LTAP center is located within CTRE. Iowa LTAP/CTRE provided 25 surveys to each LTAP center. Each LTAP center was asked to identify 25 transportation agencies or officials in their respective state and distribute the survey to them. Iowa LTAP/CTRE provided the surveys, the postage for each LTAP center mailing, and the completed survey return postage.

ANALYSIS

Simple Linear Regression

Simple linear regression was used to identify relationships, or rates, between pairs of variables provided from the survey. Simple linear regression uses a best-fit linear equation \( y = a + bx \) to define the relationship between an independent variable, \( x \), and a dependent variable, \( y \). When \( a \) is equal to zero, the relationship is defined by the coefficient \( b \), and can be expressed as a rate. A graphical representation of the linear predictive model \( y = a + bx \) is shown in Figure 2.

For each of the relationships analyzed in this report, the dependent variable could not exist without the independent variable. Therefore, all regression lines were forced to pass through the origin \((a = 0)\). For example, if \( x \) represented the number of lane miles an agency maintained, and \( y \) represented the number of signs in place, one could expect that no signs would be in place if no lane miles were maintained, i.e. if \( x \) equals zero, then \( y \) must equal zero.

In addition to determining linear predictive models, \( R^2 \) values were calculated to determine the reliability of each relationship. \( R^2 \) is a number between zero and one that can be described as the proportion of variance in \( y \) attributable to the variance in \( x \). Table 1 shows the \( R^2 \) values for all the relationships that were analyzed.

![Figure 2 Graph of linear predictive model](image)

TABLE 1 \( R^2 \) Values for Attribute Pairs

<table>
<thead>
<tr>
<th>Dependent Variable</th>
<th>Independent Variable</th>
<th>Total Signs</th>
<th>Vandalized Signs</th>
<th>Lane Miles</th>
<th>Population</th>
<th>Employees</th>
</tr>
</thead>
<tbody>
<tr>
<td>Repair/ Replacement Cost</td>
<td>0.35</td>
<td><strong>0.76</strong></td>
<td>0.06</td>
<td>0.36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Signs</td>
<td><strong>0.79</strong></td>
<td>0.33</td>
<td>0.09</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vandalized Signs</td>
<td><strong>0.49</strong></td>
<td>0.28</td>
<td>0.26</td>
<td>0.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Bold type indicates an acceptable relationship
Although seven data fields were requested in the survey, only those shown in Table 1 resulted in data sets suitable for analysis. Due to the wording of the questions and inconsistent data collection methods, there seemed to be confusion on the part of the respondents in providing responses to some of the questions.

The results of the $R^2$ analysis indicated that three of the attribute pairs showed possible relationships ($R^2 > 0.48$):

- Repair/replacement costs vs. vandalized signs
- Total signs vs. lane miles
- Vandalized signs vs. total signs

Data point plots were prepared for each of the relationships and are shown in Figures 3 through 5. The data points in Figures 3 through 5 are clustered close to the origin (0,0), which indicates that more responses were received from smaller jurisdictions than larger jurisdictions.

The results of the linear regression analysis for the complete data set as well as the jurisdictional subsets are shown in Table 2.

**Survey Results**

The analysis of these three pairs provided rates (coefficient $b$) that could be used to estimate the cost of sign vandalism. The data set was then divided into three jurisdiction levels (city, county and state). Linear regression models were then created for all three data pairs at each jurisdiction level. This resulted in $b$ coefficients that could be written as rates for each jurisdiction level.

**TABLE 2 Linear Regression Results - $R^2$ Values and Linear Coefficients**

<table>
<thead>
<tr>
<th>Attribute pairs</th>
<th>Jurisdiction level</th>
<th>$R^2$</th>
<th>Linear coefficient, $b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Signs vs. Lane Miles</td>
<td>All</td>
<td>0.79</td>
<td>14.61</td>
</tr>
<tr>
<td></td>
<td>City</td>
<td>0.72</td>
<td>69.85</td>
</tr>
<tr>
<td></td>
<td>County</td>
<td>0.26</td>
<td>12.19</td>
</tr>
<tr>
<td></td>
<td>State</td>
<td>0.86</td>
<td>14.40</td>
</tr>
<tr>
<td>Vandalized Signs vs. Total Signs</td>
<td>All</td>
<td>0.49</td>
<td>0.026</td>
</tr>
<tr>
<td></td>
<td>City</td>
<td>0.73</td>
<td>0.038</td>
</tr>
<tr>
<td></td>
<td>County</td>
<td>0.19</td>
<td>0.018</td>
</tr>
<tr>
<td></td>
<td>State</td>
<td>0.51</td>
<td>0.027</td>
</tr>
<tr>
<td>Rep./Repl. Cost vs. Vandalized Signs</td>
<td>All</td>
<td>0.76</td>
<td>35.68</td>
</tr>
<tr>
<td></td>
<td>City</td>
<td>0.74</td>
<td>28.07</td>
</tr>
<tr>
<td></td>
<td>County</td>
<td>0.52</td>
<td>21.97</td>
</tr>
<tr>
<td></td>
<td>State</td>
<td>0.82</td>
<td>42.18</td>
</tr>
</tbody>
</table>
The linear coefficients \((b)\) from Table 2 can also be expressed as rates. The following rates are taken from the complete data set and can be applied at the national level:
- 15 signs per lane mile
- 2.6 percent signs vandalized
- $36 for cost of sign repair/replacement (preliminary)
- $85 for cost of sign repair/replacement (final)

The revision of the sign repair/replacement cost will be described later.

**NATIONAL SIGN VANDALISM COST ESTIMATE**

The rates identified by the survey analysis were then used in the following equation to estimate the annual national cost of sign vandalism:

\[
\text{cost} = (\text{signs per lane mile})(\% \text{ of signs vandalized})(\text{cost of repair/replacement})(\text{lane miles})
\]

The following equations can be used for various levels of government by inserting the rates determined from the analysis:
- national cost = \((15 \text{ signs / lane mile})(2.6\% \text{ are vandalized})(\$36/\text{ sign})(\text{national lane miles})\)
- city cost = \((70 \text{ signs / lane mile})(3.8\% \text{ are vandalized})(\$28/\text{ sign})(\text{city lane miles})\)
- county cost = \((12 \text{ signs / lane mile})(1.8\% \text{ are vandalized})(\$22/\text{ sign})(\text{county lane miles})\)
- state DOT cost = \((14 \text{ signs / lane mile})(2.7\% \text{ are vandalized})(\$42/\text{ sign})(\text{State lane miles})\)

If the number of lane miles are known, these equations can be used to make a rough estimate of the cost of sign vandalism for a single jurisdiction or the entire country.

**Sign Cost Revision**

A second activity conducted during the study was a National Workshop on Sign Vandalism. The national survey analysis had been completed prior to the workshop. As a part of the program for the national workshop, the results of the survey analysis were presented to the attendees and each of the identified rates were discussed and evaluated. The rates of 15 signs per lane mile and 2.6 percent of signs vandalized were accepted as reasonable numbers. However, the cost of $36 per sign was not considered realistic. The attendees felt that the number was too low and all agreed to adjust it to $85 per vandalized sign. The lower cost figure ($36 per sign) was a result of the survey not being specific enough in asking what should be included in the cost requested. This newer figure ($85 per sign) more than doubled the national sign vandalism cost estimate.

The revised $85 sign cost should be used in each of the previous equations to estimate sign vandalism costs for any jurisdiction level.

Using the revised cost per vandalized sign results in the following calculation of national cost:

\[
\text{Cost} = (15 \text{ signs / lane mile})(2.6\% \text{ are vandalized})(\$85/\text{ sign})(8,269,205 \text{ lane miles}) = \$274,124,145 \text{ annually.}
\]

Although a larger sample size would have led to more reliable results, an annual national cost estimate was calculated. It should also be noted that the $274 million estimate does not include the additional costs associated with crashes and delays caused by missing or vandalized signs.

**ACKNOWLEDGMENTS**

The authors would like to recognize several people who helped make this report possible.

The Sign Vandalism Planning Committee provided the guidance and support throughout the study and also helped secure financial support. The steering committee includes: Sergeant Shane Antle, Iowa State Patrol; John Ballantyne, Federal Highway Administration (FHWA); Wil Brannon, Pierce County, Washington; Jim DeLozier, Taylor County, Iowa; Captain Jim Ehresman, Iowa State Patrol; Jim Ellison, Pierce County, Washington; Jim Hogan, FHWA, Iowa; Mike Ireland, American Traffic Safety Services Association; J. Michael Laski, Iowa Governors Traffic Safety Bureau; Tom Meyers, Creston, Iowa; Dale Picha, Texas Transportation Institute; and Tim Simodynes, Iowa Department of Transportation. Jim DeLozier, in particular, pushed us all to make a difference through his continual efforts and passion against sign vandalism.

Zach Hans, CTRE, provided the statistical analysis and helped formulate the estimates. Ian MacGillivray and Walt McDonald also provided guidance from the Iowa DOT.

Ray Griffith and Roger Port from the FHWA provided the national level support and assisted with the National Workshop. The National Workshop facilitators were: Pat Weaver, Kansas University; Greg Schertz, FHWA; and Bill Reitenger, National Highway Transportation Safety Association.

And finally we would like to thank the many financial sponsors including the FHWA, Iowa DOT, Iowa County Engineers Association, Washington DOT, Missouri DOT, and Tennessee DOT.

**REFERENCES**

Geotextiles and Loess: Long-Term Flow

Paul Denkler, John Bowders, and Erik Loehr

The ability of a geotextile to separate soils having varying grain size distributions while still allowing for flow makes them ideally suited for erosion control and filtration applications. A well-engineered and constructed silt fence will satisfy three design criteria: adequate permeability, soil retention, and soil compatibility. This project is focused on soil-geotextile compatibility. Several test methods exist to evaluate soil-geotextile compatibility. The Long-Term Flow (LTF) test, in which soil is placed above the candidate geotextile and a constant head of water is applied above the soil was used in this study. Water flows through the soil/geotextile for a long period to determine the compatibility condition of steady flow or non-steady flow rates of excessive clogging or soil piping. Nine geotextiles supplied for the silt fence market are being tested. Mid-Missouri loess is the candidate soil. Test results to date indicate soil-geotextile compatibility, i.e., the geotextiles do not clog excessively nor do the loess particles pass through the geotextiles. However, the flow rates are low initially and subsequently decrease, becoming steady after 2,000 hours of flow. While not clogging the geotextile per se, the subsequent decrease in flow rate may contribute to the geotextiles having an insufficient flow capacity to be effective silt fences or filters in loess. Key words: loess, silt fence, excessive clogging, piping, soil-geotextile compatibility, filtration.

INTRODUCTION

Erosion accounts for 3.3 billion metric tons of soil loss per year in the US (1). Fifteen percent or 500 million tons of that loss is attributed to construction sites, many of which are road, rail and other transportation sites. During construction, it is often not practical to cover all the open soil to prevent erosion, and a rainstorm can create a significant amount of sediment-laden run-off. This sediment load can flow off the property and pollute native waterways (fish habitat), clog sewers, collect on other property, and result in the loss of valuable topsoil. Erosion is particularly critical in the Missouri river system, a region known for its wind-deposited, highly erodible loess soil. Significant structural damage to infrastructure is attributed to erosion, and the United States Environmental Protection Agency’s National Pollutant Discharge Elimination System (NPDES) regulations now cover non-point sources, including sediment load to surface waters (2).

Geotextiles offer a solution for controlling runoff and sedimentation and can significantly reduce river, lake and stream pollution from unwanted sediment. A well-designed silt fence (Figure 1) will initially screen silt and sand particles from the runoff water forming a soil filter and reducing the ability of water to flow. The initial clogging of the geotextile creates a pond of relatively still water that serves as a sedimentation basin to collect the suspended soil from the runoff water (Figure 1b). A silt fence must retain water long enough for suspended particles to settle out while still maintaining adequate permeability to prevent overtopping. Two concerns in the use of geotextile silt fences are the potential of the geotextile to excessively clog and prevent flow through the fence and for the soil to pipe or pass through the geotextile.

A geotextile that neither clogs nor pipes soil fines is considered compatible with the soil in question. Silts and other fine soil with little cohesion have been notably problematic and exhibit poor compatibility (3). Loess, a low or non-cohesive, highly erosive soil located throughout the Midwest falls in this “problematic” category. The objective of this study is to evaluate the long-term filtration compatibility of geotextiles with loess. Nine geotextiles, marketed for the silt fence applications are being tested for compatibility with a central Missouri loess.

METHODS AND MATERIALS

Several test methods have been established to evaluate the long-term flow compatibility of geotextile-soil systems (3). The Long-Term Flow (LTF) test is being used in this study, see Figure 2. In this procedure, soil is placed above the candidate geotextile and a supply of water under constant head is applied above the soil.

Flow is continuous and flow rate measurements are taken periodically. The flow rates are examined for changes over time. The resulting flow rates are plotted as shown in Figure 3.

The time-to-transition, shown in Figure 3 as t, is the time where the soil-geotextile system will begin its field-simulated behavior (4). The initial slope, m1, is due to the densification of the soil due to the downward flowing water and is not of direct interest if only excessive clogging or piping of the geotextile are of concern (3). If after this time, the slope of the curve becomes steady (zero), the geotextile is considered compatible with the soil. If the initial or terminal slope, m2, becomes positive the soil is considered to be piping through the geotextile. If the slope continues to be negative, the soil is considered to be excessively clogging the geotextile. For the latter two cases, the geotextile may not be suited, i.e., able to meet its design criteria of adequate permeability and/or soil retention for this type of soil. Tests typically require about 1,000 hours of flow to establish the terminal slope shown in Figure 3.

Test Setup and Procedure

A diagram of the apparatus used in this study is shown in Figure 2. Six test cylinders were designed and built in order to test a series of geotextiles simultaneously. The flanged test cylinders were constructed from two cylindrical pieces of clear plastic. Each test cylinder has an inside diameter of 100 mm (4 in). The candidate geotextile, wire mesh support screen and rubber O-ring are placed in the seat of the lower cylinder (Figure 2.A-A). A thin film of vacuum grease is applied to the flange of the lower cylinder to prevent leaks and bolted to the upper cylinder. A constant hydraulic head is maintained through the fixed inlet and outlet ports of the upper cylinder. The system...
hydraulic gradient can be varied by adjusting the elevation of the clear plastic tubing attached to the outlet port of the lower cylinder. The apparatus is back saturated with water from the outlet port of the lower cylinder to 25 mm (1 in.) above the geotextile. The site-specific soil is placed above the geotextile to a depth of 100 mm (4 in.). For loosely placed samples, the soil is placed with a scoop from the top of the upper cylinder in 25 mm (1 in.) to 40 mm (1 ½ in.) layers and leveled with a wooden rod. For compacted samples, the soil is placed in the same manner with each layer receiving 15 blows with a 25 mm (1 in.) diameter by 1 m (3.32 ft) long wooden rod (mass = 438 g) dropped 25 mm (1 in.) per blow. To begin the experiment, the upper cylinder is slowly filled with water from its inlet port until the water reaches the overflow port. Initial flow rates are measured, and the flow is continuous over a long period (up to 2,000 hours in these tests). Flow rate readings are terminated when sufficient definition to the terminal portion (slope m) of the flow rate vs. time curve is obtained (Figure 3).

**Geotextile Properties**

Nine geotextiles supplied for the silt fence market are being tested. The designation and description of the three geotextiles tested to date are provided in Table 1. All geotextiles are a woven polypropylene. Samples 1, 2 and 3 are different-styled silt fences produced by one manufacturer. Tests on materials by other manufacturers are ongoing. To date, twelve tests have been performed.
TABLE 2 Engineering Properties of Easely Loess Used in this Study

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Method</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural moisture content (%)</td>
<td>D2216</td>
<td>6.0</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>D854</td>
<td>2.64</td>
</tr>
<tr>
<td>Liquid Limit (%)</td>
<td>D4318</td>
<td>34.6</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>D4318</td>
<td>14.6</td>
</tr>
<tr>
<td>% Passing #200 sieve</td>
<td>D422</td>
<td>100</td>
</tr>
<tr>
<td>% &lt; 0.002 mm (clay fraction, %)</td>
<td></td>
<td>22</td>
</tr>
<tr>
<td>Maximum Dry Density (pcf)</td>
<td>D698</td>
<td>109.0</td>
</tr>
<tr>
<td>(Standard Proctor) (kN/m³)</td>
<td></td>
<td>17.1</td>
</tr>
<tr>
<td>Optimum Moisture Content (%)</td>
<td>D698</td>
<td>16</td>
</tr>
<tr>
<td>USCS Group Symbol</td>
<td>D2487</td>
<td>CL</td>
</tr>
<tr>
<td>USCS Group Name</td>
<td></td>
<td>Low Plasticity Clay</td>
</tr>
</tbody>
</table>

Soil Properties

Mid-Missouri loess is the candidate soil being tested in this study. In Missouri, approximately one-half of the state is covered by loess. The soil has unique characteristics including high stability and strength in its undisturbed, unsaturated state. However, the soil is highly erodible and becomes unstable once it becomes saturated (5). The engineering properties of the loess, taken from Easley Missouri, used in this study are shown in Table 2.

The soil was classified as a low plasticity clay (CL) in the USCS. The soil had an average nature moisture content of 6.0%. All of the particles passed the number 200 sieve (0.074 mm) and the clay size fraction was 22% by weight. The specific gravity was 2.64. The liquid limit was 34.6 and the plasticity index was 14.6. The maximum dry density, as determined by ASTM D698, was 17.1 kN/m³ (109.0 pcf) at an optimum moisture content of 16%.

RESULTS AND DISCUSSION

Twelve long-term flow tests have been performed to date using geotextile samples 1, 2 and 3 (Table 3). All soil samples were placed loose except trial 1A, which was compacted to a density of 13.7 kN/m³ (87.3 pcf). Densities of loosely placed samples ranged from 9.3 kN/m³ (59.2 pcf) to 11.6 kN/m³ (73.9 pcf). The moisture content of the soil for trial 1A was 20.0 percent. Remaining trials were tested at moisture content of 11.0 percent.

The results of the long-term flow tests are shown for each geotextile in Figure 4. Initial flow rates ranged from 0.016 to 0.073 L/sec/m² (1L/sec/m² = 0.024 gal/sec/ft²). This is about 4 orders of magnitude lower than the flow rates reported for the geotextiles alone (Table 1). Obviously, the loess dominates the flow rate, as we would expect. It is interesting to note that these flow rates are one to two orders of magnitude lower than those for mica silt as reported by Koerner 1998. Loess is a much finer-grained soil, and the lower flow rate is expected.

In all cases, the flow rates immediately begin to decrease with continuous flow of water through the soil-geotextile systems (Figure 4). The initial decrease (slope m₁, as defined in Figure 3) is due to densification of the soil from the effects of seepage forces. After a given time of flow, referred to as transition time (tₜ, the slope of the flow rate versus log time plot changes. At this point, it is assumed that all soil densification has taken place and changes in flow rate are primarily a function of the soil-geotextile compatibility, i.e., if the geotextile is clogging, flow rates continue to decrease, or if the flow rate is increasing, fines are piping through the geotextile. Both cases would be of concern for long-term performance of the geotextile with that particular soil.

In the cases tested, the transition times ranged from 360 to 2,000 hours (Table 3, Figure 4). Typical transition times are around 10 hours for granular soils and 200 hours for fine-grained soils (3). The longer transition times measured herein are thought to result from the nature of the loess – all particles less than 0.074 mm and very low cohesion. The larger the fine fraction and the lower the cohesion, the longer the time to transition.

After the time to transition, all of the geotextile-soil combinations showed a relatively steady flow rate. This finding indicates that the loess did not tend to clog or pipe through the geotextiles. From this perspective, the loess and the geotextiles tested are compatible. No measurable fines were collected from the effluent water. It must also be noted that the final, steady flow rates through the geotextile-loess systems were low and dramatically decreased from the initial flow rates. In all but one case, the flow rates decreased (from initial values) by more than 50% (Table 3). While the final flow rates were steady, the low magnitude could result in excessive ponding in the case of silt fence applications or in excessive porewater pressure buildup in the case of filtration applications.
FIGURE 4 Results for geotextile silt fence material and Easely loess
LESSONS LEARNED

Long-term flow tests have been completed on three of nine candidate geotextiles with loess. Several insights have been gained and lessons learned from the test results that extend to field applications and future considerations of geotextile applications in loess.

Test Results

- Transition times, \( t_{t} \), for geotextile-loess can be excessive (up to 2,000 hours).
- Final flow rates through the geotextile-loess system can be more than 50% less than the initial flow rate.
- No excessive fines passed through the geotextile.

Implications for the Field

- The tested geotextiles should perform well for silt fence in loess application.
- Geotextiles in filtration applications in loess may provide insufficient flow rates and could result in build up of excessive porewater pressures.
- Designers should evaluate required flow rates to assure excess porewater pressures do not build up, especially in filtration applications in loess.

Further Considerations

- Long transition times make routine laboratory testing prohibitive. It appears that as the percent fines increase, the time to transition increases. The effect of percent fines on time to transition should be evaluated. Designers could then predict time to transition simply based on the grain size distribution for a soil.
- The long-term flow test results are directly applicable to the field performance of geotextiles in filtration applications, i.e., wrapping aggregate in a “French Drain” application. However, the performance of geotextiles used as silt fence in loess should be evaluated using a specifically designed performance test for silt fences, such as ASTM D5141, “Determining Filtration Efficiency and Flow Rate of a Geotextile Silt Fence Application, Using Site Specific Soil”.

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The authors thank Ms. Anna Klousek for initiating the construction of the apparatus and collecting the geotextile materials. The following manufacturers have graciously supplied their products for testing: Amoco Fabrics and Fibers Company, Belton Industries, Inc., LINQ Industrial Fabric, Inc., and Webtec, Inc. This project is supported by the Civil and Environmental Engineering Department at the University of Missouri-Columbia.

REFERENCES

Construction Methods for Slope Stabilization with Recycled Plastic Pins

LEE SOMMERS, J. ERIK LOEHR, AND JOHN J. BOWDERS

A new method for reinforcement of surficial slope failures is currently being developed that utilizes reinforcing elements, similar to soil nails, manufactured from recycled plastics and other waste materials. Using recycled plastics has the advantage of providing reinforcing members with high resistance to degradation while providing a market for materials that might otherwise be landfilled. A key aspect of development of the new stabilization scheme is development and evaluation of equipment and methods for installing the plastic reinforcing members. In this paper, results of a series of laboratory and field tests performed to evaluate alternative installation techniques are presented and installation activities performed to construct a full-scale field demonstration are described. Results of development activities to date indicate that recycled plastic members can be reliably and efficiently installed to produce a cost-effective alternative to more conventional slope stabilization techniques. Evaluation of the suitability of recycled plastic pin stabilization schemes for effecting long-term stabilization is ongoing.

Key words: slope stabilization, recycled plastic, construction methods, soil improvement.

INTRODUCTION

Maintenance costs due to slope failures on public and private transportation routes amount to a significant portion of yearly expenditures for infrastructure maintenance programs. Costs for repair and maintenance of slope failures for U.S. highways alone have been estimated to exceed $100 million annually (1). A new technique for stabilizing slopes is being developed that uses Recycled Plastic Pins (RPPs) in procedures comparable to soil nailing and conventional piling (Figure 1) that will offer a cost-effective alternative to current slope repair techniques. RPPs are composite members of plastic, wood, and other waste products. RPPs are lightweight, can be produced in various sizes, are easily modified with conventional construction equipment, and have excellent resistance against chemical and biological attack as shown by laboratory testing summarized in Table 1.

A critical component for development of the technique is development of construction equipment and methods for cost-effective and reliable installation of the plastic members. While equipment for installation of reinforcing elements is available, the unique characteristics of recycled plastic pins as compared to more conventional reinforcing elements necessitates adapting currently available equipment for the purpose of installing plastic members. The criteria on which alternative construction methods and equipment were considered and evaluated for this work include:

- equipment must be simple, robust, and operable with typical construction labor,
- equipment must be readily available with minor changes to conventional equipment,
- installation rates must be sufficient to be cost effective,
- equipment must have versatility and mobility to access potentially difficult sites, and
- installation must occur with minimal damage to recycled plastic members.

The efforts undertaken to develop and evaluate potential installation methods are described in this paper. These activities were accomplished in two phases. Phase one consisted of a series of small-scale laboratory and field tests using both impact and pseudo vibratory driving methods to install reduced-scale RPPs. In phase two, full-scale 10 cm by 10 cm by 2.5 m RPPs were driven in field trials and at a demonstration site using variations of the pseudo vibratory installation method developed during small-scale testing.

REDUCED-SCALE INSTALLATION TESTS

A series of laboratory and field tests were performed on reduced-scale versions of the RPPs in order to evaluate alternative installation methods with minimal cost. Both impact and vibratory methods were evaluated in this phase of development. Impact driving was evaluated in the laboratory using a simple drop weight driving mechanism to drive 4 cm by 4 cm pins into a soil filled drum. Results of these tests showed the recycled material to be extremely resilient to driving stresses. However, installation rates were generally unacceptable. Vibratory driving was evaluated using a slightly...
modified 27 kg (60 lb) pavement breaker to drive reduced-scale pins at a field test site near Columbia, Missouri (Figure 2). These tests again demonstrated the resilience of the plastic members. In addition, penetration rates for the pseudo vibratory method far exceeded those observed in tests using the drop hammer system. The pseudo vibratory method was thus selected for subsequent trials on full-scale RPPs.

![FIGURE 2 Modified pavement breaker used for preliminary evaluation of vibratory installation methods](image)

### FULL-SCALE FIELD DRIVING TRIALS

Based on the success of the small-scale tests, a series of full-scale driving trials were conducted at a site near St. Joseph, Missouri using a scaled-up version of the pseudo vibratory mechanism. The site, which is located in the flood plain of the 102 River, is generally stratified into two strata. The upper layer consists of approximately 1.5 m (5 ft) of compacted low plasticity (CL) clay. This material overlies a natural alluvial deposit of highly plastic (CH) clay. The inplace dry densities of the site soils ranged from 1.4 ton/m$^3$ to 1.7 ton/m$^3$ (86 lbs/ft$^3$ to 107 lbs/ft$^3$). A total of seven 10 cm by 10 cm square pins of 1.2 m and 2.4 m length were driven at the site.

The driving mechanism used for the full-scale trials consisted of a modified Indeco MES 351 hydraulic breaker mounted on a rubber-tired 835 Bobcat® skid loader as shown in Figure 3. The hydraulic breaker was adapted by machining the breaker tool to create a 4 degree taper that would fit into a drive head adapter to form a compression fitting. The drive head adapter was machined from 152 mm (6 in) diameter round steel stock to form a 127 mm (5 in) receiver on one end to hold the end of the RPPs and a 4-degree tapered hole on the opposite end to accept the breaker tool.

![FIGURE 3 Pseudo vibratory driving system used for full-scale field driving trials](image)

Penetration rates observed during the field trials varied from a maximum of 3.7 m/min (12 ft/min) to a minimum of 0.24 m/min (0.8 ft/min) due to varying soil conditions at the locations of the test drives. The highest penetration rates were observed in the soft, highly plastic alluvial deposits with an average dry density of 1.4 ton/m$^3$ (86 lbs/ft$^3$). The lowest penetration rates were observed in locations of the dense, highly compacted clays of low plasticity with dry densities of up to 1.7 ton/m$^3$ (107 lbs/ft$^3$). Penetration rates more typically ranged from 0.31 to 1.2 m/min (1 to 4 ft/min) which was considered acceptable for cost effective installation.

The skid loader used for the field driving trials performed adequately for preliminary testing but had inherent limitations that preclude use of similar equipment for more typical applications. The relative short wheelbase of the skid loader makes it unstable and prone to rolling when working on slopes, particularly when operating the boom and hydraulic hammer at maximum height. In addition, the skid loader lacks sufficient headroom to drive full length (2.4 m) RPPs without pre-boring, which would add additional costs to installation.

### INSTALLATION AT FIELD DEMONSTRATION SITE

A field demonstration site north of Emma, Missouri was selected to demonstrate the constructability and evaluate the effectiveness of the RPP stabilization technique. The site is the embankment for the eastbound entrance ramp to Interstate 70 located approximately 60
miles west of Columbia, Missouri. The embankment slopes vary from 2.5:1 (horiz.:vert.) to 2:1 and have repeatedly failed for over a decade, requiring periodic repair up to five times annually. The embankment soils generally consist of alternating layers of soft to very stiff lean and fat clays intermixed with localized pockets of concrete rubble that has been deposited via side cast over several years in an attempt to stabilize the slope.

Four general slide areas were observed in the embankment just prior to the field demonstration as shown in Figure 4. Three slide areas, denoted S1, S2, and S3, are located along the south side of the embankment with an additional slide, denoted S4, on the north side of the embankment. Slide areas S1 and S2 were selected as test areas for installation of RPPs; slide area S3 will be used as a control section.

Pin installation activities at the field demonstration site were initiated in October 1999. The initial installation equipment used at the site consisted of an Okada OKB 305 1695 N-m (1250 ft-lb.) energy class hydraulic hammer mounted on a Case 580 backhoe as shown in Figure 6. This equipment proved unsuccessful for several reasons. The rubber-tired backhoe was difficult to maneuver on the slope and caused excessive rutting while trying to reach the top of the slope. Maintaining a fixed position during driving also proved difficult with the backhoe tending to slide down slope even with the outriggers placed thereby further damaging the slope and making driving pins with the correct alignment and placement extremely difficult. The average penetration rate obtained using this equipment was 0.7 m/min, which was significantly less than that obtained during the field trails. In addition, play in the backhoe boom and the inability to maintain precise alignment of the hammer and pin during driving resulted in numerous pins being broken during installation. Set-up time between installation of pins was also excessive due to difficulty in navigating the slope and the need to constantly reposition the equipment. As a result of these problems, the rate of installation (including time for set-up and repositioning) averaged only 10 m/hr. This rate was deemed unacceptable and installation was halted.

A total of 317 RPPs were installed in slides S1 and S2 during October and November 1999. The pins were installed in a 0.91 m (3 ft) staggered grid with every other row offset by 0.46 m (1.5 ft) as shown in Figure 5. Pins were driven perpendicular to the face of the slope in slide S1; pins were driven vertically in slide S2. Not all of the RPPs could be driven to the full 2.4 m (8 ft) length due to the presence of concrete rubble from previous repair attempts (Figure 5). Conditions at the site were generally dry throughout construction allowing for maximum traction and maneuverability.

Installation at the field demonstration site resumed in early November 1999, using a Davey-Kent DK 100B crawler mounted drilling rig supplied by the Judy Company of Kansas City, Kansas as shown in Figure 7. The rig is equipped with a mast capable of 50-degree tilt from vertical forward, 105-degree tilt backward, and side-to-side tilt of 32 degrees from vertical. This rig offered numerous advantages over previously used equipment in that the drilling mast ensures that the hammer and pin remain aligned during driving without requiring any movement of the chassis. In addition, the crawler-mounted rig was much easier to maneuver on the slope, thereby reducing set up time between pins. The rig was equipped with a Krupp HB28A hydraulic hammer drill attached to the mast providing a maximum of 400 N-m (295 ft-lbs) of energy at a maximum frequency of 1,800 blows/min. The hammer energy is further augmented by a push/pull of 8,165 kg (18,000 lbs) supplied by the drill mast.
Penetration rates (not including set-up time) and installation rates (including set-up time) measured during installation using the mast-mounted system are summarized in Table 2. The mast-mounted hammer clearly outperformed all previous installation equipment that was evaluated. Penetration rates for pins driven perpendicular to the slope reached 3.0 m/min (10 ft/min) and averaged 1.6 m/min (5.2 ft/min). Penetration rates for pins driven vertically were only slightly lower reaching a maximum of 2.9 m/min (9.6 ft/min) and averaging 1.3 m/min (4.1 ft/min). Installation rates were also dramatically faster than observed previously because of reduced set-up times, reaching a maximum of 37.8 m/hour (124.0 ft/hour) at peak production. The average installation rate for installation of all pins was 25 m/hour (80 ft/hour). Installation rates generally increased during installation of pins for each slide as experience was developed indicating that installation rates for future installations may be closer to the maximum rates achieved for the field demonstration.

**TABLE 2  Summary of the Penetration and Installation Rates for Mast-Mounted Hammer on Slope Failures S1 and S2**

<table>
<thead>
<tr>
<th>Rate</th>
<th>Rate</th>
<th>Rate</th>
<th>Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration</td>
<td>Installation</td>
<td>Penetration</td>
<td>Installation</td>
</tr>
<tr>
<td>Rate</td>
<td>Rate</td>
<td>Rate</td>
<td>Rate</td>
</tr>
<tr>
<td>m/min</td>
<td>m/hr</td>
<td>m/min</td>
<td>m/hr</td>
</tr>
<tr>
<td>(ft/min)</td>
<td>(ft/hr)</td>
<td>(ft/min)</td>
<td>(ft/hr)</td>
</tr>
<tr>
<td>Average Rate</td>
<td></td>
<td>Average Rate</td>
<td></td>
</tr>
<tr>
<td>1.58</td>
<td>29.26</td>
<td>1.25</td>
<td>20.73</td>
</tr>
<tr>
<td>(5.18)</td>
<td>(96.0)</td>
<td>(4.10)</td>
<td>(68.0)</td>
</tr>
<tr>
<td>Maximum Rate</td>
<td></td>
<td>Maximum Rate</td>
<td></td>
</tr>
<tr>
<td>3.05</td>
<td>37.80</td>
<td>2.93</td>
<td>29.87</td>
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<tr>
<td>(10.0)</td>
<td>(124.0)</td>
<td>(9.60)</td>
<td>(98.0)</td>
</tr>
<tr>
<td>Minimum Rate</td>
<td></td>
<td>Minimum Rate</td>
<td></td>
</tr>
<tr>
<td>0.04</td>
<td>18.0</td>
<td>0.16</td>
<td>10.06</td>
</tr>
<tr>
<td>(0.12)</td>
<td>(59.0)</td>
<td>(0.53)</td>
<td>(33.0)</td>
</tr>
</tbody>
</table>

Limitations of the Davey-Kent drilling rig necessitated that pins installed in a vertical alignment were driven with the rig being backed up the slope. While not critical, this feature did result in slightly lower installation rates for pins driven vertically as compared to pins driven perpendicular to the face of the slope (Table 2). An alternative rig, the Crawlair ECM-350 extendible boom rig manufactured by Ingersoll-Rand, capable of driving pins vertically from a forward position was also used at the site. The Crawlair rig was equipped with an EVL-130 195-kg air hammer operating at a maximum frequency of 2100 blows/min attached to a chain drive capable of 1362 kg (3000 lb.) of down force. While the rig was much more maneuverable than the Davey-Kent rig, the hammer and drive system lacked the power and down force necessary to achieve acceptable penetration rates and its use was discontinued after a few attempts.

Overall, installation of the recycled plastic pins using the mast-mounted hammer system worked extremely well and the total cost of installation was competitive with other slope stabilization methods. A total of 317 pins were driven in slightly over four days for a total installation cost of $5,250 including labor. Comparison of the drill mast system verses the backhoe-mounted system revealed the mast system was much more effective and accurate and required less skill to operate. The crawler system also caused much less damage to the slope than the rubber-tired equipment. The crawler system did become marginally stable when operating on the steepest parts of the embankment and had to be tethered to the top of the slope in some locations.

**FUTURE ENHANCEMENTS TO DRIVING SYSTEM**

Several enhancements of the installation equipment are currently being considered. One alternative being considered is to mount the mast system on equipment with booms to expand the reach of the driving system. Candidate equipment for this purpose includes crawler-mounted excavators (track-hoes) and extendible boom excavators (grade-alls). The additional reach provided by such equipment will allow the equipment to remain off of the slope during installation which will further limit damage to the slope and reduce set-up time. An excavator/grade-all mounted mast system will also have greater swing range than the track mounted system allowing a larger number of pins to be driven without movement of equipment.

Several mechanical problems slowed progress during installation at the demonstration site. The connection between the hammer and the pins proved particularly troublesome as several different connections failed during installation. In the early stages of installation with the mast-mounted hammers, a welded drive head was used to receive the drill bit connector allowing the transfer of energy from the hammer to the RPPs. The repetitive impacts from the hammer inevitably caused the welded connections to fail. While several spare connectors were kept on site, construction was stopped on two occasions to permit re-welding of the connectors. In the latter stages of installation, the connection used during the field driving trials was adapted for use with the mast-mounted hammers. This connection performed much better than the welded connections but eventually failed due to incompatibility between the steel used for the connection and the steel used in the hammer. Modifications are currently in progress developing a more reliable drive head connector.

Extensive instrumentation was installed during construction to permit the performance of the slope to be monitored over time to establish the effectiveness of the plastic reinforcing members for stabilizing the slope. Instrumentation monitoring is expected to continue for several years. Results of instrumentation measurements will be presented in forthcoming papers.

**SUMMARY**

A new technique for slope stabilization is being developed that used Recycled Plastic Pins (RPPs) for reinforcement of unstable slopes. A particularly important part of this development has been consideration and evaluation of methods and equipment for...
installation of recycled plastic members into slopes. Activities undertaken in this work include a series of reduced-scale laboratory and field tests to evaluate the performance of both impact and pseudo vibratory driving mechanisms, a full-scale field driving trial, and installation of over three hundred plastic pins at a field demonstration site. The results of this evaluation indicate that a mast-mounted vibratory hammer system can produce efficient, reliable, and cost effective installation of recycled plastic members. The mast-mounted driving system was clearly superior to the other installation techniques evaluated to date.

ACKNOWLEDGEMENTS

The authors greatly appreciate the collaboration and spirit of Mr. Kurt DeRuy, Tamko Composite Products (formerly DuraBoard, Inc.) for supplying the recycled plastic pins for this work. The cooperation of John Bestgen and Bill Coats, Bestgen Construction, St. Joseph, Missouri, and Pat Carr of the Judy Company, Kansas City, Kansas, is greatly appreciated. Funding for this effort has been provided through the Research, Development, and Technology Division of the Missouri Department of Transportation, Director Jim Murray. The active participation and collaboration of project liaison, Mr. Tom Fennessey of the Soils and Geology section of the MoDOT is especially appreciated. The opinions, findings, and conclusions expressed in this publication are not necessarily those of the Missouri Department of Transportation or the Federal Highway Administration. This document does not constitute a standard, specification, or regulation.

REFERENCES

Service Life of Iowa Bridge Decks Reinforced with Epoxy-Coated Bars

FOUAD S. FANOUS AND HAN-CHENG WU

In an effort to minimize corrosion of the reinforcement in bridge decks, the Iowa Department of Transportation started using epoxy-coated rebars as top reinforcing mat around 1976. However, the presence of cracks in bridge decks has raised some concerns among bridge and maintenance engineers. The impact of deck cracking on the service life of a bridge deck was investigated herein. This was accomplished by collecting core samples from 80 bridges, calculating the chloride content in these cores, developing a relationship for chloride infiltration through the deck, examining the condition of several rebar samples, and developing a rebar rating-age relationship, and estimating a bridge deck service life. No signs of corrosion were observed on the rebars collected from uncracked locations. In addition, no delaminations or spalls were found on the decks where bars at cracked location exhibit some signs of corrosion. Considering a corrosion threshold for epoxy-coated rebars that range from 3.6 lb/yd³ to 7.2 lb/yd³, the predicted service life for Iowa Bridge decks was over 50 years. Key words: epoxy-coated bars, bridge deck, durability, corrosion, chloride.

INTRODUCTION

The problem of corrosion of the reinforcement in concrete bridge decks due to the intrusion of chloride ion resulting from the use of deicing salts was recognized in the mid 1970s. In a bridge deck, a corrosive reinforcement expands its volume by 3 to 6 times and eventually could cause delamination and spall of the surrounding concrete. This could induce cracking, delamination, and spalling of the surrounding concrete. The length of this stage is proposed. To prevent the reinforcing steel from corrosion, epoxy-coated rebars (ECR) were first used in the construction of a four-span bridge deck over Schuylkill River in Pennsylvania in 1973. Since then, ECR have been the most widely used corrosion protection method in bridge components in the United States.

Although the performance of ECR in corrosive environments is thought to be superior to typical black reinforcing bars, presence of cracks in bridge decks has caused some concern as to the condition of the ECR in these areas. This paper summarizes the results of an investigation with the objectives of determining the impact of bridge deck cracking on deck durability and approximately estimating the remaining functional service life of a bridge deck.

CORROSION PROCESS

Corrosion of reinforcing steel in concrete can be modeled in a two-stage process. The first stage is known as initiation or incubation period in which chloride ion transport to the rebar level. In this stage, the reinforcing steel experiences negligible corrosion. The time, T₂, required for the chloride concentration to reach the threshold value at the rebar level can be determined by the diffusion process of chloride ions through concrete following Fick’s second law. In the second stage, which is known as active and deterioration stage, corrosion of reinforcing steel occurs and propagates resulting in a noticeable change in reinforcing bar volume. This could include cracking, delamination, and spalling of the surrounding concrete. The length of the second stage, T₃, depends on how fast the corroded reinforcing bars deteriorate resulting in an observable distress. To the authors’ knowledge, no information regarding the prediction is available in the published literature. However, in this work, an approximate length of this stage is proposed.

ESTIMATING BRIDGE DECK SERVICE LIFE

Estimation of bridge deck durability involves defining the time required for rehabilitation. For a bridge deck, the end of functional service life is reached when severe deterioration occurs. An intensive opinion survey among 60 bridge engineers to quantify the end of functional service life was conducted, and the results were documented in reference. The study concluded that it is likely that the end of functional service life for concrete bridge decks is reached when the percentage of the worst traffic lane surface area that is spalled, delaminated, and patched ranges from approximately 9% to 14%. In addition, reference documents that based on current local practices, it is likely that the end of functional service life for concrete decks is reached when the percentage of the whole deck surface area that is spalled, delaminated, and patched ranges from 5.8% to 10.0%.

The diffusion-spalling model is among the models that are available to estimate the service life of a bridge deck. This method utilizes the concepts of a chloride diffusion period plus a deterioration period to determine when to rehabilitate a bridge deck. The length of the diffusion period can easily be calculated using Fick’s second law. This can be accomplished if the surface chloride exposure, the corrosion threshold, the mean and the standard deviation of the cover depth, and the chloride diffusion constant are known. These variables can be different for bridges from state to state or even among bridges in one state.

The diffusion-spalling model is often used to assess corrosion of black reinforcing bars. A similar approach was used herein to
estimate the service life of bridge decks constructed using ECR. However, a higher chloride threshold than that used for black steel bars and a longer deterioration stage need to be considered.

CORROSION THRESHOLD

For black bars, the corrosion threshold at the reinforcing steel level was determined to be 0.2% of weight of the cement content of concrete (4,5). Cady and Weyers (6) estimated the corrosion threshold for unprotected reinforcement to be 1.2 lb/yd³ (0.73 kg/m³) of concrete based on 6½ sacks of cement per cubic yard of concrete. However, it is believed that the use of ECR will delay the time required for initiating corrosion. As a result, the corrosive threshold should be higher than that for the black steel bar. A corrosive threshold for ECR of 3.6 lb/yd³ (2.19 kg/m³) has been suggested (7).

CHLORIDE IONS INGRESSION IN CONCRETE

Fick’s second law (2) to determine the length of the initiation stage, i.e., time T₁, it takes chloride ions to migrate through a bridge deck to reach the top reinforcing steel in an isotropic medium can be expressed as:

\[ C(x,t) = C_o \left[ 1 - \text{erf} \left( \frac{x}{2\sqrt{(D_{ac}t)}} \right) \right] \]  \hspace{1cm} (1)

where:
- \( C(x,t) \) = the measured chloride concentration at a desired depth,
- \( x; C_o = \) the surface concentration measured at 0.5 in below the deck surface, lbs/yd³;
- \( t = \) the time in years; and
- \( D_{ac} = \) the diffusion constant, in in/yr.

The \( \text{erf} (y) \) function is the integral of the Gaussian distribution function from \( \theta \) to \( y \).

CONCRETE COVER DEPTH

A sufficient cover depth can effectively provide corrosion protection for reinforcement. As reinforcing steel cover depth increases, the corrosion protection increases, and hence the initiating time, T₁, increases. However, to calculate a realistic time, \( T_1 \), for chloride ion to reach the reinforcing bar level, one must make use of the end of functional service life. Reference (3) recommended using an average cumulative damage of 11.5%, i.e., the average of 9% to 14%, damages in the worst traffic lane for a bridge deck as the end of functional service. In other words, one may assume that after the period of time elapsed, the chloride ions have been transported adequately to critically contaminate 11.5% of the top reinforcing steel (3). In this case, the depth, \( x \), used in Equation 1 needs to be calculated as:

\[ x = x_m + \alpha \sigma \]  \hspace{1cm} (2)

where:
- \( x_m = \) the mean reinforcing steel cover depth, in.;
- \( \alpha = \) the a standard normal cumulative distribution of 11.5%;
- \( \sigma = \) the standard deviation of the cover depth.

CHLORIDE CONTENT ANALYSIS

Four concrete powder samples were collected from each core for chloride content analysis. The locations of these samples were at 1/2 in. below the surface, midway between the first sample and the rebar level, at the rebar level and, and at about 1/2 in. below the rebar level. Powder samples from cracked cores were drilled from the uncracked quadrant to avoid splitting the cores in half. Drilling penetrated through the crack so that the sample contained powders collected from the cracked surface. The chloride concentration was tested in the material laboratory at Iowa State University using PHILIPS PW 2404 x-ray fluorescence spectrometer (8). This is a device that could be used to determine and identify the concentration of elements contained in a solid, powdered, and liquid sample (8).
SURFACE CHLORIDE AND DIFFUSION CONSTANTS

When reviewing the collected data, it was noticed that some data appeared to be unrealistic. For instance, the chloride analysis showed that, in some cases, a higher percentage of chloride existed at deeper locations than at shallower locations. These results were filtered out, and the remaining results were utilized to determine the coefficients $C_o$ and $D_{ac}$ needed to calculate the time for the corrosion initiation stage, $T_1$. The computational process involved the utilization of Matlab software to perform the iterative solution. Approximate ranges of $C_o$ and $D_{ac}$ were selected, and an iterative solution was carried out for several combinations of these two variables. The solution was terminated when the minimum of the sum of squared errors between the predicted and measured values was reached. This process yielded a surface chloride content, $C_o$, and diffusion constants, $D_{ac}$, for the state of Iowa bridges of 14 lb/yard$^3$ and 0.05, respectively.

CONDITION OF ECR VERSUS BRIDGE DECK AGE

According to the Federal Highway Administration (9), bridges are inspected every two years. Thus, it was reasonable to subgroup the rebar samples from the bridge decks according to age in two-year intervals. All samples collected from uncracked locations appeared to have no corrosion and were given a rating value of 5 or 4 (see Table 1). In contrast, 5%, 11%, and 3% of the reinforcing bar samples obtained from cracked locations were evaluated at rating of 3, 2, and 1 respectively, indicating some degree of corrosion of these rebar samples.

Since there is a range of possible values of reinforcing bar samples that can be rated at a specific rating condition, one would naturally be interested in some central value such as the average. However, there are different probabilities that different numbers of rebars in each time interval can be associated with different rating condition. Therefore, a weighted average, i.e., the expected value of the rating within each interval, would be more representatives rather than just using a straight average value ($\bar{r}$).

Having calculated an expected rating value, $E(r, j)$, where, $r$ is the rating condition within an interval, $j$, the Matlab program was utilized in conjunction with the second order polynomial model given in the following equation to develop a rebar-condition-age relationship (11).

$$r(t) = \beta_o + \beta_1 t + \beta_2 t^2 + \epsilon$$

(3)

where:

$r(t) =$ rebar rating at specified deck age $t$ in years,
$\beta_0 =$ a constant, and
$\epsilon =$ an error term that represents the degree of uncertainty between predicted and measured values.

For a new bridge deck, i.e., $t = 0$, the recorded rebar rating should always be 5, i.e., $\beta_2$ should equal 5. Although it is meaningful to force the intercepts to be 5, it is statistically unnecessary to force that since the raw data are empirical. For this reason, the second order polynomial regression analysis was made without forcing the intercept to be five. This regression yielded the following two relationships:
ILLUSTRATIVE EXAMPLE TO CALCULATE SERVICE LIFE OF A BRIDGE DECK

As previously mentioned, a corrosive threshold for ECR was defined as about 1.2 to 3.6 lb/yd\(^3\) (0.73 kg/m\(^3\) to 2.19 kg/m\(^3\)); and for black steel bar is 1.2 lb/yd\(^3\) (0.73 kg/m\(^3\)) (12). However, the data collected in this work revealed that an average chloride concentration of 7.5 lb/yd\(^3\) (4.56 kg/m\(^3\)) existed in locations where rebar samples had a rating of 3 (see Table 1). This is the condition at which corrosion becomes noticeable on ECR. Therefore, a corrosive threshold for ECR from range 3.6 lb/yd\(^3\) to 7.5 lb/yd\(^3\) (2.19 kg/m\(^3\) to 4.56 kg/m\(^3\)) was selected in this work.

Utilizing Fick’s Second Law, one can then calculate the time in which the chloride concentration at the rebar level reached the corrosive threshold for black or epoxy coated rebars. Assuming an additional time needed for spalling to take place in bridge decks, one can then determine the service life of a bridge deck. However, searching the published literature did not reveal any data related to the time required for spalling to occur in bridge decks with ECR. In this work, spalling is assumed to occur when approximately 60% or more of the rebar surface was corroded, i.e., when reinforcing bars reach a condition rating of 1. Using this information in conjunction with Equations 3 or 4, a time period of approximately 15 years can be estimated for ECR to deteriorate from condition rating 3 to 1. The following example illustrates how to incorporate the above assumptions to estimate the functional service life of a bridge deck in the state of Iowa.

Example: Given an Iowa bridge deck with \(C_0 = 14.0\) lb/yd\(^3\), and \(D_i = 0.05\) m\(^2\)/yr. End of functional life = 11.5% which is the average of 9% to 14% damage in the worst traffic lane (2). Average concrete cover depth \(T = 2.75\) in. associated with standard deviation \(s = 0.444\) in. The corrosive chloride threshold ranged from 3.6 lb/yd\(^3\) to 7.5 lb/yd\(^3\) for ECR. Assuming that 11.5% of the rebar is contaminated by the chloride ion and an \(a\) value of -1.2, the time required reaching the corrosive threshold and time to rehabilitation.

For a threshold of a 3.6 lb/yd\(^3\) and a cover depth, \(T\), of 2.75 in. (see Equation 2), one calculates a time, \(t\), of 38 years. Similarly, for a corrosion threshold of 7.5 lb/yd\(^3\), one estimates a time of 126 years. Assuming an additional 15 years for spalling to occur, the time required for spalling to occur would range approximately from 53 to 141 years. In comparison to black steel bar where a corrosive threshold of 1.2 lb/yd\(^3\) was used, one estimates a time, \(t\), of 17 years. Assuming an average time for spalling of 3.5 (13) years, time required to rehabilitation for unprotected steel equals \(17 + 3.5 = 20.5\) years. As can be noted, this example illustrates the significant increase in the service life of a bridge constructed with ECR.
Mechanized Vibration of Bridge Deck Concrete in Illinois

JEFFREY M. SOUTH

Mechanized methods of vibratory concrete consolidation were used on all or part of fourteen bridge decks in Illinois from 1996 to 1998. The method used on twelve decks consisted of vertical insertion of vibrators in a grid pattern. A transverse drag-through process was used on two decks. Cores were analyzed from decks of both types and compared with cores from conventionally poured decks in order to measure differences in entrapped air voids and in-place density. Based on statistical analysis, the grid pattern method was found to be significantly better than both the drag-through and conventional methods for reducing entrapments per square inch and increasing in-place density. The drag-through method was found to be better than the conventional method for reducing entrapments, but no difference was found for density. Segregation was found in the drag-through cores, but not for either the grid pattern or the conventional method. Some problems were noted on construction sites, mainly having to do with increased weight of the finishing machine and the effect of greater vibration intensity on superplasticized concrete. Costs associated with use of the grid pattern method were approximately $6.58 per square meter ($5.50 per square yard) of bridge deck. Key words: concrete consolidation, grid vibration, entrapped air.

INTRODUCTION

This paper discusses mechanized methods used to provide uniform vibratory consolidation of bridge deck concrete on several bridges in Illinois. The main topics are constructability, discussion of entrapped air and unit weight data, and the effect of the methods on improving those properties. Grid vibration refers to the process of vertical insertion of several vibrators in a matrix pattern. Drag-through vibration refers to a process by which vibrators attached to the finishing carriage were dragged transversely through the deck concrete just above the top reinforcement layer. Conventional vibration refers to the standard practice of consolidating bridge deck concrete.

Contractors in Illinois built all or part of twelve bridge decks using mechanized grid pattern consolidation equipment. Two other decks were built using a drag-through vibration process. The decks included in the study were a wide assortment of interstate and primary, 2 and 4-lane, stage and full width construction, skewed and normal, and superelevated and normally crowned structures. Table 1 shows these bridges.

<table>
<thead>
<tr>
<th>TABLE 1 List of Bridge Decks Paved Using Mechanical Methods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grid Pattern Vertical Insertion</td>
</tr>
<tr>
<td>I-57 Southbound over EJ&amp;E Railroad near Matteson (District 1)</td>
</tr>
<tr>
<td>I-80 Eastbound over NS Railroad near Mokena (District 1)</td>
</tr>
<tr>
<td>I-80 Westbound over US 30 near Joliet (District 1)</td>
</tr>
<tr>
<td>IL 1 over Little Calumet River (District 1)</td>
</tr>
<tr>
<td>IL 1 over Cal Sag &amp; Ashland Avenue (District 1)</td>
</tr>
<tr>
<td>OR 66 over Rooks Creek (District 3)</td>
</tr>
<tr>
<td>OR 66 over Turtle Creek (District 3)</td>
</tr>
<tr>
<td>US 67 over Edwards Creek (District 4)</td>
</tr>
<tr>
<td>I-70 Eastbound over Camp Creek (District 7)</td>
</tr>
<tr>
<td>IL 4 over Little Silver Creek (District 8)</td>
</tr>
<tr>
<td>IL 4 over Tributary to Little Silver Creek (District 8)</td>
</tr>
<tr>
<td>OR 5 over Chain-of-Rocks Canal (District 8)</td>
</tr>
<tr>
<td>Drag-Through Vibration</td>
</tr>
<tr>
<td>Brighton TWP 467 over Girder Brook (District 6)</td>
</tr>
<tr>
<td>Monroe Street (Southbound) over Spring Creek in Decatur(District 5)</td>
</tr>
</tbody>
</table>

CONSOLIDATION EFFECTS ON MATERIAL PROPERTIES

The objective of improved concrete consolidation is to remove entrapped air in plastic concrete with the goal of producing a deck with better, more uniform properties. An entrapped air void is defined by the Portland Cement Association (PCA) as a void larger than 1mm (1/25 inch) in diameter. Better consolidation increases the unit weight (density) of in-place concrete. Previous research (1) shows that improved density is highly correlated to improvements in compressive strength, impermeability, and bond strength to reinforcing steel. Improvements in these properties are all desirable. Reduction of entrapped air can affect deck smoothness and ride quality by reducing plastic concrete subsidence.

It was assumed that real improvements in compressive strength, bond strength, and permeability to chloride ions can be inferred when statistically significant differences in either increased density or removal of entrapped air voids are found. It was also of interest to note the variability of the data between conventional, grid pattern, and drag-through vibration methods.

There is general agreement in Illinois that achieving design concrete compressive strength is not a problem. However, lack of adhesive bond strength between concrete and epoxy-coated reinforcement emphasizes the importance of improving mechanical bond strength. Limiting permeability to chloride ions makes sense as a prudent general improvement to concrete mixtures in civil engineering structures.
FIGURE 1  Two by eight grid pattern device with finishing carriage on same machine. This device was typically used for most grid vibration projects.

FIGURE 2  Drag-through vibration device used in districts 5 & 6
CONSTRUCTION EXPERIENCE

Four grid vibration configurations were used. A prototype machine used a 2x4 (longitudinal by transverse) vibrator grid. Later machines used 4x4, 2x8, and 1x12 grids. A typical 2x8 configuration is shown in Figure 1. The 2x4 and 4x4 devices were mounted on paving bridges separate from the bridge deck finishing machine while the 2x8 and 1x12 configurations were attached to the finishing machines. The 2x4, 4x4, and 2x8 configurations were developed and manufactured by Allen Engineering of Paragould, Arkansas. The 1x12 device was built by a contractor. The drag-through vibration configuration is shown in Figure 2 and was also built by a contractor. The vibrators were spaced in each configuration to allow their radii of influence to overlap by no more than 25 mm (1 inch).

The devices and procedures were new, so contractors were sometimes reluctant to use them. Sub-contractual arrangements often resulted in scheduling problems and misunderstandings. By contrast, two contractors took the initiative and either developed or acquired their own mechanized vibration devices.

For a staged construction process, the weight of the device could potentially require that either the finishing machine rails be mounted on the fascia and the last interior beams, or that the rail support system be redesigned to accommodate the increased weight. Sometimes a significant amount of hand work was still required outside the area of grid vibration because the narrower the bridge, the less width that was actually being grid vibrated. This led to debate on the economics of the device on stage projects.

Use on four-lane divided stage construction was complicated by lease agreements. On most projects, the contractor would almost never concurrently pour both sides of a four-lane divided facility. This lag time was naturally charged by the lease agreement as ‘time on the job’ and could be one to two weeks. Second stage paving could typically be months later. This delay necessitated returning the device to the manufacturer and then paying for shipping, handling, and setup for subsequent pours.

Problems were encountered with supereleved bridge decks. First, the vibration effort caused the plastic concrete to flow downhill. This could result in excessive waste at the low side. Second, if the resultant overturning moment of the combined finishing and vibration machinery was not accounted for, the capacity of the adhesives used on supports for the finishing rails might not be adequate.

Most of the mix designs used in Illinois consist of one coarse aggregate and one fine aggregate. The resultant gap-graded mix design is sometimes difficult to pump. To improve pumpability, producers often add superplasticizer at the jobsite. Grid vibration of highly superplasticized concrete occurred on part of one deck, and the process was halted due to the visual assertion that the concrete was being segregated. That something already so flowable should need to be vibrated to remove entrapped air was difficult to rationalize, and the operation continued without the grid vibration process.

The experimental special provision was written as an addition to a section of the Standard Specifications for Road and Bridge Construction and was included in the bidding documents. A separate pay item was not developed, however, and the cost was included in the bid price for Concrete Superstructure. Sometimes contractors do not closely read the inserted special provisions and miss an addition that could be a significant cost item.

The grid vibration devices were subject to breakdown, as is any other mechanical device on a construction site. There were apparently few mechanical difficulties after the prototype 2x4 grid device. Placement times were apparently as good as or better than conventional work (2).

There were no apparent mechanical difficulties experienced with the drag-through device. However, vibrator influence radius and rate of paver advance combined to impart multiple vibration passes to the same transverse strip of material. Placement times were unaffected by the drag-through method because the vibrators were attached to the finishing carriage.

DATA ANALYSIS

Consultant Study of Grid Process

Nine cores were collected from both I-57 bridges and both I-80 WB bridges. These cores were analyzed by a consultant for entrapped and entrained air contents and density as part of a contract through FHWA (2). Important findings of the study were that the automated system 1) produced a more uniform deck, 2) reduced the amount of entrapped air by half, 3) did not reduce the amount of entrained air, 4) did not segregate the concrete, and 5) did not effect placement rates.

Consultant Study of Drag-Through Process

Six cores were cut from the Brighton TWP bridge, and eight cores were cut from the Decatur bridge. These cores were analyzed by the same consultant as for the grid cores. The cores were evaluated for entrapments per square inch, density, and evidence of segregation (4,5). The consultant found evidence of segregation in both cases. This finding is significant because no segregation was found in either the conventionally vibrated or grid pattern cores.

Entrapped Air and Unit Weight Data

Summary statistics for entrapments per square inch and density data from the core studies were independently developed and are presented in Table 2. These statistics were used to develop statements of significance for the relative effectiveness of the three methods. One-tailed tests were used. The t-statistic was used for the entrapments per square inch. The t-statistic was used for the density comparisons due to smaller sample sizes (6). Comparison of concrete mix designs used in different contracts was considered reasonable based on similarities of aggregates, paste percentages, and aggregate percentages.
TABLE 2  Grid, Drag-Through, and Conventional Vibration Summary Data

<table>
<thead>
<tr>
<th></th>
<th>Entrapments/in²*</th>
<th>Density (lb./ft³)**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grid Pattern</td>
<td>Average = 2.94</td>
<td>Average = 146.88</td>
</tr>
<tr>
<td></td>
<td>Standard Deviation = 1.47</td>
<td>Standard Deviation = 1.74</td>
</tr>
<tr>
<td></td>
<td>n = 208</td>
<td>n = 19</td>
</tr>
<tr>
<td>Drag-Through</td>
<td>Average = 3.98</td>
<td>Average = 143.56</td>
</tr>
<tr>
<td></td>
<td>Standard Deviation = 1.75</td>
<td>Standard Deviation = 4.12</td>
</tr>
<tr>
<td></td>
<td>n = 162</td>
<td>n = 15</td>
</tr>
<tr>
<td>Conventional</td>
<td>Average = 5.65</td>
<td>Average = 143.67</td>
</tr>
<tr>
<td></td>
<td>Standard Deviation = 2.23</td>
<td>Standard Deviation = 3.62</td>
</tr>
<tr>
<td></td>
<td>n = 208</td>
<td>n = 25</td>
</tr>
</tbody>
</table>

*To convert to entrapments/mm², multiply by 0.00155. ** To convert to kg/m³, multiply by 0.00160.

Comparison of Methods

Relative comparisons for each of the three methods in terms of reducing entrapped air voids and increasing density are presented in Table 3. Grid vibration is better at reducing air entrapments and increasing density than either drag-through or conventional vibration. Drag-through is better at reducing entrapments than conventional, but no difference was found in terms of density.

TABLE 3 Relative Comparison of Vibration Methods*

<table>
<thead>
<tr>
<th></th>
<th>Entrapped Air Reduction</th>
<th>Uniformity Increase</th>
<th>Density Increase</th>
<th>Uniformity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grid Pattern</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Drag-Through</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Conventional</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

*Rankings made at 95 percent confidence level.

COST ANALYSIS

The cost for grid pattern vibration on two projects is outlined in Table 4. The I-80 bridge over US 30 cost is based on a lease estimate. The figures for I-70 over Camp Creek are actual costs. Costs included travel to and from the jobsite, setup, dry run, two technicians, and estimated days of paving.

TABLE 4 Cost of Grid Pattern Vibration

<table>
<thead>
<tr>
<th></th>
<th>Quantity</th>
<th>Deck Thickness, in**</th>
<th>Cost ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Paved, cu yd*</td>
<td>Total</td>
<td>Per cu yd ** ***</td>
</tr>
<tr>
<td>I-80 WB over US 30</td>
<td>700</td>
<td>8</td>
<td>17,526 25.04 5.56</td>
</tr>
<tr>
<td>I-70 EB over Camp Creek</td>
<td>603</td>
<td>7.5</td>
<td>15,911 26.38 5.50</td>
</tr>
</tbody>
</table>

*To convert to m³, multiply by 0.76455. ** To convert to mm, multiply by 25.4. *** To convert to $/m³, multiply by 1.308. **** To convert to $/m², multiply by 10.764.

The average quantity of Concrete Superstructure (in-place deck and parapet) placed in Illinois from 1996-1998 was 481.9 cubic meters (368.4 cubic yards). The statewide average cost for Concrete Superstructure for 1996-1998 was $873.50 per cubic meter ($667.79 per cubic yard). Using the above prices, the estimated cost of the grid vibration process for an average structure is between 3.75 and 3.95 percent of the average cost of Concrete Superstructure. For an average project, the cost increase would be $9,471.53. Given that the average bridge superstructure cost is just over $246,000, it appears that grid vibration adds very little to the cost of the finished deck. Since Concrete Superstructure is only part of the total cost of a project, the impact on a typical bridge rehabilitation project is low.

CONCLUSIONS

The following conclusions are made based on the observations and data analysis:
1. The grid pattern method was a statistically significant improvement over both the conventional and drag-through methods in terms of both reduced air entrapments and increased density.
2. The drag-through method was a statistically significant improvement over the conventional method in terms of entrapments. No statistical difference was found between drag-through and conventional methods in terms of density.
3. Based on analysis of cores, the drag-through method is more likely to cause segregation than either the grid pattern or conventional methods.
4. The grid pattern method is applicable to a wide range of construction scenarios, given proper attention to issues related to superplasticized concrete and paver weight.
5. The cost of the grid pattern method was a minor addition to the total cost of the bid item for Concrete Superstructure.

RECOMMENDATIONS

The following recommendations and guidelines for use are made:
1. The grid pattern method should be the preferred Illinois DOT method for consolidation of bridge deck concrete in the absence of practical limits. Practical considerations for limiting
use include deck super-elevation, variable deck width, and effects of finishing machine weight on beam deflection during paving. Grid pattern vibration and high dosages of superplasticizer should be discussed and accounted for in the pre-pour meeting.

2. IDOT has shown a willingness to pay for improved quality and should be willing to pay something for mechanized grid pattern vibration. The unit price for mechanized vibration of bridge decks should be $3.48-4.63 per square meter ($3-4 per square yard). This is a minimal price addition to an average contract and shares the lease, purchase, or development cost of a device between the department and the contractor.

NOTICE

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policy of the Federal Highway Administration or the Illinois Department of Transportation. This report does not constitute a standard, specification, or regulation.

Neither the United States Government nor the State of Illinois endorses products or manufacturers. Trade names or manufacturers’ names appear herein solely because they are considered essential to the object of this report.

ACKNOWLEDGMENTS

The author gratefully acknowledges the kind assistance and support of many Illinois Department of Transportation District Construction and Materials personnel, Professor Tom Parsons of Arkansas State University, The Allen Engineering Corporation, the Bid-Well Division of CMI Corporation, and the late Mr. Harry Horn.

REFERENCES

Statewide Traffic Engineering and Safety Informational Series Survey: The Results

KEITH K. KNAPP, THOMAS M. WELCH, AND JAMES W. STONER

Effective communication with decision-makers and the general public is a vital task of state, county, and local transportation professionals. Questions must be answered in a clear and consistent manner and based on accepted engineering reference material. During the past year, the Iowa Department of Transportation, the Center for Transportation Research and Education (CTRE) at Iowa State University, and the University of Iowa developed and distributed a traffic and safety informational series survey. The objective of the survey was to determine the 20 most commonly asked traffic and safety questions in Iowa. The survey was sent to approximately 300 transportation professionals and was also available on the worldwide web. It requested that respondents estimate, on a scale of 1 to 5, how often the public asked them 12 specific questions. The survey also asked the respondents to add and rank questions they thought should be considered in the series. About 100 responses were received, and 77 questions were suggested by the respondents. Two-page answer sheets are being prepared and reviewed for 10 of the 12 questions included within the survey and 10 questions that summarize those suggested by the respondents. This paper discusses the objectives and tasks included in the informational series project. The survey and its results are summarized, and the format of the proposed answers are described and presented. The status of the project and its schedule are also provided. The informational series project should provide a useful and understandable public information and education tool for transportation professionals. Key words: safety, traffic, public information.

INTRODUCTION

Transportation professionals are often asked to explain relatively complicated traffic safety and engineering issues. In fact, different people often ask the same type of questions. The goal is to answer these questions in a clear, consistent, and correct manner. The informational series project described in this paper was designed to address the most commonly asked traffic safety and engineering questions/issues raised by the general public and governmental officials.

PROJECT OBJECTIVES

Communication with the general public and decision-makers is a vital task for transportation professionals. Answers to questions must be consistent and based on accepted engineering reference and research material. The objective of the project described in this paper was to address commonly asked traffic and safety questions in a series of two-page answer sheets. Upon completion, these answer sheets will be distributed to transportation professionals throughout Iowa.

PROPOSED PROJECT TASKS AND SCHEDULE

The informational series project included five tasks. The first task involved the construction and distribution of a survey. This survey was sent to transportation professionals in Iowa and was used to identify the most commonly asked traffic and safety questions. The second task of the project included an analysis of the survey results and the selection of 20 questions to answer in this informational series. This task is the focus of this paper and is discussed in the following paragraphs. The third and fourth tasks of the project included the completion and editing of two-page answer sheets for each question and a review of the answers by an advisory committee. This review is currently being completed. Finally, the fifth task included the update and distribution of the entire informational series to transportation professionals throughout Iowa.

The project began in the spring of 1999 and tasks one to three are primarily complete as of December 1999. The answer sheets are currently being reviewed by an advisory committee and may also be reviewed by some of the survey respondents. Distribution of the informational series is currently scheduled for spring 2000.

THE SURVEY

To be useful, an informational series must address those questions and issues addressed most often by transportation professionals. The first task of this project was the construction and distribution of a survey to identify these questions/issues within Iowa. The survey constructed and distributed is shown in Figure 1. It included a short paragraph of introduction that explained the objectives of the survey and project and asked the respondents to weigh how often they were asked 12 specific questions. The scale provided went from one (frequently asked) to five (never asked). The respondents were also asked to suggest and weigh their own questions. The results of the survey are discussed in the following paragraphs.

SURVEY RESULTS

Approximately 300 surveys were mailed, along with a postage-paid and pre-addressed return envelope, to transportation professionals throughout Iowa. The survey was also available on the Center for
Center for Transportation Research and Education  
Traffic Safety and Engineering Informational Series Project  
Survey of Iowa Transportation Professionals

We need your help! The Center for Transportation Research and Education (CTRE) at Iowa State University is distributing this survey to transportation professionals in Iowa. The results of the survey will be used to identify common questions/topics that should be addressed by one to two page informational answer packets. These packets will be distributed to transportation professionals throughout Iowa (as a handout for the public).

Please rate the following questions and topics, from 1 (something that the public frequently asks) to 5 (something that is rarely or never asked). Also, feel free to add and rate your own questions or topics. The 20 questions/topics with the most weight will be addressed.

We would like to develop the answer packets at the end of the summer. Please mail, fax, or phone the survey results by August 15, 1999 (submittal information at the end of the survey). This survey will also be available at our website: www.ctre.iastate.edu. Thanks.

Name: ________________________________  
Position: ________________________________  
City or Co.: ________________________________  
Address: ________________________________  
Phone #: ________________________________  
Fax #: ________________________________

<table>
<thead>
<tr>
<th>QUESTION OR TOPIC</th>
<th>LEVEL OF IMPORTANCE</th>
<th>(1=frequently asked)/(5=never asked)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Why can’t we have a 4-way stop to reduce accidents?</td>
<td></td>
<td>☐ ☐ ☐ ☐ ☐</td>
</tr>
<tr>
<td>2. Why can’t we have stop signs to reduce speeding along my street?</td>
<td></td>
<td>☐ ☐ ☐ ☐ ☐</td>
</tr>
<tr>
<td>3. Won’t a traffic signal reduce accidents?</td>
<td></td>
<td>☐ ☐ ☐ ☐ ☐</td>
</tr>
<tr>
<td>4. Won’t a lower speed limit lower travel speeds and the number of accidents?</td>
<td></td>
<td>☐ ☐ ☐ ☐ ☐</td>
</tr>
<tr>
<td>5. Why aren’t there better, longer lasting stripes on the road?</td>
<td></td>
<td>☐ ☐ ☐ ☐ ☐</td>
</tr>
<tr>
<td>6. Won’t a “CHILDREN AT PLAY” sign help protect our children?</td>
<td></td>
<td>☐ ☐ ☐ ☐ ☐</td>
</tr>
<tr>
<td>7. Why can’t I have several driveways to my property wherever I want them?</td>
<td></td>
<td>☐ ☐ ☐ ☐ ☐</td>
</tr>
</tbody>
</table>

FIGURE 1 Sample informational series question survey.
<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>8.</td>
<td>Safe driving procedures at railroad crossings.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>9.</td>
<td>What is the harm in installing an unwarranted traffic control device?</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>10.</td>
<td>Why do light poles have to be located so far from the street?</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>11.</td>
<td>How many bullet holes does it take to kill a sign-(sign vandalism)?</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>12.</td>
<td>Less is better-Why converting a four lane street to a three lane street may improve safety and not increase congestion.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>ADD YOUR OWN QUESTIONS:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13.</td>
<td></td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>14.</td>
<td></td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
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<tr>
<td>15.</td>
<td></td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>16.</td>
<td></td>
<td>☐</td>
<td>☐</td>
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<tr>
<td>17.</td>
<td></td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
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<tr>
<td>18.</td>
<td></td>
<td>☐</td>
<td>☐</td>
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<tr>
<td>19.</td>
<td></td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>20.</td>
<td></td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
</tbody>
</table>

Please return in self-addressed stamped envelope: 2901 S. Loop Drive, Suite 3100 Ames, Iowa 50010-8632 Attention: Dr. Keith Knapp Phone (515)-294-7082, Fax 515-294-0467

FIGURE 1 Sample informational series question survey (continued).
Transportation Research and Education webpage. Overall, about 100 surveys were returned (a 33 percent response rate). The 12 questions suggested in the survey were weighed by the respondents and subsequently ranked by their average weight by CTRE. These questions, along with their average weight (i.e., ranking), are shown in Table 1. Ten of the questions, as indicated in Table 1, are being considered in this project.

**TABLE 1 Traffic Safety and Engineering Informational Series Survey Results: Suggested Questions/Issues**

<table>
<thead>
<tr>
<th>Suggested Questions/Issues</th>
<th>Average Weight1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Won’t a “CHILDREN AT PLAY” sign help protect our children?</td>
<td>4.12</td>
</tr>
<tr>
<td>Why can’t we have stop signs to reduce speeding along my street?</td>
<td>4.03</td>
</tr>
<tr>
<td>Won’t a lower speed limit lower travel speeds and the number of accidents?</td>
<td>3.79</td>
</tr>
<tr>
<td>Why can’t we have a 4-way stop to reduce accidents?</td>
<td>3.47</td>
</tr>
<tr>
<td>Why can’t I have several driveways to my property wherever I want them?</td>
<td>3.42</td>
</tr>
<tr>
<td>What is the harm in installing an unwarranted traffic control device?</td>
<td>3.12</td>
</tr>
<tr>
<td>Why aren’t there better, longer lasting stripes on the road?</td>
<td>2.97</td>
</tr>
<tr>
<td>Won’t a traffic signal reduce accidents?</td>
<td>2.94</td>
</tr>
<tr>
<td>Safe driving procedures at railroad crossings.</td>
<td>2.13</td>
</tr>
<tr>
<td>How many bullet holes does it take to kill a sign?</td>
<td>2.13</td>
</tr>
<tr>
<td>Less is better—Why converting a four lane street to a three-lane street may improve safety and not increase congestion.</td>
<td>2.04</td>
</tr>
<tr>
<td>Why do light poles have to be located so far from the street?</td>
<td>1.99</td>
</tr>
</tbody>
</table>

1Weighting factors reversed from survey weighting scheme (See Figure 1) so the highest number represents the most frequently asked question. Average weight in this table is based on range of one (never asked) to five (frequently asked).

Seventy-seven individual questions were also provided and weighed by the survey respondents. Many of the questions were similar, however, and 12 question groups were formed. A summary question for 11 of these question groups was developed (the twelfth group consisted of six miscellaneous questions). The summary questions, along with their average weight (i.e., ranking) and the number of respondent questions they represent, are shown in Table 2. Ten of these questions are being considered in this project, as indicated in Table 2. Overall, the summary questions addressed represent 69 of the 77 questions suggested by the survey respondents.

**TABLE 2 Traffic Safety and Engineering Informational Series Survey Results: Summary Questions/Issues**

<table>
<thead>
<tr>
<th>Summary Questions/Issues</th>
<th>Average Weight1</th>
<th>Number of Respondent Questions Represented</th>
</tr>
</thead>
<tbody>
<tr>
<td>How does the county make decisions about paving gravel roadways?</td>
<td>5.00</td>
<td>9</td>
</tr>
<tr>
<td>How do you decide where to place signs?</td>
<td>4.50</td>
<td>2</td>
</tr>
<tr>
<td>Why can’t I place a business-related directional sign within the roadway right-of-way?</td>
<td>4.50</td>
<td>4</td>
</tr>
<tr>
<td>What factors are considered when locating, controlling, and/or marking pedestrian/bicycle crossing?</td>
<td>4.38</td>
<td>8</td>
</tr>
<tr>
<td>Why can’t all the signals be timed so I receive a green light at every intersection?</td>
<td>4.36</td>
<td>11</td>
</tr>
<tr>
<td>How are signals timed to accommodate pedestrians?</td>
<td>4.25</td>
<td>4</td>
</tr>
<tr>
<td>Why isn’t there a “School Bus Stop Ahead” everywhere a bus stops?</td>
<td>4.25</td>
<td>4</td>
</tr>
<tr>
<td>How does the county make decisions about dust control on gravel roadways?</td>
<td>4.20</td>
<td>5</td>
</tr>
<tr>
<td>Why can’t speed bumps be used on all streets to slow traffic?</td>
<td>4.17</td>
<td>12</td>
</tr>
<tr>
<td>How do you choose the posted speed limit and where you put signs?</td>
<td>4.00</td>
<td>5</td>
</tr>
<tr>
<td>When do intersections receive stop signs (2-way or 4-way) and signals?</td>
<td>4.00</td>
<td>7</td>
</tr>
</tbody>
</table>

1Weighting factors reversed from survey weighting scheme (See Figure 1) so the highest number represents the most frequently asked question. Average weight in this table is based on range of one (never asked) to five (frequently asked).

**CONCLUSION AND FUTURE WORK**

The survey constructed and distributed as part of this statewide informational series project provided invaluable direction for the selection of the questions it should address. It is believed the questions/issues chosen for consideration in this project will assist and possibly improve communication between transporta-

**ACKNOWLEDGMENT**

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