Thursday, August 21

	A	B	C	D	E	F
Concurrent Session I 10:15 to 11:45	Asset Management I	Bridge I	Planning	Traffic and Access Management	Asphalt	Transit I
Concurrent Session II 1:15 to 2:45	Asset Management II	Bridge II	Big Picture Planning	Traffic and Safety I	PCC Pavements I	Transit II
Concurrent Session III 3:15 to 4:45	Pavement Management	Bridge III	Planning Methods	Traffic and Safety II	PCC Pavements II	Transit III

Friday, August 22

Concurrent Session IV 8:00 to 9:30	F-SHRP	Winter Maintenance I	Design and Construction I	Traffic and Safety III	Summer Maintenance	
Concurrent Session V 9:45 to 11:15	Education	Winter Maintenance II	Design and Construction II		Security	

A

B

Thursday, August 21

Asset Management I

Local Government Adoption of Asset Management

Ernie Wittwer, Jason Bittner, Char Kasprzak, Midwest Regional University Transportation Center at the University of Wisconsin-Madison

GASB Statement 34—the On-Ramp to Transportation Asset Management or a Detour Leading to Business as Usual?

Tom Maze, Omar Smadi, CTRE at Iowa State University

Transportation Asset Management Today: An Evaluation of an Emerging Virtual Community of Practice

Jeromie Winsor, Laxmi Ramasubramanian, Sue McNeil, University of Illinois at Chicago; Lou Adams, New York State DOT

Asset Management II

Utilization of LIDAR Technology for Highway Inventory

Huda Shamayleh, Aemal Khattak, University of Nebraska-Lincoln

Extraction of Transportation Infrastructure from Hyperspectral Remote Sensing Data

James Gerjevic, University of Northern Iowa

Application of GIS to Emergency Management

Cynthia Wilson Orndoff, Raman Deep Mata, University of Missouri-Columbia

Pavement Management

Demonstrating the Use of Pavement Management Tools to Address GASB 34 Requirements

Angela Wolters, Kathryn Zimmerman, Applied Pavement Technology, Inc.

Integrating Preventive Maintenance and Pavement Management Practices

Kathryn Zimmerman, David Peshkin, Applied Pavement Technology, Inc.

Adaptability of AASHTO Provisional Standards for Condition Surveys for Roughness and Faulting in Kansas

Kamesh Vedula, Mustaque Hossain, James Peterson, Kansas State University; Rick Miller, Kansas DOT; Gaylord Cumberledge, CGH-Pavement Engineering, Inc.

Roughness Progression Model on Kansas PCC Pavements

Victoria Felker, Mustaque Hossain, Yacoub Najjar, Kansas State University; Richard Barezinsky, Kansas DOT

Friday, August 22

F-SHRP

Accelerating the Renewal of America's Highways

Thomas Cackler and Stephen Andrle, CTRE at Iowa State University

F-SHRP Safety: Making a Significant Improvement in Highway Safety

Presenter to be announced

F-SHRP Reliability: Providing a Highway System with Reliable Travel Times

Presenter to be announced

F-SHRP Capacity: Providing Highway Capacity in Support of the Nation's Economic, Environmental, and Social Goals

Presenter to be announced

Education

Training—the Key to Technology Implementation

Leland Smithson, AASHTO Snow and Ice Cooperative Program

Iowa DOT Technical Training Academy

Christie Anderson, Iowa DOT

Turning Students on to Transportation: A Pilot Program for Recruiting High School Students into Transportation Careers and Programs of Study

Marcia Brink, Michele Regenold, CTRE at Iowa State University

Thursday, August 21

Bridges I

FHWA Bridge Research and Technology Deployment Initiatives

Ian Friedland, FHWA

An Overview of Scour Types and Scour-Estimation Difficulties at Bridge Abutments

Robert Ettema, Tatsuaki Nakato, Marian Muste, University of Iowa

Monitoring and Evaluation of the Iowa River Bridge Launch

B.M. Phares, R.E. Abendroth, T.J. Wipf, N. McDonald, S. Abraham, Iowa State University

Bridges II

Construction of Laboratory and Field Demonstration Modified Beam-in-Slab Bridges

T.F. Konda, F.W. Klaiber, T.J. Wipf, Iowa State University

Intermediate Diaphragms for Laterally Impacted PC Girder Bridges

Robert Abendroth, Fouad Fanous, Iowa State University

Strengthening of Steel Girder Bridges Using FRP

B.M. Phares, T.J. Wipf, F.W. Klaiber, A. Abu-Hawash, Y.S. Lee, Iowa State University

Repair of Damaged Prestressed Concrete Bridges Using CFRP

F.W. Klaiber, T.J. Wipf, B.J. Kempers, A. Abu-Hawash, Iowa State University

Bridges III

Development of Bridge Load Rating Techniques Using Physical Testing

B.M. Phares, T.J. Wipf, F.W. Klaiber, A. Abu-Hawash, Iowa State University

Monitoring and Evaluation of Plate Girder Bridges with Fatigue Problems

Douglas Wood, Lowell Greimann, Terry Wipf, Brent Phares, Iowa State University; Bruce Brakke, David Tarries, Iowa DOT

Experimental Evaluation of Deteriorated Precast Channel Bridges
T.J. Wipf, F.W. Klaiber, J.S. Ingersoll, Iowa State University

Railroad Flatcar Bridges for Economical Bridge Replacement Systems
Justin Doornink, Terry Wipf, Wayne Klaiber, Iowa State University

Friday, August 22

Winter Maintenance I

Using an Artificial Neural Network to Predict Parameters for Frost Deposition on Iowa Bridgeways
Bradley Temeyer, William Gallus, Jr., Iowa State University; Karl Jungbluth, National Weather Service; Dennis Burkheimer, Diane McCauley, Iowa DOT

Bridge Frost: Observations and Forecasts by Numerical Methods
Tina Greenfield, Eugene Takle, Brian Tentinger, Jose Alamo, Iowa State University; Dennis Burkheimer, Diane McCauley, Iowa DOT

Bridge Prioritization for Installation of Automatic Anti-icing Systems in Nebraska
Aemal Khattak, Geza Pesti, University of Nebraska-Lincoln

Winter Maintenance II

Cost-Effectiveness of Abrasives, Salt Brine, and LCS for Winter Operations in Nebraska
Yu Liu, Geza Pesti, Mid-America Transportation Center at the University of Nebraska-Lincoln

Deploying the Winter Maintenance Decision Support System in Iowa
Dennis Kroeger, CTRE at Iowa State University; Dennis Burkheimer, Iowa DOT

A Business Case for Winter Maintenance Technology Applications: Highway Maintenance Concept Vehicle
Dennis Kroeger, Reggie Sinhaa, CTRE at Iowa State University

Thursday, August 21

Planning

Planning the Mississippi River Trail in Iowa Using Geographic Information Systems
Jamie Luedtke, David Plazak, CTRE at Iowa State University

Intracity Bicycle Connectivity: The Sustainable Transportation Alternative for Midwestern Cities (A Case Study of the Waterloo–Cedar Falls Metropolitan Area)
Nanmi Tissing, Andrey Petrov, Jess Elder, University of Northern Iowa

Iowa’s New Rest Area Buildings
Mark Masteller, Iowa DOT

Big Picture Planning

Exclusive Facilities for Trucks: An Investigation of the Potential for Reserved Truck Lanes and Truckways on the Florida State Highway System
Stephen Reich, Janet Davis, Anthony Ferraro, Martin Catalá, Center for Urban Transportation Research at the University of South Florida

US 83—FHWA Economic Development Highway Corridor Initiative in Texas: Border Crossings and Rural Communities
Richard Burruss, Mark Berndt, Wilbur Smith Associates

Applications of MITSIMLab in the Des Moines Metropolitan Area
Stu Anderson, Iowa DOT; Mithilesh Jha, Jacobs; Tom Kane, Des Moines Area Metropolitan Planning Organization

Planning Methods

Collaboration, Automation, and Cognitive Aids Technologies to Solve Complex Transportation Problems
Deepa Mani, University of Missouri-Columbia

A Dynamically Configurable Network-Scanning Tool for the Spatial Cluster Analysis of Linear Network Feature Corridors
Dan Gieseman, CTRE at Iowa State University

Friday, August 22

Design and Construction I

Iowa’s Statewide Urban Design Standards Promote Improved Access Management
David Plazak, Dale Harrington, CTRE at Iowa State University

An Object-Oriented CAD Specification System for Iowa DOT and Urban Standards
Russell Walters, Edward Jaselskis, Iowa State University

Development and Maintenance of the Electronic Reference Library
Russell Walters, Charles Jahren, Iowa State University

Construction Scheduling for Urban Freeway Renewal Projects: A Case Study
Dong Chen, Charles Jahren, Augusto Canales, Iowa State University

Design and Construction II

Pilot Study on Improving the Efficiency of Transportation Projects Using Laser Scanning
Edward Jaselskis, Zhili Gao, Iowa State University; Alice Welch, Dennis O’Brien, Iowa DOT

Use of LIDAR-Based Elevation Data for Highway Drainage Analysis: A Qualitative Assessment
Zachary Hans, Reginald Souleyrette, Ryan Tenges, CTRE at Iowa State University

Web-Based Database for Highway Erosion and Sedimentation Control Measures
M. Muste, R. Ettema, K. Yu, R. Gomez, University of Iowa

Thursday, August 21

Traffic and Access Management

Process to Identify High Priority Corridors for Access Management Near Large Urban Areas in Iowa Using Spatial Data

David Plazak, Reginald Souleyrette, CTRE at Iowa State University

Access Management Techniques to Improve Traffic Operations and Safety: A Case Study of a Full vs. Directional Median Opening

Sunanda Dissanayake, Kansas State University; John Lu, University of South Florida

Impact of Modern Roundabouts on Vehicular Emissions

Eugene Russell, Srinivas Mandavilli, Margaret Rys, Kansas State University

Traffic and Safety I (Rural)

Reducing Crashes at Rural Thru/Stop-Controlled Intersections

Howard Preston, CH2M Hill

Young Drivers and Run-Off-the-Road Crashes

Sunanda Dissanayake, Kansas State University

Centerline Rumble Strips on Two-Lane Roads: Pattern Research and US Usage

Eugene Russell, Margaret Rys, Troy Brin, Kansas State University

Deer-Vehicle Crash Countermeasures Effectiveness Research Review

Keith Knapp, Tanveer Oakasa, Xin Yi, University of Wisconsin-Madison

Traffic and Safety II

Improving Left-Turn Safety Using Flashing Yellow Arrow Permitted Indications

David Noyce, University of Wisconsin-Madison

Evaluating the Kansas City Scout Traffic Management System

Eric Meyer, University of Kansas; Carlos Sun, University of Missouri-Columbia

Urban Four-Lane Undivided to Three-Lane Roadway Conversion Guidelines

Keith Knapp, Woochul Lee, University of Wisconsin-Madison; Karen Giese, Kittelson and Associates, Inc.

Friday, August 22

Traffic and Safety III

Effectiveness of the MPH D-25 Speed Advisory Sign System in Reducing Traffic Speeds Upstream of Traffic Slowdowns

Vijay Kannan, Geza Pesti, Mid-America Transportation Center at the University of Nebraska-Lincoln

The Role of the Street Environment in How People Cross Roads in Urban Settings

Michael Baltes, Center for Urban Transportation Research at the University of South Florida

Roadway Conditions as Contributing Factors in Florida Traffic Crashes

Michael Baltes, Center for Urban Transportation Research at the University of South Florida

Fiber Optic Implementation of a Cumulative Momentum Model for Natural Urban Intersection Traffic Management

Chung Yu, Sheldon Muir, North Carolina A&T State University; Albert Harbury, Green Light Associates

Thursday, August 21

Asphalt

HMA Mix Design Validation Study

Dan Redmond, Iowa DOT

Effect of PG Binder Grade and Source on Performance of Superpave Mixtures under Hamburg Wheel Tester

Aneel Gogula, Mustaque Hossain, John Boyer, Stefan Romanoschi, Kansas State University

Correlation between the Laboratory and Field Permeability Values for Superpave Pavements

Aneel Gogula, Mustaque Hossain, Stefan Romanoschi, Kansas State University; Glenn Fager, Kansas DOT

Foamed Asphalt Stabilized Base in Reclaimed Asphalt Pavement: A Promising Technology for Midwestern Roads

Stefan Romanoschi, Mustaque Hossain, Kansas State University; Michael Heitzmann, Iowa DOT; Andrew Gisi, Kansas DOT

PCC Pavements I

Effect of Curling on As-Constructed and Early Life Smoothness of PCC Pavements

Zahidul Siddique, Mustaque Hossain, John Devore, Kansas State University; William Parcells, Jr., Kansas DOT

Evaluation of Premature PCCP Longitudinal Cracking in Colorado

Ahmad Ardani, Shamshad Hussain, Robert LaForce, Colorado DOT

Alternative Dowel Bars

Max Porter, Iowa State University

PCC Pavements II

Effects of Curing Materials and Methods on Properties of Concrete in Pavements

Kejin Wang, James Cable, Ge Zhi, Iowa State University

A Structural Model for the Rapid Analysis of Concrete Pavement Systems

Halil Ceylan, Iowa State University; Erol Tutumluer, Ernest Barenberg, University of Illinois at Urbana-Champaign

Implications of Wetting-Drying Cycles for the Performance of Pavement Systems
Radhey Sharma, Iowa State University

Friday, August 22

Summer Maintenance

Synthesis of Best Practice for Increasing Protection and Visibility of Highway Maintenance Vehicles
Ali Kamyab, Western Transportation Institute; Tom McDonald, CTRE at Iowa State University

Assessment of the Benefits of Automatic Vehicle Location for Highway Maintenance
Eric Meyer, University of Kansas

Quantitative Guidelines for Use of Thin Maintenance Surfaces
Charles Jahren, Jacob Thorius, Iowa State University

Cost Comparison of Treatments Used to Maintain or Upgrade Aggregate Roads
Charles Jahren, David White, Duane Smith, Jacob Thorius, Mary Rukashaza-Mukome, Iowa State University; Greg Johnson, Minnesota DOT

Security

Highway Infrastructure Security
Mac Lister, FHWA

Costs and Benefits of Technology to Enhance the Safety and Security of Hazardous Materials Transportation
Joseph Delorenzo, Federal Motor Carrier Safety Administration

Transportation Security: A State Perspective
Presenter to be announced

Thursday, August 21

Transit I

Transit Capacity and Quality of Service Manual: Transit Level of Service
Stephen Andrle, CTRE at Iowa State University

Evaluation of First-Year Florida MPO Transit Capacity and Quality of Service Reports
Victoria Perk, Chandra Foreman, Center for Urban Transportation Research at the University of South Florida

Statistical Estimation of the Importance Customers Place on Specific Elements of Bus Rapid Transit
Michael Baltes, Center for Urban Transportation Research at the University of South Florida

Conducting a Successful On-Board Survey of Public Transit Customers
Michael Baltes, Center for Urban Transportation Research at the University of South Florida

Transit II

Spatial Data Integration for Low-Income Worker Accessibility Assessment: A Case Study of the Chicago Metropolitan Area
Piyushimita Thakuriah, Juan Ortega, P.S. Sriraj, Urban Transportation Center at the University of Illinois at Chicago

Analysis of the Environmental Justice Compliance of the Chicago Transit Authority
Geoff Fruin, P.S. Sriraj, Urban Transportation Center at the University of Illinois at Chicago

Public Transportation Information Systems: Application to Labor Market and Transportation Problems
Sudeshna Sen, P.S. Sriraj, Piyushmita Thakuriah, Urban Transportation Center at the University of Illinois at Chicago

Challenges in Collecting the NTD Data and Available Automated Methods
A. Nilgün Kamp, formerly Center for Urban Transportation Research at the University of South Florida (Victoria Perk presenting)

Transit III

Development of an In-House Automated Vehicle Location
Juan Ortega, Audrey Wennink, Urban Transportation Center at the University of Illinois at Chicago

Travel Demand Modeling of Automated Small Vehicle Transit on a University Campus
Stanley Young, Rick Miller, Kansas DOT; Dean Landman, Kansas State University

Iowa Rural Transit ITS Implementation
Jeff Stratton, Iowa DOT

Accomplishments of an Innovative Statewide Van Lease and Purchase Program
Frederick Wegmann, Theodore Newsom, University of Tennessee

Lateral Impacts to PC Girders in Bridges with Intermediate Diaphragms

Robert Abendroth and Fouad Fanous
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ABSTRACT

Bridge engineers are concerned about the response of PC girder bridges which are hit by over-height-vehicle loads. The roll of intermediate diaphragms in providing impact-damage protection to the PC girders is not clearly defined. An analytical study was conducted to assess the role of intermediate diaphragms in reducing the damage to the girders of a PC girder bridge that is struck by an over-height object on a highway vehicle. Also, the study investigated whether a structural steel, intermediate diaphragm would essentially provide the same degree of impact protection to the PC girders as that provided by a reinforced concrete, intermediate diaphragm. Finite-element models were developed for non-skewed and skewed, PC girder bridges. Each model was analyzed with one reinforced concrete and two types of steel intermediate diaphragms that were located at mid-span of an interior span of the bridge. The bridge models were analyzed for a lateral-impact load that was applied to the bottom flange of the exterior girders at the diaphragm location and away from the diaphragm location. The induced strains and displacements in the girders were established for each diaphragm case. When a lateral-impact load was applied at the diaphragm location, the reinforced concrete, intermediate diaphragm provided more protection for the girders than that provided by the two types of structural steel, intermediate diaphragms. The three types of intermediate diaphragms provided essentially the same degree of impact protection for the PC girders when the load was applied away from the diaphragm location.

Key words: bridge diaphragms—finite element analysis—lateral impact

INTRODUCTION

Bridge engineers are concerned about the damage that impacts from over-height-vehicle loads cause to prestressed concrete (PC) girder bridges. Shanafedt and Horn (1) noted that about 120 PC girder bridges in the United States are damaged each year by over-height-vehicle loads. The actual number of impacts to bridges is probably significantly higher than these numbers, since many minor collisions are not reported to authorities.

Engineers with the Bridges and Structures Design Section of the Iowa Department of Transportation (Iowa DOT) have historically believed that the mass of a reinforced concrete (RC), intermediate diaphragm will provide a better degree of damage protection to the PC girders than that provided by a steel intermediate diaphragm. However, to reduce the construction time and to simplify the construction process, bridge contractors in the State of Iowa have always expressed a desire to install steel, intermediate diaphragms rather than to construct RC, intermediate diaphragms for PC girder bridges.

This paper discusses whether a steel, intermediate diaphragm with relatively simple connections to the PC girders can provide the same degree-of-damage protection to the PC girders as that provided by the RC, intermediate diaphragm currently being used by the Iowa DOT for both non-skewed and skewed bridges. Detailed finite-element models of several bridges were analyzed with the ANSYS software (2). Even though this investigation did not involve experimental work, the finite-element models were calibrated using the experimental test results that were obtained from published literature.

FINITE ELEMENT CALIBRATIONS

Description of an Experimental Bridge Model

The single-span bridge that was constructed and tested by Abendroth et al. (3) at Iowa State University was used to guide the development of the finite-element models for prototype bridges. The experimental bridge had three, Iowa DOT Type-A38, PC girders that were spaced at 6 ft – 0 in. (1830 mm) on center. The girders supported a 4-in. (100-mm) thick RC deck that was 40 ft – 4 in. (12300-mm) long and 18-ft (5490-mm) wide. At each end of the bridge model, a 42-in. (1070-mm) deep by 18-in. (460-mm) wide, RC abutment supported the PC girders. The ends of the girders were embedded 8 in. (200 mm) into a full-depth, 14-in. (360-mm) thick, RC, end diaphragm. Different types of intermediate diaphragms and locations within the span were considered in those tests.

Finite-Element Model of the Experimental Bridge

The deck and the PC girders of the experimental bridge were modeled using solid elements. Shell elements were used to model the end-diaphragms and the abutments. The different types of intermediate diaphragms were modeled using different types of finite elements. For each diaphragm type, interface elements, which can model sliding and separation between the diaphragm elements and the adjacent elements for the PC girder and RC deck, were used in the analytical models. These models were analyzed for transverse, horizontal loads that were applied to the bottom flange and at the mid-span of either of the exterior girders. Andrawes (4) presented the complete details of the finite-element-modeling techniques for the experimental bridge.

About a 20% difference occurred between the experimentally-measured and analytically-predicted, horizontal displacements for the experimental bridge. The calculated, longitudinal strains in the PC girder were compared to the measured strains. The predicted, longitudinal strains in the PC girders and in the intermediate diaphragms were within about 20% of the measured longitudinal strains. This difference in the predicted and measured strains was attributed to the presences of concrete cracks that existed in the experimental bridge. These cracks were not modeled in the finite-element models. The relative closeness of the analytical predictions to the measured bridge responses revealed that the analytical-modeling techniques were applicable to analyze PC girder bridges that are subjected to lateral forces.

BRIDGES SELECTED FOR THE ANALYSIS

Non-skewed Bridge

The prototype, non-skewed bridge that was selected for this study has four spans, three frame-type piers, and two integral abutments. The length of each end span is 35 ft – 9 in. (10900 mm), while the length for each inner span is 96 ft – 6 in. (29400 mm). An 8-in. (200-mm) thick, bridge deck is supported by five, equally-spaced, Iowa Type-D, PC girders. A 3-ft (910-mm) thick, RC end diaphragm (abutment backwall) was cast at each abutment. At the ends of the PC girders and at the location of the RC intermediate diaphragm, two, ¾-in. (20-mm), diameter, coil rods passed through the bottom flange of each girder and extended into the end diaphragms and intermediate diaphragms. Bent-reinforcing bars connected the bridge deck to the end diaphragm.

Skewed Bridge

A skewed, PC girder bridge was analytically investigated to determine the effect of a skew angle on the response of the bridge to lateral impacts. The skewed bridge that was selected for this study has a 20.4° skew angle, four spans, three frame-type piers, two integral abutments, and five PC girders. Each end span is 45 ft – 9 in. (13900-mm), long and each inner span is 96 ft – 6 in. (29400-mm) long. The bridge girders and deck have similar geometric and material properties and have the same connections between the diaphragms and the PC girders and RC deck as that for the non-skewed bridge.

INTERMEDIATE DIAPHRAGMS

Two steel and one, RC intermediate diaphragm were considered in this study. The steel diaphragms were an X-braced diaphragm with a horizontal strut and a K-braced diaphragm with a horizontal strut. All three types of intermediate diaphragms are standard diaphragms that are used by the Iowa DOT. Andrawes (4) provided additional descriptive information about these intermediate diaphragms.

FINITE ELEMENT MODELING

For all of the finite-element models, the PC girders and the 8-in. (200-mm) thick, RC slab were modeled using solid elements. An isometric view of a single-span, finite-element model is shown in Fig. 1. The 10-in. (250-mm) thick, RC intermediate diaphragms, shown in Fig. 2, were modeled by solid elements. A three-dimensional, truss element was selected to idealize the coil rods that connected the RC intermediate diaphragm to the PC girders. For the two, steel, intermediate diaphragms that are shown in Figs. 3 and 4, shell elements were used to model the horizontal struts, bent plates, and flat plates. Beam elements were used to model the X-braced and K-braced members.

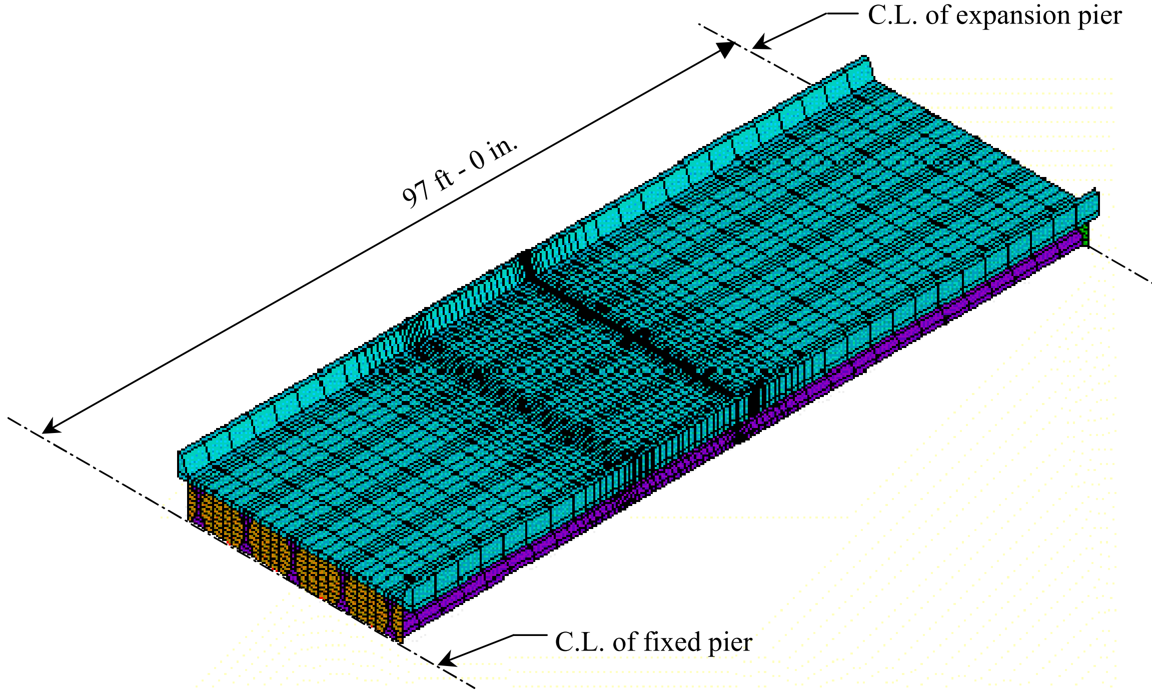
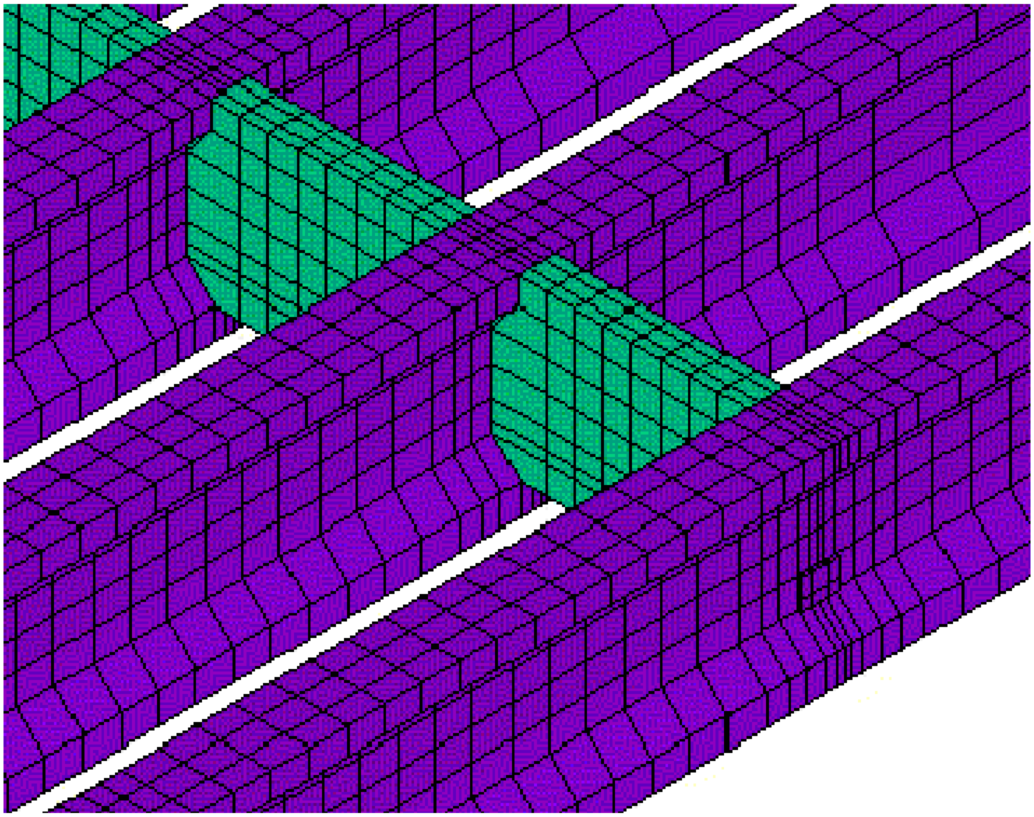


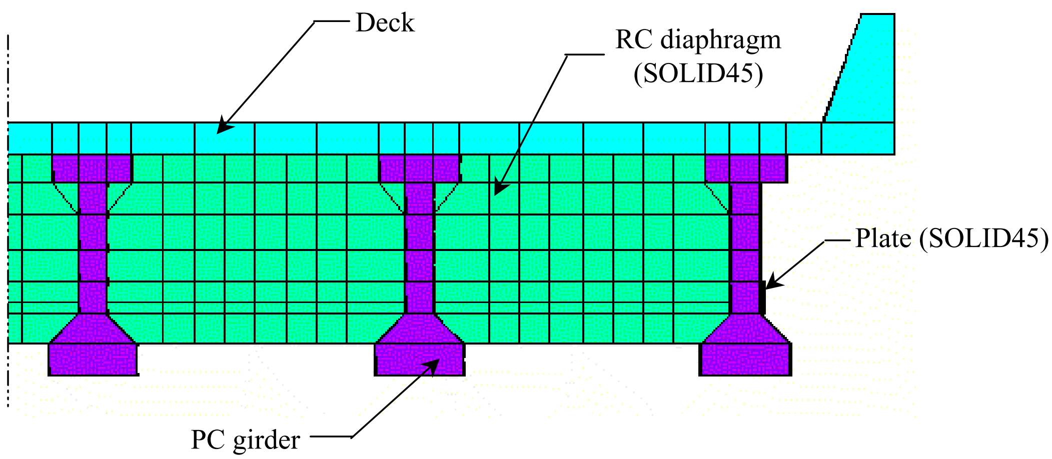
FIGURE 1. Finite-Element Model of the Interior Span of a Non-Skewed Bridge (1 ft. = 305mm)

Finite-element modeling of the connection between the members in the steel intermediate diaphragms and the bent plates that were used to attach the diaphragms to the webs of the PC girders included the use of rigid-link elements; contact elements; and pairs of counteracting, compressive forces. The length of the rigid-link elements accounted for the eccentricity between the center of gravity for the diaphragm members and the outstanding leg of the bent plate. Contact elements with a coefficient of friction equal to 0.33 were placed between the interface surfaces of the connection to permit slippage between the parts. The counteracting, compressive forces represented the clamping force that is developed in a connection when high-strength bolts are installed in a fully-tensioned condition.

The bolts that connected the steel diaphragms to the PC girders were modeled as three-dimensional, truss elements. Since the high-strength bolts for the steel diaphragms and the coil rods for the RC diaphragms provided the connections between the diaphragms and the girders, common nodes were not used between the elements for the diaphragms and the elements for the bridge deck or girders. Contact elements were utilized on all interface surfaces where slippage and separation might occur along those surfaces. The concrete and steel, material strengths were assumed to be linearly elastic.

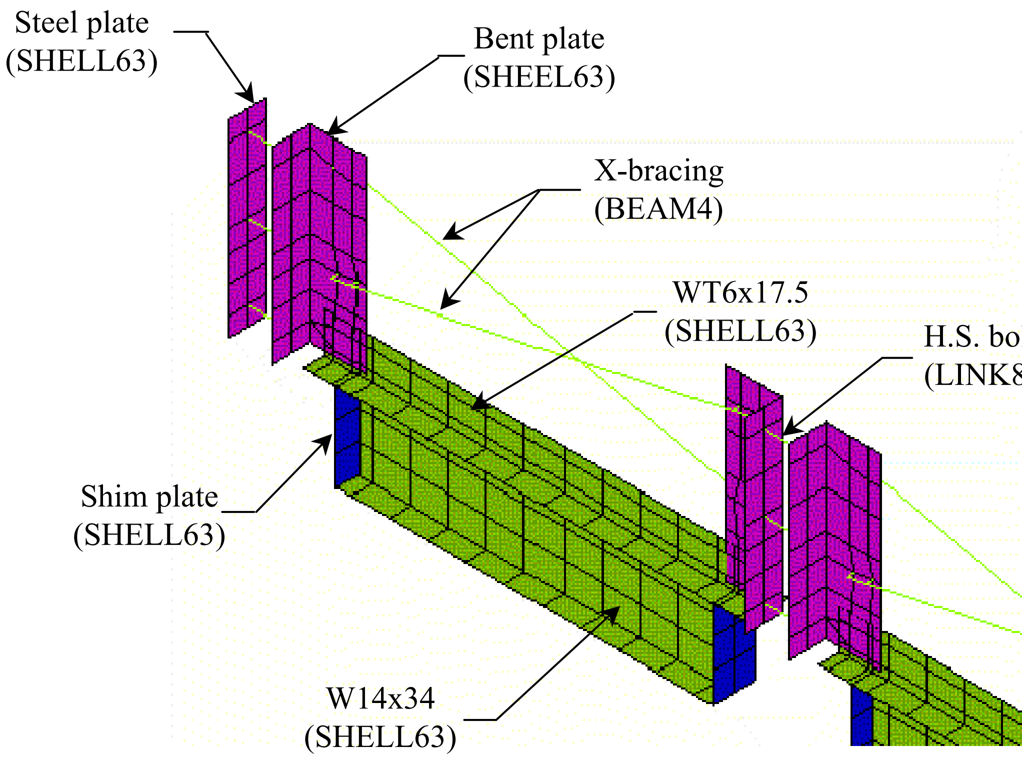


(a) Isometric View

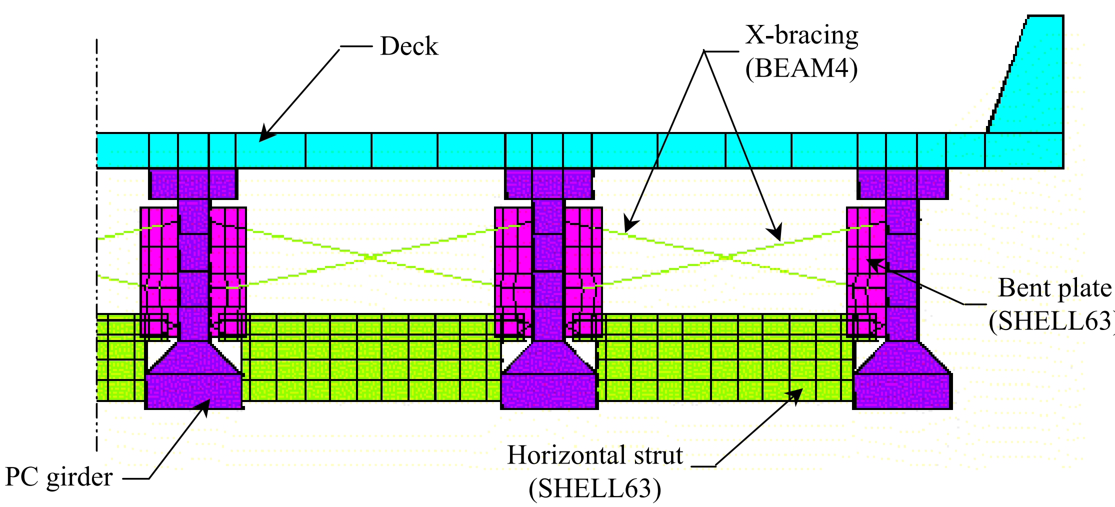


(b) Elevation View

FIGURE 2. Finite-Element Model of the Reinforced Concrete Intermediate Diaphragm

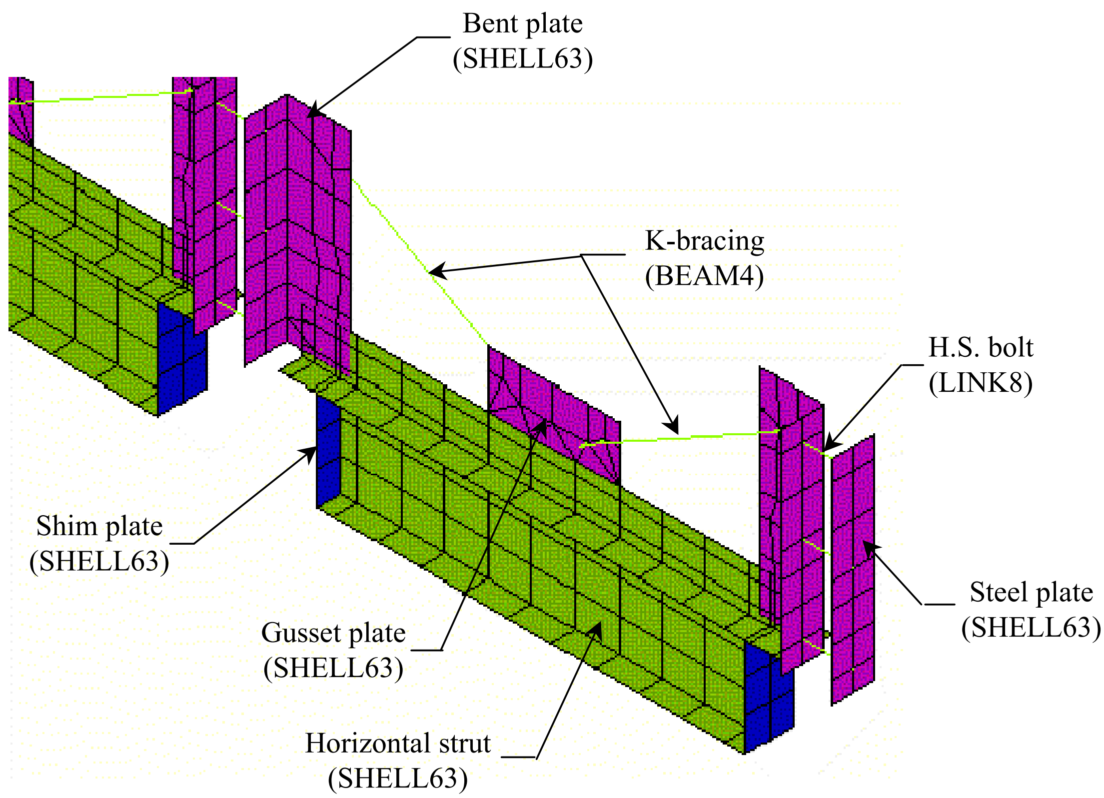


(a) Isometric View

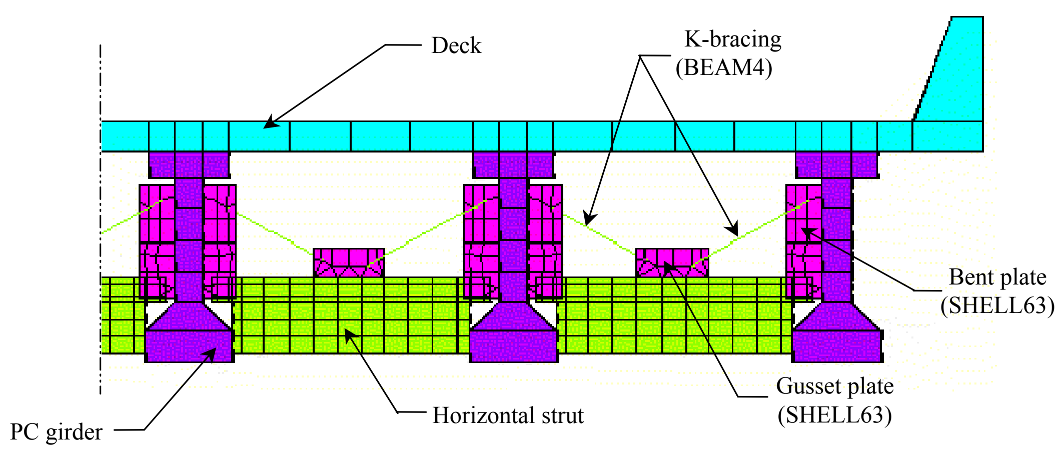


(b) Elevation View

FIGURE 3. Finite-Element Model of the Steel X-Braced Intermediate Diaphragm



(a) Isometric View



(b) Elevation View

FIGURE 4. Finite-Element Model of the Steel K-Braced Intermediate Diaphragm

LOADS

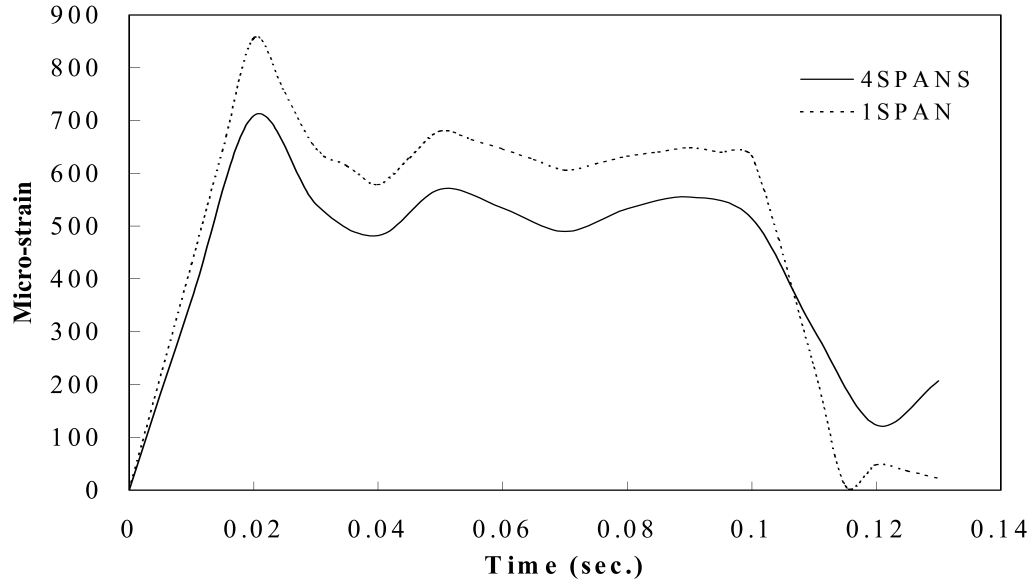
The main factors that are associated with a moving object and influence the magnitude of an impact load that is generated by the object as it impacts a surface are the object's mass, speed, geometrical configuration, and hardness. A search of the published literature that addressed vehicle or object impacts did not reveal any information regarding over-height-vehicle loads striking bridges. Since the main objective of this research was to conduct a comparative study that evaluates the effectiveness of different types of intermediate diaphragms in minimizing structural damage to a bridge superstructure, a precise forcing function did not need to be defined.

One scenario that may occur when an over-height-vehicle load strikes a bridge is as follows: First, the over-height object would impact the bottom flange or web of the first exterior girder. Then, because the vehicle would not suddenly stop, the object being transported could displace downward, as the vehicle-suspension system reacts to the impact, which would allow the object to pass beneath the girder. As the vehicle-suspension system rebounds, the object could displace upwards and cause additional impacts with some or all of the other bridge girders at either their bottom flange or somewhere on their web. Multiple-girder impacts were not included in this study because the reduction in the impact-force magnitude resulting from a reduction in the speed of the vehicle after the first impact is unknown. In this research, a single-impact load was applied on either exterior girder for a single direction of travel, since these loading conditions induce the most severe responses for an impacted girder.

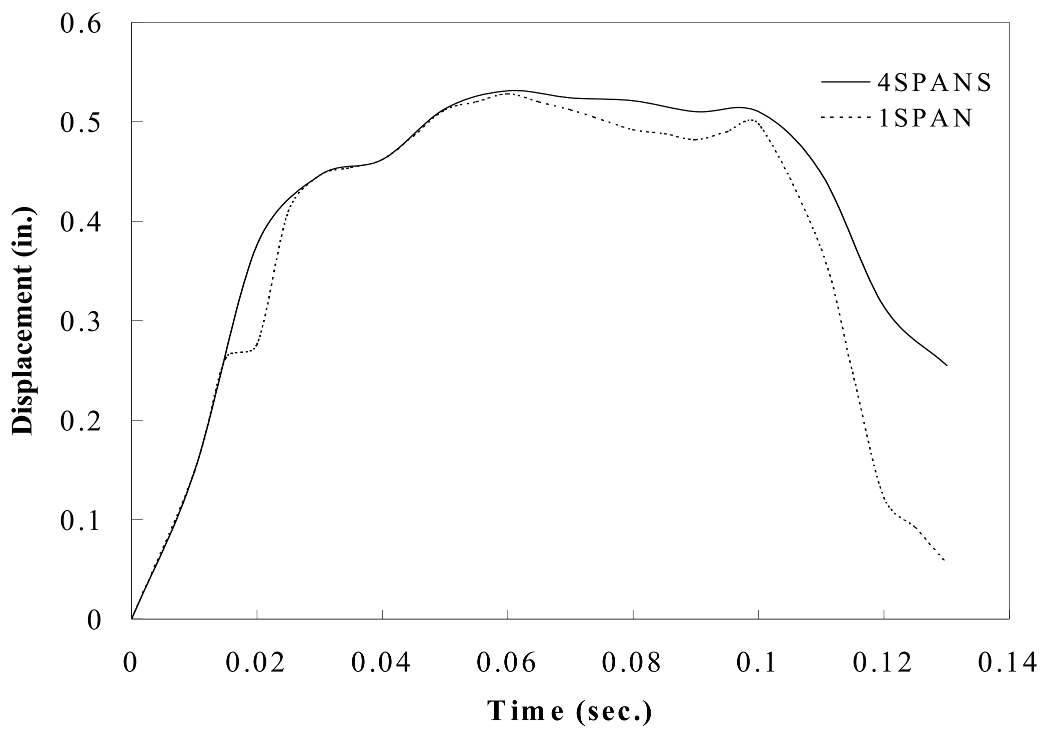
To simulate the impact resulting from an over-height-vehicle load passing beneath the bridge and striking the bottom flange of an exterior bridge girder, an impact load was defined by its duration time and magnitude. Based on published articles that discussed car-crash tests, a 0.10-sec., impact-duration time was selected. A constant-magnitude, impact force was selected to keep the maximum, principle-tensile strains near to or below the modulus-of-rupture strain for the concrete in the PC girders when intermediate diaphragms were incorporated in the analytical models. Two different rectangular, impact pulses were used to represent an impact force that is generated by an over-height-vehicle collision with a PC girder. A maximum load of 120 kips (530 kN) and 60 kips (270 kN) were selected when the impact load was applied directly at and away from, respectively, the midspan of the bridge.

ANALYSES OF ONE SPAN AND COMPLETE BRIDGE MODEL

To simplify the analytical work and to reduce the computational requirements, an investigation was conducted to determine whether a finite-element model for only an interior span of a four-span bridge would predict, with sufficient accuracy, the PC girder strains and displacements that are obtained from an analysis of a four-span, finite-element model. Even though the pier structures were not included in the finite-element models, their lateral stiffness was represented by horizontal springs at the pier locations. These two analytical models did not have any intermediate diaphragms.

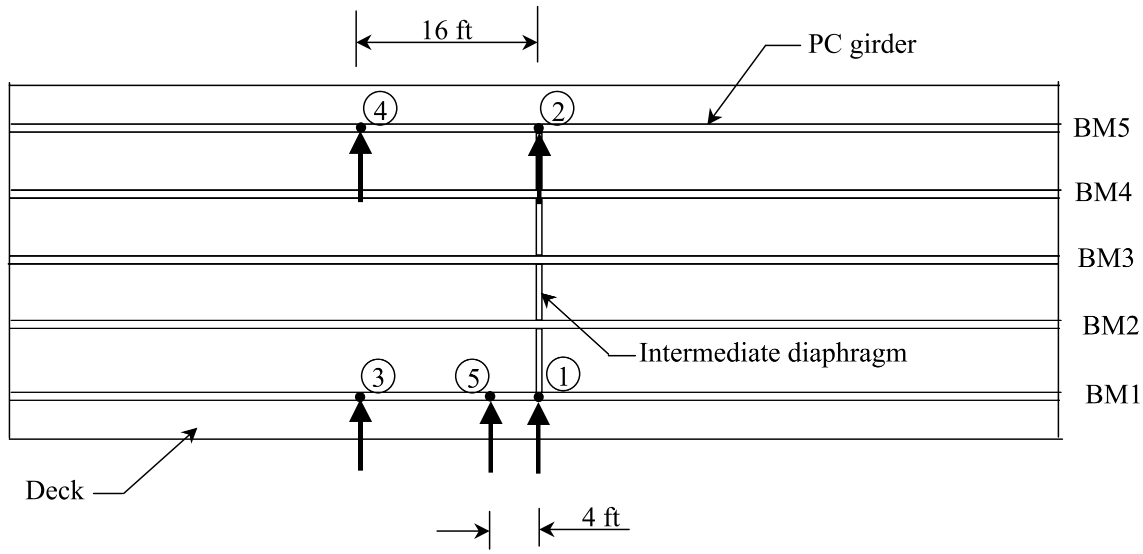


(a) Maximum Principal-Tensile Strains

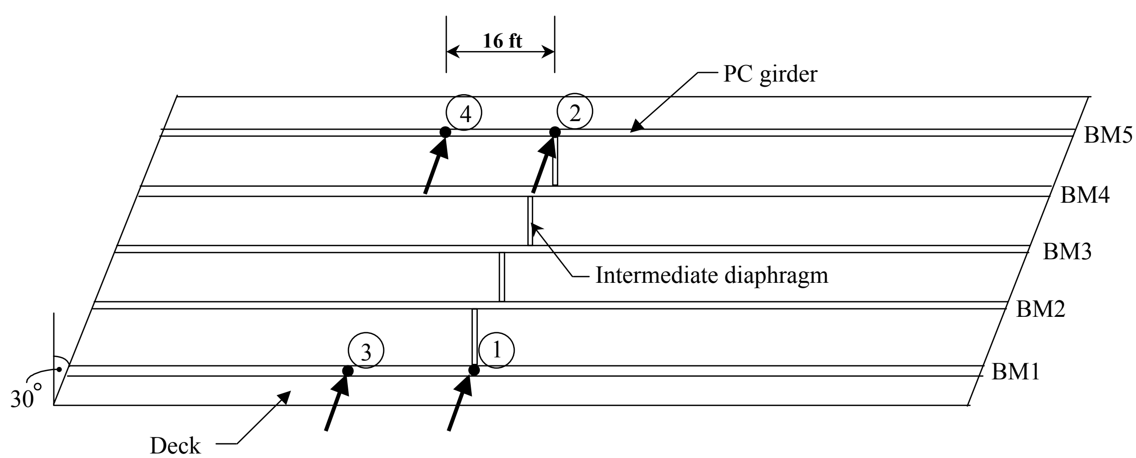


(a) Horizontal Displacements

FIGURE 5. Finite Element Results for the Four-Span and One-Span Models When the Load is Applied at Mid-Span of Girder BM1 (1 in. = 25.4 mm)



(a) Locations of Loading Points for the Non-Skewed Bridge



(b) Location of Loading Points for the Skewed Bridge

FIGURE 6. Loading Points (1 in. = 305 mm)

The maximum, principal-tensile strains in and the corresponding, horizontal displacements for the loaded girder are presented in Figs. 5a and 5b, respectively, of the four-span model and the one-span model. These strains and displacements were induced by a 120-kip (530-kN) lateral-impact load with a 0.10-sec. duration time that was applied at the mid-span of the first exterior girder (girder BM1 shown in Fig. 6a) for traffic that travels under the bridge. These strains occurred in the girder cross section containing the applied load for the top fibers of the girder web. The displacements were at the bottom flange and at the mid-span of the girder. Both the strain and displacement responses were the largest of those load effects for the loaded girder that were predicted by the finite-element models. The figure shows very similar strain and displacement behaviors for the two analytical models. Only about a 15% difference occurred in the magnitudes of the maximum, principal-tensile strains. The displacement responses predicted by the two, finite-element models were essentially identical for the 0.10-sec. period for the applied impact load. The differences in the girder responses that were predicted by the two, finite-element models and shown in Fig. 5 were not significant enough to affect the objectives of the research. Therefore, the single-span, finite-element model was selected to adequately represent the girder responses to lateral-impact forces. A similar, single-span model was developed for the skewed bridge. Andrawes (4) provided additional information regarding the finite-element models for both the non-skewed and skewed bridges.

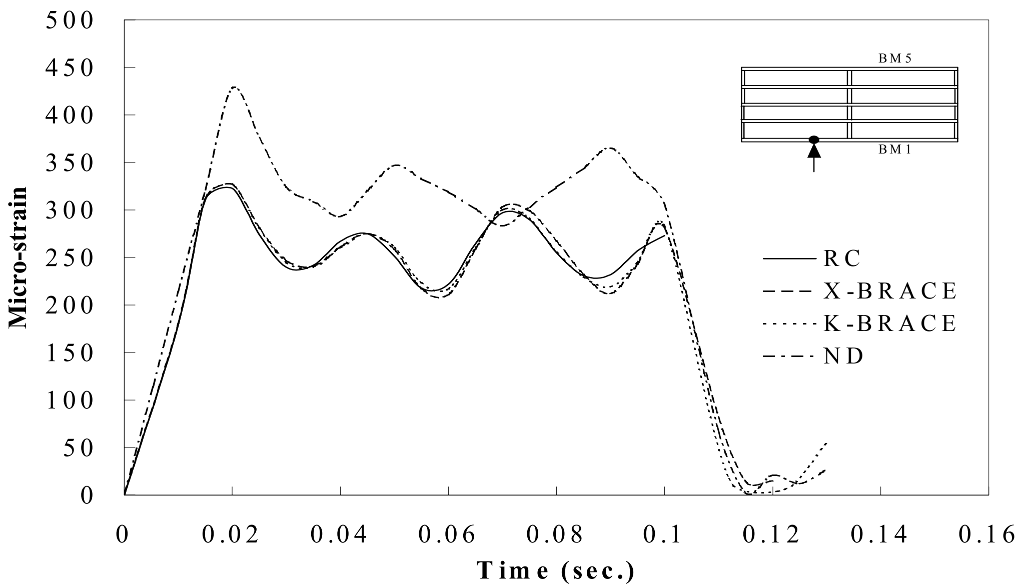
ANALYSIS OF PROTOTYPE BRIDGES

For the non-skewed bridge, the horizontal, impact load was applied to the bottom flange of an exterior girder at one of five locations shown in Fig. 6a. Load positions 1 and 2 were at the mid-span of girders BM1 and BM5, respectively. These locations matched the location for the intermediate diaphragms. Load positions 3 and 4 were applied at 16 ft (4880 mm) to the left of the intermediate diaphragm and on girders BM1 and BM5, respectively. As the analytical study progressed, the researchers decided to apply an impact load at load position 5 that was at 4 ft (1220 mm) to the left of the intermediate diaphragm location. This fifth, load location was considered to account for an impact load that was applied close to but not at the intermediate diaphragm location. Figure 6b shows four, impact-load locations for a skewed bridge. An impact load was not placed at load-position 5 for the skewed bridge.

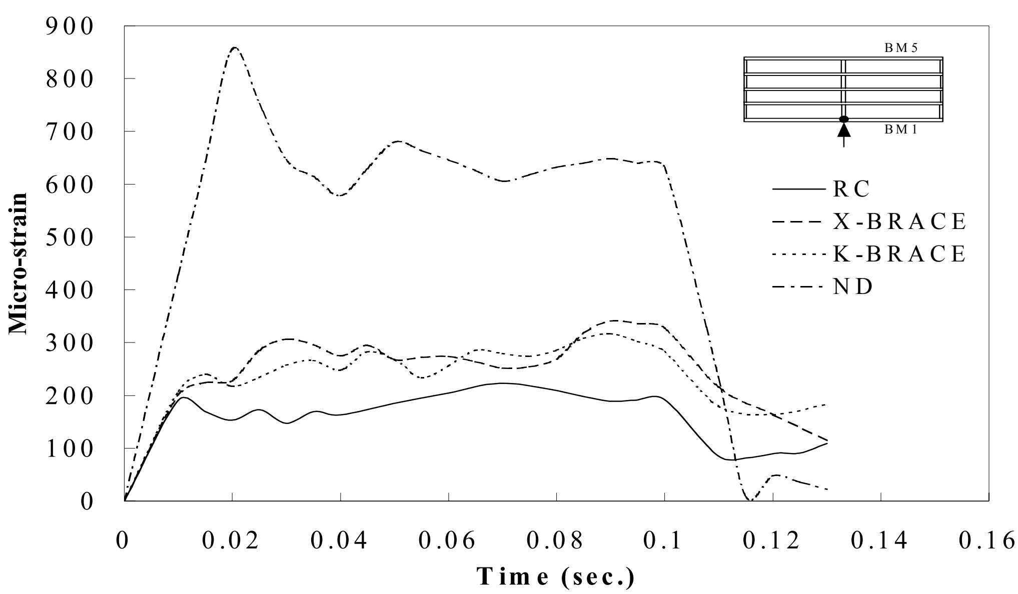
When the impact load occurred at the location of an intermediate diaphragm, as shown in Fig. 7a, the decrease in the maximum, principle-tensile strains from those strains for the bridge without intermediate diaphragms (ND curve) was significant and dependent on the type of intermediate diaphragm. When the impacted load occurred at 16 ft (4880 mm) from the midspan of girder BM1, as shown in Fig. 7b, those strains were about 25% less than the corresponding strains for the bridge without intermediate diaphragms and basically independent on the type of intermediate diaphragm.

Figures 8a and 8b show the magnitude of the maximum, principal-tensile strains in the impacted girder (girder BM1) and the first interior girder (girder BM2), when the impact load is applied at the intermediate diaphragm location for a non-skewed bridge and a skewed bridge, respectively. For both bridge alignments, the RC intermediate diaphragms provided the largest degree of impact protection to the impacted girder, as indicated by the heights of the strain bars shown in these graphs. The steel, X-brace and K-brace, intermediate diaphragms provided essentially the same amount of impact protection to the impacted girder, as indicated by the essentially, equivalent-strain magnitudes. The presence of the RC intermediate diaphragms reduced the magnitude of the maximum, principal-tensile strain to about 28% and 35% of that strain for the no intermediate diaphragm condition for the non-skewed and skewed bridges, respectively. The use of either the steel, X-braced or K-braced, intermediate diaphragms reduced the maximum, principal-tensile strain in girder BM1 to about 40% and 60% of that strain for the no intermediate diaphragm condition for the non-skewed and skewed bridges, respectively. Figure 8a shows that the magnitude of the maximum, principal-tensile strains, which are induced in the first interior girder (girder BM2) for the non-skewed bridge, are smaller for the bridge with RC intermediate diaphragms than for the bridge with either steel, X-braced or K-braced, intermediate diaphragms. However, for the skewed bridge, Fig. 8b shows that the use of either configuration for the steel intermediate diaphragms will induce less strain in girder BM2 than that for the same bridge with

RC intermediate diaphragms. This difference in the behavior between the skewed and non-skewed bridges was attributed to the staggered alignment of the intermediate diaphragms for a skewed bridge. When intermediate diaphragms are in alignment for a non-skewed bridge, a larger amount of the impact force, which is induced in the first intermediate diaphragm, is transferred to the rest of the diaphragms than that for the skewed bridge where the intermediate diaphragms are not in alignment.

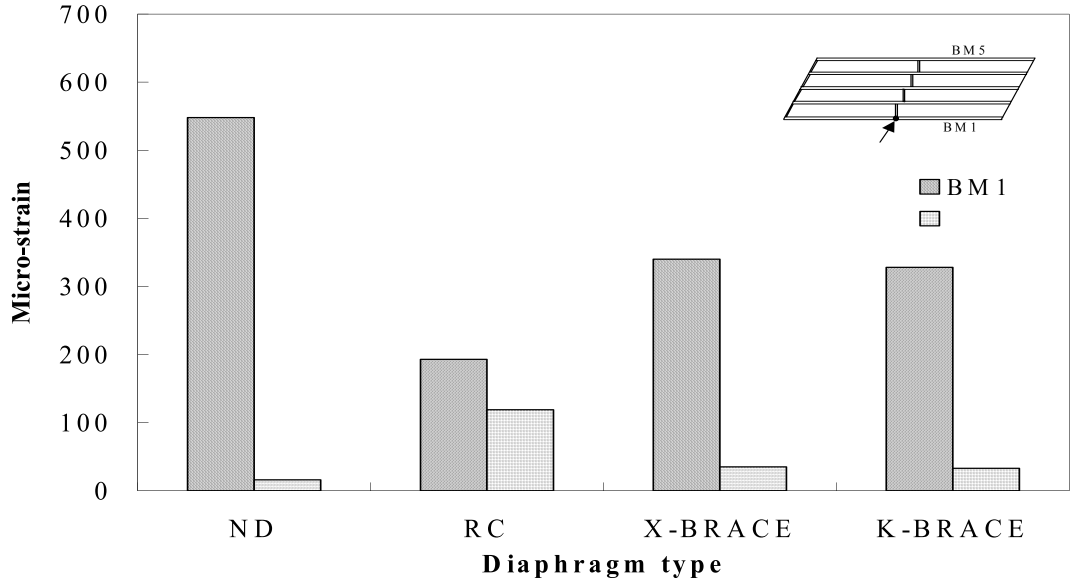


(a) Load at the Mid-Span

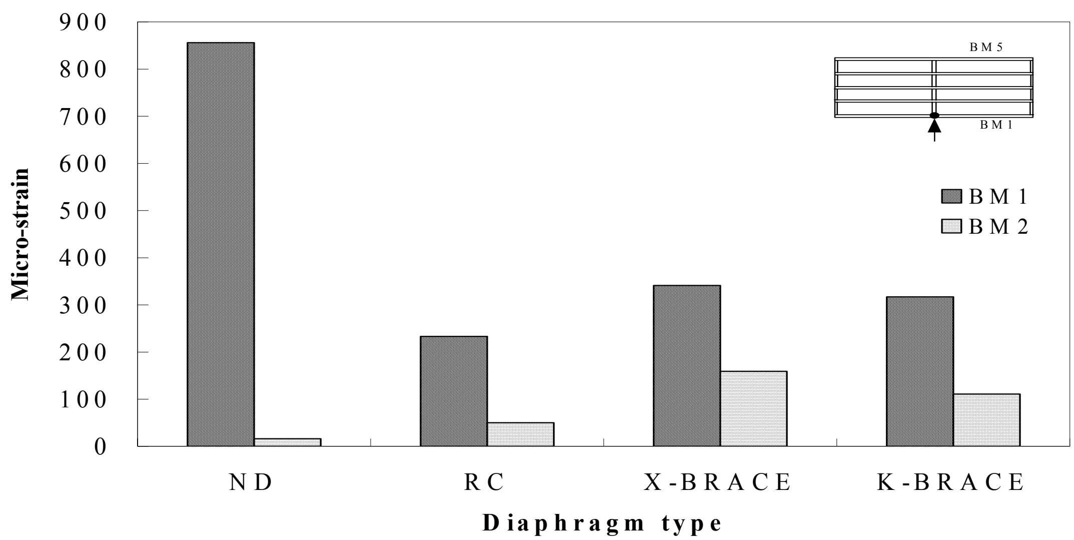


(b) Load at 16 ft (4880 mm) from the Mid-Span

FIGURE 7. Maximum Principal Tensile-Strain in Girder BM1 for the Non-Skewed Bridge



(a) Non-Skewed Bridge



(b) Skewed Bridge

FIGURE 8. Maximum Principal-Tensile Strains in Girders BM1 and BM2

CONCLUSIONS

The results summarized in this manuscript illustrated that when a lateral-impact load was applied directly at the diaphragm locations in a bridge structure, the RC intermediate diaphragm provides the bridge girders with a higher degree of impact protection than that provided by either of the two types of steel intermediate diaphragms that were presented in this study. However, when an over-height-vehicle load strikes a bridge girder away from a diaphragm location, the degree of impact protection provided by each of the three types of intermediate diaphragms was almost the same.

ACKNOWLEDGEMENTS

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Iowa Department of Transportation Technical Training Activities

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ABSTRACT

Properly trained technicians in the areas of material testing and construction inspection are a major factor in the quality of highways and structures. Technicians from both government and industry perform testing and inspection. To ensure that these technicians have the skills to perform testing and/or inspection on construction projects, training and examinations need to be provided. The training can be provided in a number of ways through classroom instruction, hands-on laboratory classes, and on-the-job training. Examinations for certification are given to make sure the individuals fully understand the area they are testing and/or inspecting.

One of the challenges of the Iowa Department of Transportation Technical Training and Certification Program is to ensure that all technicians who are performing testing and/or inspection on projects in Iowa are qualified. This involves holding over 200 classes annually with approximately 3,000 individuals participating.

The Iowa Department of Transportation is using a new approach to dealing with staff shortages by using Maintenance Equipment Operators to perform testing/inspection duties and construction and materials inspectors to perform maintenance duties in their off seasons respectively. This has brought a whole new set of challenges in the area of cross training at the Iowa Department of Transportation.

Key words: certification—construction inspection—cross training—materials testing—technical training

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INTRODUCTION

A good training program is an important part to the success of any organization. Individuals that have been properly trained in the skills they will be using will not only be more competent at their jobs, but as important, will feel more comfortable performing their job duties.

Change in the structure of organizations is requiring that employees have multiple skills and flexibility. This is true for the Iowa Department of Transportation (Iowa DOT), contractors, and consultants that perform testing and inspection on Iowa's construction projects.

The Iowa DOT's Technical Training and Certification Program (TTCP), which is now over 30 years old, continually changes to meet the needs of the technicians working on Iowa's construction projects. The purpose of this paper is to describe the Iowa DOT TTCP. This will include problems faced in providing adequate training for technicians and how Iowa's TTCP is working to alleviate those problems. This paper discusses the history, progression, and future of technical training in Iowa. The paper also touches on Iowa's activities in regional and federal training efforts.

HISTORY OF IOWA'S TECHNICAL TRAINING PROGRAM

Technical training in the Iowa DOT originally was accomplished with on-the-job training. Testing and inspection procedures were passed down from technician to technician by working with an experienced technician for a period of time. There was a more "formal" training for some of the technicians, and then they were responsible to take what they had learned and pass it on to others in the field offices.

A certification program in aggregate was developed in the 1970s when quality control by aggregate producers was initiated. Training courses were held for Iowa DOT and producers' technicians. Exams were taken and certifications were issued to those who successfully passed the exam. Certification was required by both Iowa DOT and industry to perform aggregate inspection.

The Quality Control/Quality Assurance Program (QC/QA) expanded in the mid 1980s to include portland cement concrete (PCC) and hot mix asphalt (HMA) plant inspection and profilograph testing. The QC/QA program has continued to expand and training and certifications were added to the point we are at today. The Iowa DOT TTCP was formalized in 1995 to organize and standardize Iowa's technical training.

There was a change in the program due to changes at the federal level (Federal Register, Vol. 60, No. 125, June 29, 1995). The Federal Highway Administration (FHWA) started requiring qualified personnel in their Code of Federal Regulations (23 CFR Part 37), which states that after June 29, 2000, all sampling and testing data to be used in the acceptance decision or the Independent Assurance (IA) program shall be executed by qualified sampling and testing personnel.

Iowa already had their certification program in place, which was approved by the FHWA to meet the requirements of the Code of Federal Regulations. Iowa was ahead of many states that didn't have any formal training or qualification program in place. This did, however, put a demand on the TTCP to train and certify all individuals that would be performing acceptance testing. There were several project inspectors that had been trained to perform tests but had never been through the formal TTCP to receive certification.

CURRENT PROGRAM

The current Iowa TTCP is very different today from its inception in the 1970s. It has gone from having a few hundred technicians certified to 3,000, from a handful of classes to close to 300. The TTCP now instructs and certifies personnel from industry, counties, cities, and DOT in the areas of aggregate, PCC, HMA, nuclear gauge, profilograph, prestress, and soils. The current program has over 9,000 certifications issued.

Until three years ago the Iowa DOT arranged and instructed all of their technical training classes. The growth of the TTCP and the downsizing of departments required more classes with fewer instructors available. The Iowa DOT began working with Des Moines Area Community College (DMACC) to instruct many of the certification classes. This joint effort between the TTCP and DMACC has been successful. DMACC has a facility to house all the TTCP training, complete with a lab facility and computer training set-up. They also supply instructors for the courses using outside sources or hiring DOT employees in an adjunct or in-kind status. DMACC's involvement with the TTCP is with the training portion. The Iowa DOT exclusively handles the certification program.

TTCP classes run from October through June. Class sizes range from 8 in some of the laboratory classes to 25 in the lecture classes. All of the higher-level certification classes are held at the DMACC facility in Boone, Iowa. Most of the lower level certification classes are held in each of the Iowa DOT district materials facilities.

The classes consist of technicians and inspectors from the Iowa DOT, counties, cities, contractors, producers, and consultants, ranging from newly hired employees to engineers. The diversity of the group can be a struggle for instructors but can also be an advantage with more experienced individuals helping those that are new and by the sharing of experiences. Table 1 lists the certification classes that are available through the TTCP and brief course descriptions.

The TTCP also develops and administers training for informational purposes. These courses do not include exams or certifications but are to help technicians better understand skills they may be performing. Table 2 is a list of courses that are offered annually to improve technician skills.

The TTCP also sponsors specialty training that is offered as needed in a variety of areas such as bolt inspection, wood post inspection, instructor development, etc. One of the goals of the TTCP is to ensure technicians receive the information needed to perform their jobs properly, which requires a variety of courses since these technicians have a number of job duties.

The TTCP conducts recertification classes each year. Technicians must recertify every five years in each level of certification they hold. The recertification classes are normally a day in length including a recertification exam. The TTCP also has a decertification process for technicians, which allows the Iowa DOT to decertify individuals for a number of reasons including fraudulent practices and reports. The Iowa DOT has an Instructional Memorandum (I.M. 213) that covers certification, recertification, and decertification.

A registration and information booklet is published each fall by the TTCP. This booklet is distributed to all state offices, counties, cities, contractors, and consultants. (It is also available at Iowa DOT materials offices and appears on the Iowa DOT website at www.dot.state.ia.us/materials/training.htm.) The booklet contains information about the TTCP, course competencies, class schedules, and registration forms.

TABLE 1. Iowa DOT Certification Training

Certification	Length	Description
Level I Aggregate	0.5 days	Instructs the individual on proper sampling techniques used in sampling aggregates for testing.
Level II Aggregate	3.5 days	Instructs the individual on aggregate production and testing of aggregates.
Level I HMA	5 days	Covers the duties of the technician inspecting HMA plants and HMA materials testing.
Level II HMA	5 days	Instructs the individual on developing HMA mix designs and the testing and calculations used in the mix design process.
Level I PCC	2 days	Instructs the individual on performing tests on fresh concrete including air, slump, temperature, etc. The course covers developing maturity curves and maturity testing. Testing beams and cylinders and calculating yields is also included.
Level II PCC	4 days	Covers the duties of the technician inspecting PCC plants. The course also instructs the individual on calculating batch weights, cement yields, water/cement ratios, and adjusting batch weights for moisture loss or gain in aggregate.
Level III PCC	4 days	Instructs the individual on developing PCC mix designs and the testing and calculations used in the mix design process.
Profilograph	2 days	Instructs the individual on operating a profilograph, reducing profilograph traces, and the specifications for profilograph usage.
Prestress	3 days	Covers inspection procedures at a prestress/precast plant. This would include tensioning and detensioning, concrete placement, strength requirements, etc.
Nuclear Gauge	2 days	Instructs the individual on the proper use of a nuclear gauge and the safety requirements for handling and transporting gauges.
Grade Technician/Soils	2 days	Instructs the various tests used in soils inspection and information on the types of soils. Currently only instructed as a pilot on an as-needed basis.

TABLE 2. Iowa DOT Non-Certification Training

Course	Length	Description
Basic Materials	1 day	Gives the technician a brief background in all areas of materials inspection.
Basic Math	1 day	Covers basic math computations and the use of a calculator.
Practical Math	1 day	Covers math problems that the inspector will encounter on a project.
Superpave for Practicing Engineers and Technicians	1 day	Covers the decisions required to select materials, develop pavement designs, review mix designs, inspect construction techniques, and monitor the pavement's performance.
Monitor Administration	1 day	Instructs the administrative duties and problem spotting/solving of plant monitors.
Contract Administration	2 days	Covers the basic administration duties of the construction inspector.
Basic Plan Reading	1 day	Covers the basics of plan reading and working with general plans and does not go into specific types of plans.
HMA Paving Field Inspection	2 days	Instructs field inspection of HMA resurfacing/paving and covers equipment, mix placement, problems, solutions, and roles of the inspector and contractor.
PCC Paving Field Inspection	2 days	Instructs field inspection of PCC resurfacing/paving and covers equipment, mix placement, problems, solutions, and roles of the inspector and contractor.
Structure Field Inspection	2 days	Instructs field inspection of bridge and culvert construction and covers bridge and culvert components, their function, and design intent.
Grade Technician	2 days	Instructs preparation for grading, soil types, plan reading of soil sheets, soil behavior, drainage, and grading equipment.

FUTURE OF THE TTCP

The Iowa TTCP continues to grow with more areas of certification being required and more skills being taught. Many times employees are unsure of the skills they will need to perform job duties or what training is available to help obtain certain skills.

A new approach to training within the Iowa DOT is the formation of the Iowa DOT Highway Division Training Academy. The academy is being developed for Iowa DOT employees to assist them in understanding the skills needed for their jobs and the training necessary to obtain those skills. The Iowa DOT recently downsized their workforce and is utilizing employees to do a variety of skills rather than focus on one area. Examples include an equipment operator performing maintenance duties in the winter and materials testing or construction inspection in the summer or a construction technician doing a similar switch. This triggered the inception of the Training Academy to make sure everyone is aware of the competencies required in each area of work they perform and the training that is available to learn the skills needed.

The Iowa DOT Training Academy will consist of matrices for job classifications in materials, construction, and maintenance. This is a starting point for the academy but this could expand into other areas of the Iowa DOT as the academy progresses. These matrices will list all the skills an individual needs for the classification. Each matrix is broken down into basic, intermediate, and electives. The individual would have or need to take the basic training in the matrix when hired or shortly after. Intermediate training would take place within approximately the first year of employment. Finally, electives could be taken as the employee desires or as needed to perform special job duties.

Most newly hired individuals are brought into the Iowa DOT as equipment operators. They can promote into upper maintenance, construction, or materials classifications as job openings develop. Since these individuals are new to the Iowa DOT and are required to perform a variety of jobs, there will be a special basic training for this group. This training will be offered in three one-week segments. One segment will cover employee development, computer training, and safety and will be presented throughout the year at various times. Another segment will be on maintenance operations and will cover the skills of an equipment operator and will be offered in the fall before the employee starts winter maintenance duties. The third segment will cover construction inspection and materials testing and will be offered in the spring before summer construction operations begin and will cover the skills of the construction and materials technicians.

CHALLENGES OF EDUCATING THE TECHNICAL WORKFORCE

The Iowa TTCP and the Training Academy are focused on providing good educational opportunities for Iowa DOT employees, producers, contractors, consultants, and other agencies. This can be very challenging for both agency and industry. Fewer employees are performing more job duties in both the public and private sector. As individuals are hired they are many times put directly on a project without experience or proper training. To make a bad situation worse, because of the lack of workers, these new employees are sometimes left without anyone to oversee their work.

Another problem encountered, especially in the private sector, is the constant turnover of employees. Often a contractor sends a newly hired employee through training and makes sure they are certified to perform their job duties and the following construction season the individual doesn’t return to the employer. As a result, the employer has to start over. This makes it difficult to maintain an experienced staff. It also increases the number of individuals that are in the training/certification classes each year, which requires the TTCP to provide more classes.

The TTCP is working to try to maintain a variety of training available to industry personnel. We are including more of a hands-on approach in our courses to give inexperienced technicians an opportunity to perform skills in the classroom with an instructor present. The TTCP is constantly updating and organizing to make the training and certification program more accessible through better communication with field offices and industry organizations. DMACC, by providing much of the TTCP training, has also taken a strain off Iowa DOT staff. The Training Academy will clarify and organize our in-house training program for Iowa DOT employees.

Community colleges and universities in Iowa are providing qualified technicians for the construction industry. One of these, DMACC, has a Civil Engineering Technician (CET) program that focuses on technicians for the highway industry. The CET program has worked with the TTCP to train and certify students in materials testing and construction inspection. This provides students a background when they first enter the highway construction industry. There are other organizations that provide technical training in Iowa. Some of the organizations that provide training include the Iowa Concrete Paving Association, Asphalt Paving Association of Iowa, and the Center for Transportation Research and Education.

OTHER TECHNICAL TRAINING EFFORTS

The Iowa TTCP is involved in regional and national efforts, which helps to strengthen Iowa's program and contributes to other states' programs. Iowa is a lead state in the Multi-Regional Training and Certification (M-TRAC) program. This regional group consists of 12 of the mid-western states and promotes education of technicians. They have worked toward reciprocity of technician certification between states, regional material development, uniform test procedures, and coordinator exchange of state information.

Iowa's TTCP coordinator represents the M-TRAC group on the Transportation Curriculum Coordination Council (TCCC). The TCCC is a federal group consisting of FHWA, Regional Technical Training and Certification groups, industry organizations, American Association of State Highway Transportation Officials subcommittees, National Highway Institute, and Transportation Research Board. This group was developed to coordinate regional groups and states material development to avoid duplicating efforts. The group has developed matrices in materials, construction, maintenance, safety, and employee development. The matrices show skills necessary for the levels of employees involved in highway construction and the training necessary to obtain these skills. The group is looking at the training gaps in the matrices and is working on material development to fill the gaps. The TCCC has already worked with groups on the development of materials in a number of subject areas including drilled shafts, driven piling, QC/QA and web-based instruction in design. The TCCC is developing a website that will assist states in finding materials they can use in their training program.

CONCLUSION

The Iowa Technical Training and Certification Program is a large educational program that assists in providing skilled technicians and inspectors for the Iowa DOT and industry. The TTCP faces numerous challenges with downsizing and the development of a diverse workforce. The TTCP is meeting the Federal Code of Regulations by providing qualified technicians to perform testing and inspection on Iowa's construction projects. The goal of the TTCP is to improve the quality on Iowa's construction projects by promoting the education and training of the technicians involved.

Evaluation of Premature PCC Pavement Longitudinal Cracking in Colorado

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ABSTRACT

This report presents an evaluation of several portland cement concrete pavements with premature longitudinal cracking. All of the locations discussed are in Region 1 of the Colorado Department of Transportation. Included in this report is an overview of the causes of the premature longitudinal cracking, a description of field and laboratory investigations, and a list of strategies to eliminate such occurrences.

Although there were some isolated cases of differential settlements related to the high swell potential (high PI/lower R-value) of the subgrade soil and/or poor subgrade compaction, this study found the shallow depth of the longitudinal saw-cuts at the shoulder joint to be the main reason for longitudinal cracking. Malfunctioning paver vibrators and unsuitable soils also contributed to the problem. The tests also proved that the wider slab design (14-foot slab) was not a contributing factor.

Implementation: This study resulted in two new specifications: (1) requiring the engineer to measure saw-cut depth at intervals of 1 per 1/10 of a mile (528 feet); (2) requiring paving contractors to equip their paving machines with vibrator monitoring devices.

Key words: premature longitudinal cracking—saw-cut depth—vibrator monitoring devices

Proceedings of the 2003 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2003.

INTRODUCTION

Many factors are responsible for premature longitudinal cracking in portland cement concrete (PCC) pavements. They are primarily improper construction practices, followed by a combination of heavy load repetition and loss of foundation support due to heave caused by frost action and/or swelling soils. This study focused on distresses related to improper construction practices.

Possible construction related causes of premature longitudinal cracking include the following:

- Time of saw-cutting of the longitudinal joints at the shoulders and centerline
- Depth of the longitudinal saw-cuts at the shoulder and centerline joints
- Vibrator trails caused by malfunctioning vibrators on the paver
- Improperly treated swelling soils with high plasticity index and lower R-value
- Inadequate compaction of the subbase soil
- Misaligned dowel bars

Design features such as slab thickness and width, and base/subgrade type and stiffness, strength, and drainage can cause, or have dramatic impact on the severity of, premature longitudinal cracking. In addition, materials properties, including the mix constituents (aggregate type, cement type, admixtures, etc.) and their proportions, may influence longitudinal cracking. Whatever the cause, premature longitudinal cracks have detrimental effects on the overall performance of portland cement concrete pavements. This study determined their causes and recommended ways to prevent or reduce their occurrence in future projects.

BACKGROUND

The Colorado Department of Transportation (CDOT) Region 1 has been experiencing premature distresses on some of its concrete pavement primarily in the form of longitudinal cracking. Because of its significant nature, the problem was presented to the Materials Advisory Committee (MAC) for their input and feedback. The MAC recommended establishing a task force to investigate the causes of the longitudinal cracking and to recommend remedial measures. Personnel from CDOT, the Colorado/Wyoming chapter of the American Concrete Paving Association (ACPA), and the paving industry were invited to serve on the task force.

The task force members toured problem areas on I-70 east of Agate, on US-287 south of Kit Carson, and on I-70 west of Deer Trail to evaluate the distresses. The task force reached the consensus that the following could be responsible for these distresses and needed further evaluation:

1. Cracking with differential settlement, attributable to either or both of the following:
   - Unsuitable soils possessing high swelling potential (i.e., high plasticity index/lower R-value).
   - Inadequate compaction of the sub base during construction.
2. Cracking without differential settlement, attributable to improper construction practices or design, caused by one or more of the following mechanisms:
   - Malfunctioning vibrators on the paver, i.e., vibrator frequency set too high causing over-consolidation the concrete mix.
   - Shoulder joint saw-cuts shallower than required by specifications (1/3 of the pavement thickness - D/3 – for 12-foot-wide slabs and 0.4D for 14-foot-wide slabs).
   - Wide (14-foot) slabs.

OBJECTIVES

The primary objectives of this study were as follows:

- To identify and confirm the causes of the premature longitudinal cracking observed at several locations in Region 1.
- To develop strategies to prevent recurrence of the problems.

DATA ACQUISITION AND ANALYSIS

To accomplish the objectives set forth above, the study panel developed and recommended the following tasks:

- Task 1. Visit locations on I-70 near Agate, on I-70 near Deer Trail, and on US-287 near Kit Carson to identify possible causes of the premature longitudinal cracking.
- Task 2. Conduct field and laboratory investigations of the three sites to confirm the causes responsible for the cracking. The field investigation consisted of extracting cores for visual evaluation and to measure saw-cut depth, and surveying the number of longitudinally cracked slabs as a percentage of the total number of slabs in the project. The laboratory investigation consisted of conducting air-void system analysis on cores taken on and adjacent to the cracks.
- Task 3. Analyze the acquired data.
- Task 4. Develop strategies to minimize or eliminate premature longitudinal cracking in the future.
- Task 5. Prepare a final report to document the entire research project. Include recommendations for the implementation of the results as part of the report.
- Task 6. Present the results of the study to the MAC and the ACPA and request implementation of the recommendations on future highway construction projects.

Investigation of I-70 Near Agate

Construction for the I-70 Agate Design-Build Project No. 10458 (mp 338.3 to mp 348.15) started in 1998 in the westbound lanes. The eastbound lanes were completed in 1999. During the month of August 2001, distresses, mainly in the form of longitudinal cracks, were encountered at several locations in the westbound lanes (no distresses were reported in the eastbound lanes). The majority of the cracks were scattered between mileposts 340 and 346. Four cracks, each measuring approximately 200 feet in length, and located about four to five feet to the left of the shoulder paint stripe, appeared to be related to improper vibration practices. These cracks are very straight **(Figure 1)** and at a consistent distance from the edge of the pavement.

At some locations near Agate, cracks have developed with substantial differential settlement across the cracks. They tend to wander back and forth between the center of the lane and the right wheel path **(Figure 2).** These cracks are probably due to under-compaction of the subgrade fill materials or to the high expansive potential of the highly plastic clayey natural soils in the area (or to the combined effects of both).
The following steps were taken to investigate the causes of these cracks and to attempt to stop their further progression:

1. Pairs of reference pins were installed across the longitudinal cracks.
2. Cores were drilled at the ends of the longitudinal cracks to attempt to stop their progression.
3. Steel rebar stitches were installed across a crack to prevent further movement.
4. Cores were drilled on the longitudinal cracks to investigate the concrete.
5. Undisturbed soil samples of the subgrade natural soils were obtained.
6. The cracks were sealed by CDOT Maintenance.

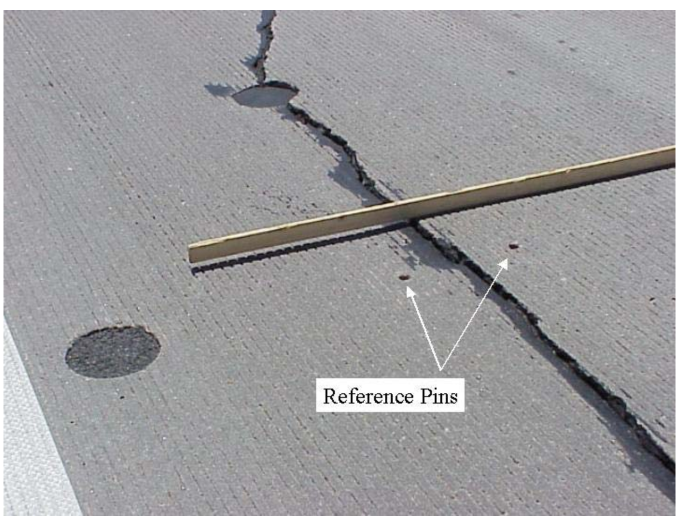

FIGURE 2. Differential Settlement on I-70 Westbound Near Agate


FIGURE 3. Reference Pins Installed Across a Joint

Early in 2001, pairs of steel reference pins (**Figures 2 and 3)** were installed on either side of the longitudinal cracks to provide permanent reference points for monitoring changes in the crack widths and the differential settlement across the crack. Pins were installed at MP's 340.260, 340.453, 340.529, 340.751, 340.891, 341.059, 342.504, and 345.275. The crack widths were measured at 1-2 month intervals. During the spring of 2001, core holes were drilled at each end of the cracks in an attempt to prevent further progression. However, several months later, at some locations, the cracks were observed to have propagated several feet beyond the core holes. The crack propagation beyond the core holes was mainly in the direction of traffic (to the west) and mostly at locations where substantial differential settlement had occurred.


FIGURE 4. Installation of Stitches on I-70 at Agate

During October of 2002, the Region 1 Limon Residency arranged to cross-stitch the cracks between MP 346 and 340. At 18-inch intervals, on alternate sides of the crack, holes were drilled through the concrete at about a 45º angle to the surface of the pavement so they crossed the crack near the middle of the concrete depth **(Figure 4)**. Twenty-inch-long pieces of 5/8-inch steel rebar were coated with epoxy cement and driven through the concrete into the base. The ends of the bar were driven below the surface of the pavement and the holes sealed with epoxy cement. The performance of the cross stitches is being monitored.

During October 2001, Region 1 Materials, with the help of its drilling crew and CDOT maintenance personnel from Limon, extracted numerous cores between mileposts 340.0 and 348.0. The cores were extracted on and next to longitudinal cracks believed to have been caused by a malfunctioning vibrator with the intention of investigating the concrete paste and the orientation of the aggregate particles in comparison with cores taken from adjacent locations where cracks had not developed. Cores were also taken from the adjoining shoulder slabs.

Undisturbed soil samples were taken in the Agate area where the concrete pavement had undergone differential settlement. The soil samples were sent to the CDOT Central Laboratory for testing. These test results confirmed the results of the previous soil survey and laboratory tests. The soil plasticity and the swelling potential are high (PI = 30-39) and fraction passing No. 200 sieve is very close to 100 percent.

Soils having that much swelling potential are very sensitive to moisture change and can undergo substantial movement with moisture penetration.

Cracks help surface water penetrate into the subgrade natural soils where it becomes a source for soil swelling and freeze-thaw action resulting in slab movement. That movement promotes cracking begun due to other types of distresses. Region 1 Maintenance sealed the cracks to help prevent further water penetration into the subgrade.

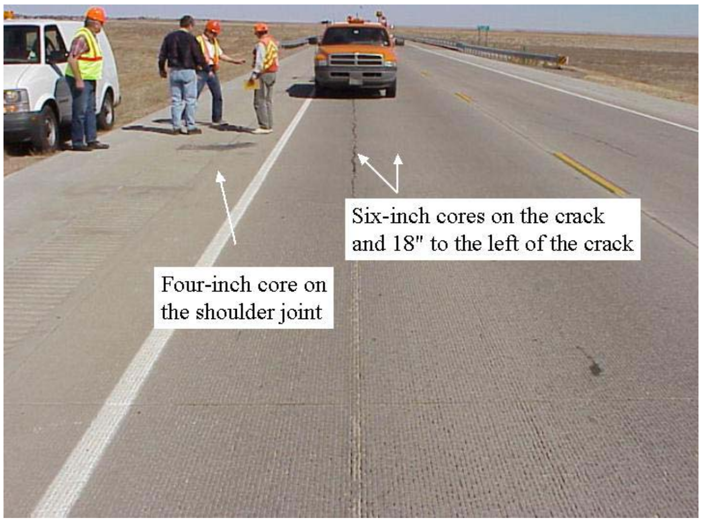

FIGURE 5. Core Locations on the Vibrator Trail—US-287 Southbound

Investigation of US-287 at Kit Carson

US-287, south of Kit Carson, was a 9-mile-long project built in 1998. The two-lane concrete pavement has 14-foot-wide slabs and is 9 inches thick with the shoulder thickness tapered from 9 inches at the shoulder joint to 5 inches at the outside edge of the pavement. At the approaches to structures the shoulders are full depth. Both lanes were paved monolithically from north to south.

The longitudinal cracks on US-287 were assumed to be due to shallow saw-cut depth at the shoulder joint or to vibrator trails created by malfunctioning paver vibrators. Both factors were found to have contributed to the cracking. It should be noted that a total of only 0.30 percent of the slabs in the 9-mile-long project were cracked.

FIGURE 6. Cores from US-287 Southbound

A set of three core samples was taken at each of three different locations southbound through the project. A set consisted of a six-inch core on the longitudinal crack, a six-inch core about 18 inches to the left of the first core, and a four-inch core on the shoulder joint **(Figure 5)**. The six-inch cores were sent to Construction Technology Laboratories (CTL) for petrographic analysis; the four-inch cores were used to determine if the shoulder joint had cracked through, as it should **(Figure 6)**.

The results of CTL's testing showed the air content of the six-inch cores taken on the vibrator trails to be consistently lower than the air content of the cores taken from the center of the lane. This proved that vibrator operation at too high frequency or prolonged vibration at low paver speed was the main cause of the vibrator trails.

Examination of the four-inch cores taken on the shoulder joint revealed that the saw-cuts were not deep enough (less than 0.4 D for 14-foot slabs) to instigate a weakened plane. As a result, longitudinal cracks developed in the travel lanes to release the strength gain of the concrete.

An overactive vibrator apparently created a vibrator trail, which caused a weakened plane in the concrete. In some areas, longitudinal cracks developed along the vibrator trail even where the shoulder joint was working, i.e., the saw-cut was at a proper depth and the joint had cracked through.

FIGURE 7. Wandering Cracks in the Northbound Lane of US-287

Split tensile strength tests were performed on 6-inch-diameter cores extracted from the vibrator trail where the tining tracks were deeper, but no longitudinal crack was evident – see the foreground of Figure 5. Split tensile strength tests were also conducted on cores taken next to the vibrator trails **(Figure 6)**. The main purpose was to determine if the cores taken from the vibrator trials showed lower strength than cores taken next to them. This investigation was inconclusive. The results are shown in Table 1.

The southbound cracks occurred primarily in the vibrator trail and are a consistent distance from the edge of the pavement **(Figure 5)**. In contrast, while most of the longitudinal cracks in the northbound lanes are in the right wheel path, they wander from side to side **(Figure 7)**.

TABLE 1. Split Tensile Strength Comparison (psi), US-287

Vibrator Trail Path	Between the Trails
585	630
650	645
670	535
550	570
614 (Average)	595 (Average)

FIGURE 8. Longitudinal Cracks on 12' Wide Slabs—I-70 at Deer Trail

FIGURE 9. Cores from the Nonworking Shoulder Joint on I-70 at Deer Trail

Investigation of I-70 at Deer Trail

In April of 2002, CDOT's Research Branch, with the help of Maintenance Patrol 16 from Deer Trail, extracted 4 cores from the longitudinal shoulder joint on I-70 eastbound near Deer Trail between Mileposts 326-327 **(Figure 8)**. The slab width in the driving lane, unlike those investigated previously on US-287 (14-foot slabs), was standard 12 feet. The longitudinal cracks at this location consistently began at the shoulder joint and wandered diagonally through the wheel path to about mid-slab. Cracks were also noted on the shoulder parallel to the joint. I-70 at this location was built in 1987 with lower cement content (565 lb. total cementitious, including 455 lb. of cement and 110 lb. of fly ash) and with skewed transverse joints on 18' spacing.

As shown in **Figure 9**, none of the saw-cuts at the shoulder joint were working. The pavement depth was 10.25 inches; the saw-cut measured between 2 and 2.25 inches (35 percent shallower than D/3). This proves, once more, that the longitudinal cracks are due to shallow saw-cut depth at the shoulder joint.

CONCLUSIONS AND RECOMENDATIONS

The conclusions and recommendations presented here are based on the results of visual observations and field and laboratory investigations of several concrete pavements in Region 1 of CDOT. CDOT, ACPA and the concrete paving industry conducted this study as a joint effort.

Conclusions

- Untreated native soil with high swelling potential i.e., high plasticity index (PI), lower resistance to lateral movement (lower R-value) and poor compaction were identified as two main contributing factors in the development of premature longitudinal cracking. These types of distresses manifest themselves in the form of longitudinal cracks with differential settlements.
- The majority of the longitudinal cracks were attributed to a shallow saw-cut at the shoulder joints. In some cases the saw-cuts were only 50 percent of the recommended depth. CDOT requires D/3 for standard 12-foot lanes and 0.4D for wider slabs (0.4D mimics the European specification for concrete pavements wider than 12 feet).
- This study found that the 14-foot-wide slab design was not among the causes of longitudinal cracking. CDOT adopted the14-foot slab design in 1996 based on the results of the LTPP SPS-2 experiment, and a supplemental study conducted by Dr. Michael Darter of ERES. The wider slab design is highly recommended for rural highways.
- This study also proved that malfunctioning or improperly adjusted paver vibrators could promote premature longitudinal cracking. Vibrators working at too high frequencies over-consolidate the concrete mix causing non-uniform dispersion of the aggregate and forming vibrator trails. Laboratory investigation of cores extracted on and adjacent to the vibrator trails clearly confirmed this phenomenon. The cores obtained on the vibrator trails had consistently lower air content than cores obtained 18 inches away - in the center of the lane.

Recommendations

- Premature longitudinal cracking due to improper depth of the longitudinal saw-cut can be prevented by adhering to CDOT's present specification, which requires saw-cut depth of D/3 for 12-foot slabs and 0.4D for 14-foot slabs. It is recommended that project engineers in the field measure saw-cut depth at intervals of 1 per 1/10 of a mile (528 feet). This specification has already been incorporated into CDOT's *Field Materials Manual*.
- It is strongly recommended that the paving contractors equip their paving machines with frequency monitoring devices for the vibrators. These monitoring devices provide the transportation agencies and the paving contractor a necessary tool to help achieve a quality concrete pavement that is long-lasting. CDOT recommends following Iowa DOT's specification for monitoring the frequencies of the vibrators. CDOT is in the process of fine-tuning Iowa's specification and plans to implement the results in the near future.
- Accurately recognizing and predicting the potential volume change of expansive soils and their treatment prior to construction plays a major role in overall longevity and performance of pavements. It is necessary to alleviate or eliminate the detrimental effects of expansive soils.

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Roadway Conditions as Contributing Factors in Florida Traffic Crashes

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ABSTRACT

One of the primary goals of the Florida Department of Transportation (Florida DOT) is to reduce the number and severity of crashes that occur on the State Highway System (SHS) each year, or that portion of Florida's roads under the Florida DOT's control. The Florida DOT's main goal is to keep the percentage of crashes on the SHS where roadway conditions are contributing factors below one percent through the year 2006. As defined by the Florida Department of Highway Safety and Motor Vehicles, roadway conditions include such factors as standing water; loose surface materials; holes, ruts, and unsafe paved edges; and worn or polished roadway surfaces, for example.

The findings from the research indicated that the use of roadway conditions as a performance indicator is not warranted. The estimate of the number of 1998 crashes on the SHS where a roadway condition was shown to be a contributory factor to its occurrence was very low. Out of the final sample crashes, only a total of 204 crashes indicated a roadway condition that could be potentially correctable by a Florida DOT action. The net result was a total number of crashes that was so minimal that the determination of a significant problem cannot be made where corrective action by the Florida DOT could minimize or reduce the number of future crashes. Projecting the sample to all of the 1998 traffic crashes that occurred on the SHS in Florida, only 0.7 percent of could have potentially been influenced by Florida DOT corrective actions.

Key words: crash data—crash report—roadway conditions—sample of crashes—traffic control

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INTRODUCTION

One of the primary goals of the Florida Department of Transportation (Florida DOT) in its *Florida Transportation Plan* is to reduce the number and severity of crashes that occur on the State Highway System (SHS), or that portion of Florida's roads under Florida DOT control. As stated in its *1999 Annual Report*, the Florida DOT's main goal is to keep the percentage of crashes on the SHS where roadway conditions are contributing factors below one percent through the year 2006.

The primary purpose of conducting this special research project for the Florida DOT was to determine whether the percent of 1998 Florida traffic crashes noting a roadway contributing condition could serve as a reliable Florida DOT performance indicator for measuring safety on the SHS. As defined by the Department of Highway Safety and Motor Vehicles (DHSMV), roadway conditions include such factors as standing water; loose surface materials; holes, ruts, and unsafe paved edges; and worn or polished roadway surfaces, for example. It should be noted that the investigation of conditions or defects caused by the design of roadways that are part of the SHS was not included as an element of the research project.

The Florida DOT requested the assistance of the Center for Urban Transportation Research (CUTR) at the University of South Florida, Tampa, to complete this project using 1998 Florida traffic crash data, the most recent available at the time. This paper presents the findings and recommendations from the fourth study.

METHODOLOGY

Data collection involved several simple steps. First, Florida traffic crash data for calendar year 1998 were requested from the DHSMV from its Statewide Accident Management Information System (STAMIS) electronic crash database. In the previous three studies, electronic traffic crash data were obtained from the SSO and not the DHSMV (with a few minor differences, the crash data are identical) (*1, 2, 3*). Second, using specific parameters, a sample of applicable crashes (those that occurred on the SHS with a roadway condition code other than Code-01 or no defects) was developed using the random sample generator in SPSS 10.0. The final step in the collection of crash data involved obtaining hard copies of the crash reports from the SSO.

By selecting certain information contained in specific crash variables, the research team was able to create a database that contained only those crashes that occurred on the SHS with a noted roadway condition numerical code value other than 01 (no defects). The variable "road system identifier" in the DHSMV crash database contained the information about roadway types. This particular variable lists seven roadway types (numerically coded as 01 through 07) and Code-77, which represents "all other" roadway identifiers not listed among the first seven such as forest and fire roads. For this variable, SHS roadways were numerically coded as follows: 01 (interstate), 02 (US), 03 (state), and 06 (tolls/turnpikes). Again, the SHS is comprised of those roadways that are included in the Florida DOT's Roadway Characteristics Inventory and are under its direct purview. This procedure resulted in a database that included a total of 3,077 crashes that occurred on the SHS with a noted roadway condition code other than 01 (no defects).

Using the crash report number as the key, information from each of the four DHSMV sub-files (Events, Driver, DOT Site, and Vehicle) were merged into a final crash database that contained the following data elements (for reference, the corresponding STAMIS sub-file is listed next to each data element):

- County (Events File)
- City (Events File)
- Crash report number (Events File)

- Date (Events File)
- Injury severity (Events File)
- Total property damage (Events File)
- Number of lanes (Events File)
- First harmful event (Events File)
- Road system identifier (Events File)
- Lighting condition (Events File)
- Road surface condition (Events File)
- Weather (Events File)
- Road surface type (Events File)
- First contributing cause road (Events File)
- First traffic control (Events File)
- DOT milepost (DOT Site Location File)
- DOT average daily traffic (DOT Site Location File)
- DOT highway location (DOT Site Location File)
- State road number (DOT Site Location File)
- Driver injury severity (Driver File)
- Driver age (Driver File)
- Department District (derived based on the variable "County" from the Events File)

The next step was to develop a sample of traffic crashes from the database that contained the 3,077 crashes that occurred on the SHS with a noted roadway condition code other than 01 (no defects). The initial sample of crashes were ultimately developed by selecting approximately 25 percent of the individual crash records within each Department district using the simple random sample generator available in SPSS 10.0. This approach ensured that each Department district would be represented proportionally in the final sample of traffic crashes. This process resulted in an initial study sample of 900 crashes out of the possible 3,077. For comparison, the size of the final samples for the three previous studies varied from 50 percent (1995), 25 percent (1996), and 15 percent (1997) of the crashes that met the criteria for sampling. Based on the results from the assignment of detailed causality codes, the crash reports that were actually coded as 01 (no defects) but transcribed incorrectly by DHSMV data entry staff, plus others that were either missing the crash narrative and/or diagram or had other incomplete information that prohibited an accurate application of one of the detailed causality codes were eliminated from the initial sample (n = 900). After these eliminations, the remaining number of crash reports in the final sample was 728.

Unfortunately, the computerized crash data obtained from the DHSMV does not contain the crash report narrative and diagram. The crash narrative and diagram were critical to properly identifying the quality and accuracy of the crash data as well as any cause-and-effect relationships that may have existed in the data. Therefore, actual crash reports archived at the SSO needed to be obtained and reviewed in detail by the CUTR research team.

Once copies of the crash records were in hand, their analysis took several steps. The first step involved the classification of roadway conditions noted on each crash report. This required a researcher to study each crash report intensively by carefully reading the narrative and examining the diagram to identify the role, if any, of the listed road-related contributing factors. If cause was determined, a code was applied to each crash report and entered into the crash database that contained the initial sample of crashes.

Since the most critical information was contained in the narrative and diagram portion of the crash reports, a set of detailed codes was used to categorize roadway conditions. Initially developed in conjunction with Florida DOT staff, these detailed codes were created to show more specific information about the contribution that roadway conditions may have played in crashes.

The first step when analyzing the crash report narratives and diagrams was to make an assessment of whether each crash report was consistent with the roadway condition code originally assigned by the reporting law enforcement officer. The next step was to determine whether the reporting law enforcement officer had described the effect of the indicated roadway condition in the crash report narrative and/or diagram. This was the most critical and time consuming step in the process of analyzing the initial sample of crash reports. Based on the methodology devised for use in the three prior studies, the initial sample of crash reports was first classified according to whether any mention or reference to the roadway condition was made in the crash report narrative and/or diagram by the reporting law enforcement officer. It was envisioned that the roadway conditions could be clearly identified and their contribution explained in the narrative and/or in the diagram of the crash reports. If no reference was found to the indicated roadway condition in either the crash report narratives or diagrams, the decision was that the listed roadway condition played no contributing role in the occurrence of the crashes but instead was merely present at the time of the crashes. For example, if a law enforcement officer indicated loose surface materials (roadway condition Code-04) but failed to mention anything specific about the nature and involvement of the loose surface materials in the crash report narrative and/or diagram, it was concluded that the reporting law enforcement officer did not consider the loose surface materials to have been a contributing factor in the occurrence of the crash (i.e., treated as a no-defect crash). Based on the detailed causality codes, these particular reports were assigned one of the following numerical codes: 299, 399, 499, 599, 699, 799, or 899. These particular detailed causality codes indicate that the reporting officers on the crash reports did not note the roadway condition in either the narratives and/or diagrams. The remainder of the crash reports were reviewed and assigned a corresponding detailed causality code. In a few instances, crash reports may have also received a second detailed causality code; particular in those instances where it was determined that an additional roadway condition may have also played a role in the occurrence of the crashes.

The last step was to identify those crashes that contained a potentially correctable roadway condition that the Florida DOT could possibly use when devising strategies to eliminate or reduce future hazardous roadway conditions. The crash reports that were coded as potentially correctable using the set of detailed codes were determined to be the best source for developing a roadway condition performance indicator. Again, this determination was based on the need for the performance indicator to determine the number of crashes that may actually be avoided or reduced through some type of corrective action by the Florida DOT via its roadway maintenance program. This means that the identified roadway conditions that are not directly correctable by an action of the Florida DOT such an environmental obstruction or a severe weather condition were eliminated from consideration during assessment. After a detailed cause code was applied to each crash report (assuming cause was determined), the code was entered into the crash database for analysis to identify any possible trends and correlations that may have existed within the crashes.

RESULTS

As mentioned, one of the primary goals of the project being reported on in this paper was to determine whether the percent of crashes noting a roadway condition could serve as a reliable agency performance indicator for highway safety. As in previous studies, the decision was once again to be based on whether one or more cause-and-effect relationships could be established within the traffic crash data. The analysis of 1998 Florida traffic crash data shows that no one such relationship could be established. Thus, there was no strategic issue present with regard to roadway conditions as contributing factors in Florida traffic crashes. In addition, for numerous reasons such as the low quality of the crash data mainly due to inaccurate application of the actual roadway condition by officers, the use of this data element in its current form is unsuitable as a measure for the Florida DOT to use when gauging its safety performance with regard to the SHS.

Several findings were uncovered related to the accuracy and validity of the 1998 Florida traffic crash data. Detailed investigation of the final sample of crashes shows a very high degree of coding agreement between the data entries in the electronic DHSMV crash database (which was assumed to be correct by users) for the roadway conditions data element and the entries actually noted by law enforcement officers on the written crash reports acquired from the SSO.

Detailed inspection indicated that only 1.5 percent of the crash reports in the final sample obtained from the SSO were found to have been coded incorrectly by DHSMV data entry staff when transcribing crash information into electronic format. In eight of the instances the hard copy crash reports were actually coded as 01 (no defects) by law enforcement officers but were mistakenly transcribed by DHSMV data entry staff as one of the other possible eight contributing cause codes (Codes 02-77). For example, in two instances, crash reports that were coded as 01 (no defects) by officers were listed in the DHSMV electronic database as Code-02 (obstruction with/without warning).

The detailed inspection of the hard copy crash reports also indicated that Code-77 (all other, explain) represents a special case with respect to the quality and accuracy of the transcribed crash data in the DHSMV electronic database. In 5.2 percent of the final sample it was revealed that DHSMV data entry staff indicated Code-77 (all other, explain) as the roadway condition when an officer did not indicate one of the possible roadway contributing cause codes (i.e., he/she either unintentionally or intentionally left the box entirely blank, wrote unknown, or crossed through the check box) on the crash report. When transcribing crash data, it appears that DHSMV data entry staff did not attempt to make a determination of the roadway contributing cause in these cases using the crash report narratives and/or diagrams and, instead, indiscriminately applied Code-77 (all other, explain). Based on the information provided in the crash report narratives and diagrams in each of these instances it was determined by the research team that Code-01 (no defects) should have originally been applied to these particular crash reports by law enforcement officers instead of leaving the box blank. It should be noted, however, that the research team believes these instances to actually be the result of DHSMV data entry procedure rather than an issue of data quality or accuracy. The degree of coding agreement by roadway condition factor is shown in Table 1.

The findings also indicated that the use of roadway conditions as a performance indicator was not warranted. The estimate of the number of 1998 crashes on the SHS where a roadway condition was shown to be a contributory factor to its occurrence was very low. Out of the final sample crashes, only a total of 204 indicated a roadway condition that could be potentially correctable by a Florida DOT action, as shown in Table 2. The net result was a total number of crashes that was so minimal that the determination of a significant problem cannot be made where corrective action by the Florida DOT could minimize or reduce the number of future crashes. Projecting the sample to all of the 1998 traffic crashes that occurred on the SHS in Florida, only 0.7 percent of could have potentially been influenced by Florida DOT corrective actions. This finding meets the Florida DOT's goal of less than one percent of the crashes on the SHS in which a roadway condition was a contributing factor.

The analysis of the final sample of 1998 traffic crash reports showed that the roadway codes utilized by law enforcement was not well documented in the narrative and/or diagram portion of the crash reports contained in the final sample. This results in an overestimation of road-related contributing condition crashes and/or the absence of useful data with which t address any actual contributions of the roadway conditions in crashes.

TABLE 1. Coding Inconsistencies by Contributing Roadway Condition Factor (1998)

Roadway Contributing Condition		Number in Sample[1]	Percent of Sample	Number of Inconsistencies	Percent Inconsistency by Factor
Code	**Description**				
02	Obstruction with or without warning	65	8.9%	4	6.2%
03	Road under repair or construction	287	39.4%	1	0.4%
04	Loose surface materials	30	4.1%	4	13.3%
05	Shoulders soft, low, or high	20	2.7%	1	5.0%
06	Holes, ruts, or unsafe paved edge	22	3.0%	0	0.0%
07	Standing water	155	21.3%	1	0.7%
08	Worn or polished road surface	65	8.9%	0	0.0%
77	All other, explain	84	11.5%	38	70.2%
Total		728	100%	49	6.7%

[1] The crash reports that were actually coded as 01 (no defects), plus the other crash reports that were missing the narrative and/or diagram or had other incomplete information that prohibited an accurate determination of the detailed roadway contributing cause were eliminated from the initial sample of 900 crashes. The final sample after these eliminations had 728 crashes.

Many law enforcement officers are simply entering one of the possible roadway conditions only when one of the conditions or factors was present instead of when it was observed to have directly contributed to the occurrence of the crash. Based on the review of the crash reports in the final sample, in many of these instances the reporting law enforcement officers should have indicated Code-01 or no roadway conditions/defects. It also appears that officers made no attempt whatsoever to determine what, if any, roadway condition(s) might have lead to the occurrence of the crashes contained in the final sample. For example, the application of the detailed causality codes revealed that most of the crashes that occurred in a construction zone (Code-03) were coded as such without any thought or regard to whether there was something specific about the construction zone that contributed to the occurrence of the crash such as a ladder suddenly falling off of a construction vehicle and causing an unexpected roadway obstruction.

Detailed investigation of the final sample of crash reports shows a very high degree of coding agreement between the data entries in the electronic DHSMV crash database (which was assumed to be correct by users of the crash data) for the variable “first contributing cause-road” and the entries actually noted by law enforcement officers on the written crash reports. DHSMV data entry staff when transcribing crash information into electronic format coded only 1.5 percent of the crash reports in the final sample incorrectly.

In 5.2 percent of the final sample, it was revealed that DHSMV data entry staff indicated Code-77 (all other, explain) as the roadway condition when an officer did not indicate one of the possible roadway conditions (i.e., he/she either unintentionally or intentionally left the box entirely blank, wrote unknown, or crossed through the check box) on the crash report. Code-77 implies that a roadway condition was present that potentially contributed to the occurrence of the crash. The dilemma created by a DHSMV data entry operator coding a blank response as Code-77 is the inability to determine if a roadway condition was actually present or if the officer really left the roadway condition code box blank.

TABLE 2. Crash Reports Identifying Road Conditions Potentially Correctable by the Florida DOT by District (1998)

Code	Detailed Causality Code	Number of Potentially Correctable Crashes by District								Total
		1	2	3	4	5	6	7	8	
Category 10 (Miscoded Reports)										
120	Blank/Unknown, Most Like 07	1							1	2
Category 20 (Obstruction with/without Warning)										
22	Barricades Without Warning		1							1
26	Stopped Vehicle Due to Construction					1				1
Category 30 (Road Under Repair/Construction)										
31	Construction Worker Struck							1		1
32	Struck Construction Vehicle/Barricades	3	6		2	6		1	1	19
36	Construction Related Confusion/Unfamiliarity	1	1	1	2		1	4		10
37	Road Surface Condition Due to Construction	2	2			2	4			10
38	Impacted by Construction Related Vehicle			1	2				1	4
311	Traffic Control Restriction Due to Construction		2					1		3
312	Flagman Present	1	3			1	1		1	7
313	Improper Traffic Control in Construction Area			1		1	1			3
314	Obstruction Related to Construction	1		1	1					3
Category 50 (Shoulder Soft/Low/High)										
51	Shoulder Soft	1		2	2	1			1	7
52	Shoulder Low		1		1					2
53	Shoulder High		1							1
Category 60 (Holes/Ruts/Unsafe Paved Edges)										
61	Holes in Pavement	2	2	2		3	2			11
62	Ruts in Pavement					1				1
63	Unsafe Paved Edge Present	2								2
Category 70 (Standing Water)										
71	Flooding Due to Poor Drainage	1						1		2
73	Excessive Water on Road/Standing Water, Not Defined	15	8	4	7	4	7	8	1	54
75	Isolated Water on Road	1		1	2	1	1	1		7
77	Hydroplaning Occurrence, Indicating Standing Water	3	15	4	4	1	3	2	1	33
Category 80 (Worn/Polished Road Surface)										
81	Fault in Road, Polished	1	1			1				3
84	Metal Bridge Grating		2							2
Category 90 (All Other)										
92	Most Like 02	1			1	2	2	1		7
95	Most Like 05	1								1
96	Most Like 06					2				2
97	Most Like 07		1	1						2
911	Traffic Control Malfunction	1	1		1					3
Total		38	47	18	25	27	22	20	7	204

As noted above, officers often elect not to enter one of the nine possible roadway condition codes but to either leave blank, write “unknown”, or cross through the check box on the crash report for this data element even though the crash report completion procedure requires that a code entry be made in all instances. In addition, if officers feel that one of the specific roadway condition codes (01-08) does not sufficiently provide an accurate description of the circumstances that lead to the crashes, they have the option of selecting Code-77 (all other, explain), which requires further explanation of the circumstances surrounding the crashes in the narrative and diagram portion of the reports. The guidelines contained in the *Instructions for Completing the Florida Uniform Traffic Crash Report Forms* does not instruct law enforcement officers to explain the codes they chose in the narrative or diagram of crash reports other than Code-77 (all other, explain). Officers are simply required to describe what happened in the crash and to make the narrative and diagram as consistent as possible. As noted in prior studies on this issue, the resulting inconsistency of reporting produces an overstatement of the number of crashes on the SHS that involve a roadway condition.

The various cause-and-effect analyses revealed by the crash reports does not appear to show any significant problems either by District or Department maintenance facility (County). While Florida DOT actions can be taken, this finding results in the conclusion that there was no specific strategic action that can be taken by the Florida DOT to reduce the observed number of roadway condition crashes on the SHS.

While the sample included a small number of crashes that occurred on the Florida Turnpike (District Eight), construction-related and standing water accounted for the bulk of these types of crashes in the final sample. It was recommended that the Florida Turnpike staff be made aware of this finding and that they should monitor and further investigate these types of crashes to reduce occurrence since these are areas where the Florida DOT could potentially make the biggest impact via its maintenance program.

Based on the review of the crash reports in the final sample, standing water often appears to be the cause of inadequate drainage or water runoff on some of the roadways that comprise the SHS. In many cases this was a problem that can readily be corrected by Florida DOT action. Of the final sample of 728 crashes, 96 involved standing water as a direct result of poor drainage (detailed cause codes 71, 73, 75, and 77). Each of the eight Districts’ maintenance departments need to be immediately informed of these problems as they are reported so that any necessary action can be taken to reduce the number the number of standing-water related crashes.

During the course of this study, it was uncovered that SSO staff included crashes in its crash database that actually occurred on non-SHS roadways or what the SSO refers to as side-road crashes. SSO staff included those crashes that occurred on a non-SHS roadway within 250 feet of an intersecting SHS roadway even when the roadway conditions on the intersecting SHS roadway played no contributory role in the crash. This resulted in an overstatement of roadway condition crashes and/or the absence of useful data to be able to address any true contributions of roadway conditions in crashes on the SHS.

RECOMMENDATIONS

Based on the findings, it was recommended that the Florida DOT *discontinue* the use of the roadway conditions data element as a performance measure to improve motorist safety on the SHS. Instead, the Florida DOT should attempt to find another method as a performance indicator for measuring its safety performance regarding the SHS.

Further, if Florida DOT management continues with the use of this data element as a performance measure, improvements must be made in the training of law enforcement officers statewide on the correct application and use of this particular data element. If accurately applied by law enforcement officers when investigating the scene of crashes and subsequently completing crash reports, this data element would be the most appropriate measure for gauging the Florida DOT's performance with respect to motorist safety on the SHS. Previous studies recommended that 15-minute training sessions be prepared and implemented as part of the overall law enforcement-training program. It was recommended that these sessions be continued and the frequency that they are administered be increased with specific emphasis given to officers about the critical need for them to correctly and uniformly apply this data element. In addition, it was recommended that the procedure that officers are trained to follow be changed so that they are mandated to indicate whether or not and how roadway conditions may have contributed to crashes that occurred on the SHS.

Compared to other roadway conditions, due to the high frequency of crashes caused by the presence of roadway repairs/construction and standing water, it was recommended that all crash reports with one of these noted roadway contributing factors be immediately forwarded by local law enforcement to the appropriate Department/District maintenance office or staff as deemed appropriate for further investigation and appropriate corrective action.

Finally, should the Florida DOT continue with these special studies, it was recommended that a departure be made from using crash data provided by the SSO to data provided by the DHSMV. Based on the inclusion of side-road crashes, the information contained in the DHSMV database is a more accurate portrayal of crashes that actually occurred on Florida DOT maintained roads, i.e., the SHS. As a result of the inclusion of side-road crashes, the use of the SSO crash database overestimated the number of crashes that could potentially be correctable by the Florida DOT.

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Statistical Estimation of the Importance Customers Place on Specific Elements of Bus Rapid Transit

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ABSTRACT

Bus rapid transit (BRT) is a rapidly growing national trend in the provision of public transportation. At present, with more than 150 new start rail projects currently in the Federal Transit Authority pipeline, a wide range of alternatives is necessary to fulfill the demand for cost-effective rapid public transportation. As a lower-cost, high-capacity mode of public transportation, BRT can serve as an option to help address the growing traffic congestion and mobility problems in both urban and non-urban areas. Careful documentation and analyses of BRT systems and the unique features of these projects will help determine the what are the most effective features offered by the BRT systems such the most successful service characteristics, level of transit demand, region size, and other amenities. This paper presents a statistical analysis of the data from two on-board customer surveys conducted in 2001 of the BRT systems in Miami and Orlando, Florida. Using data from the two on-board surveys, the simplest method for measuring the importance that customers place on specific BRT service characteristics is to calculate mean scores for each characteristic using some type of numeric scale (for example, a scale of 1 through 5, with 5 being the highest). While there are no real discernable drawbacks to this simple method, an alternate technique to measure the importance of each service attribute is to derive the importance of each attribute using STEPWISE regression. This statistical method estimates the importance of each attribute to overall customer satisfaction. The results indicate that customers place a high value on the BRT service characteristics frequency of service, comfort, travel time, and reliability of service.

Key words: bus rapid transit—customers—public transportation—service characteristics—transit demand

INTRODUCTION

One of the main goals of the Federal Transit Administration's (FTA) Bus Rapid Transit (BRT) Demonstration Program is to determine the effects of 10 nationwide BRT demonstration projects through a scientific evaluation process. The FTA designated the South Miami-Dade Busway, or Busway for short, as one of its 10 BRT demonstration sites. While not one of FTA's ten designated BRT demonstration projects, the Lynx LYMMO in Orlando was chosen by the FTA for evaluation due to its intelligent transportation systems (ITS) and as a model for the implementation of similar BRT systems. According to the FTA, careful documentation and analyses of the BRT demonstration projects and the unique features of these projects will help determine the most effective features, i.e., type of service offered, most successful service characteristics, level of transit demand, region size, and other amenities. It is anticipated that the BRT demonstration projects will serve as learning tools and as models for other locales throughout the country, and possibly the world. In order for these demonstrations to have maximum effectiveness in their respective operational capacities, a consistent and carefully structured approach to project evaluation is necessary.

The following, taken verbatim from *Evaluation Guidelines for Bus Rapid Transit Demonstration Projects* (*1*), are the four evaluation guidelines for the 10 BRT demonstration projects:

1. Determine the benefits, costs, and other impacts of individual BRT features, including ITS/APTS applications, and of the system as a whole.
2. Characterize successful and unsuccessful aspects of the demonstration.
3. Evaluate the demonstration's achievement of FTA and agency goals.
4. Assess the applicability of the demonstration results to other sites.

In addition, the FTA plans to examine specific impacts of the BRT demonstration projects. These impacts include: degree that bus speeds and schedule adherence improve; degree that ridership increases (due to improved bus speeds, schedule adherence, and convenience); effect of BRT on other traffic; effect of each of the BRT components on bus speed and other traffic; benefits of ITS/APTS applications to the demonstration project; and effect of BRT on land use and development. To meet these objectives, it is necessary to collect a variety of data on several aspects of the BRT demonstration project, including measurable impacts to BRT customers via the on-board survey process.

In keeping with the FTA's evaluation guidelines, the National Bus Rapid Transit Institute at the Center for Urban Transportation Research (CUTR), working jointly with Miami-Dade Transit (MDT) and Lynx, conducted on-board surveys of South Miami-Dade Busway customers in March 2001 and Lynx LYMMO customers in December 2001. The South Miami-Dade Busway and Lynx LYMMO are examples of different applications of BRT systems that are specifically designed to offer faster travel choices to customers compared to standard local bus service and possibly, even the private automobile. Evaluation of the various components of the Busway and LYMMO are crucial parts of the demonstration project. The two on-board surveys serve as the first phase of the independent review of the Busway and LYMMO BRT systems. The second phase will include analyses of the more detailed components of each BRT system, including engineering and construction, technical documentation, ITS, and system performance, for example.

The on-board surveys were conducted to assess customer perceptions, behavior, and to develop customer profiles. The survey instruments asked customers to evaluate the various BRT elements of service as well as overall satisfaction, with the ultimate purpose of measuring the impacts of the systems on customer perceptions. Other questions focused on customer behavior, including trip origins and destinations and frequency of use.

OBJECTIVE

This paper presents a statistical analysis of the data from two on-board customer surveys of the BRT systems in Miami and Orlando, Florida. Using data from the two on-board surveys, the simplest method for measuring the importance that customers place on specific BRT service characteristics is to calculate mean scores for each characteristic using some type of numeric scale (for example, a scale of 1 through 5, with 5 being the highest). While there are no real discernable drawbacks to this simple averaging method, an alternate technique to measure the importance of each service attribute is to derive the importance of each attribute to overall satisfaction using more advanced statistical procedures such as STEPWISE regression. This statistical method estimates the importance of each service attribute to overall customer satisfaction. While there may be a degree of inter-correlation between each of the service attributes, this method can be used to measure the relative importance of each attribute when determining what elements or combination of elements best comprise overall customer satisfaction with these two BRT systems.

ABOUT THE SOUTH MIAMI-DADE BUSWAY AND LYNX LYMMO BRT SYSTEMS

South Miami-Dade Busway

The South Miami-Dade Busway or Busway, for short, is an eight-mile two-lane bus-only roadway constructed in a former rail right-of-way (the former Florida East Coast Railroad corridor) adjacent to U.S. 1, a major north-south arterial in southern Miami-Dade County. Miami-Dade Transit opened the first phase of the Busway on February 3, 1997. The Busway was designed for exclusive use by transit buses and emergency and security vehicles. The purpose of the Busway service is to address the need for faster travel choices for MDT customers. Much of the Busway BRT service uses 20-seat minibuses to keep costs to a minimum.

Currently, there are 18 intersections and 15 stations in each direction (30 total stations), as shown in Figure 1. The Busway corridor over much of its length is within 100 feet of the west side of U.S. 1, one of the most heavily traveled corridors in Miami-Dade County. There are several types of service in the Busway corridor:

- *Local* – only operates on the exclusive Busway and makes every stop at all times (referred to as the Busway Local).
- *Limited Stop* – operates along the length of the Busway and beyond, skips stops nearest the Metrorail station during peak periods (Busway MAX or Metro Area Express).
- *Feeder* – Collects passengers in neighborhoods and then enters the Busway at a middle point (service is known as either the Coral Reef MAX or Saga Bay MAX).
- *Crosstown* – These were pre-existing routes in the corridor that now take advantage of the Busway when possible. These routes enter and exit the Busway at middle points. These routes are designed to provide access to many destinations in the region, not just to the center city (Routes 1, 52, and 65).
- *Intersecting* – Routes in the corridor that intersect with Busway routes, sometimes stopping at Busway stations.

The Busway stations are located at roughly half-mile intervals, more than twice the customary stop spacing for conventional MDT local bus service. For example, when Route 1 operated on U.S. 1, it had 19 designated stops southbound and 23 northbound (on the portion of the route using U.S. 1). When it was moved to the Busway, only 10 Busway stations served the same distance. Most stations are on the far side of intersections. In two locations there are mid-block stops to serve major generators. All stations

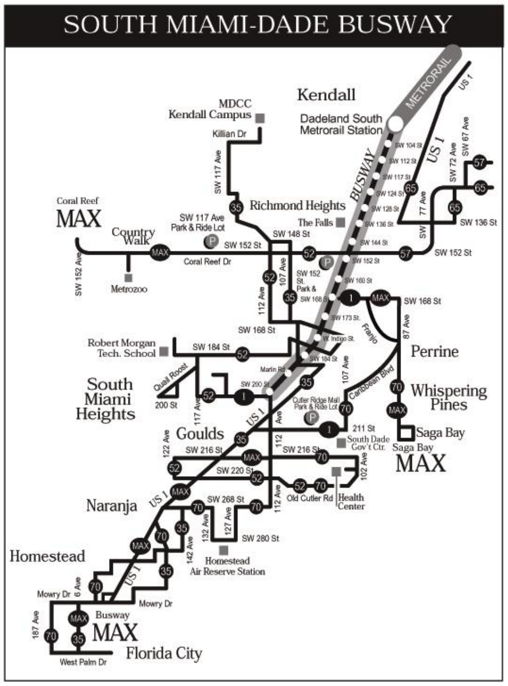

FIGURE 1. Map of the South Miami-Dade Busway System

have large shelters designed to protect customers from the weather. Stations platforms are in three lengths: 40 feet, 60 feet, and 80 feet. Busway vehicles operate parallel in a bi-directional manner with vehicular traffic operating separate from Busway vehicles.

According to MDT, bus ridership on the U.S. 1 corridor in South Miami-Dade County increased greatly with the implementation of the Busway service. As a result of Busway service, ridership in the corridor increased by 49 percent on weekdays, 69 percent on Sundays, and 130 percent on Saturdays since May 1998. MDT staff indicated that the major reasons for the increases in ridership was the increase in service provided, in terms of new areas served, more frequent service, and a greater span of service. Except for Saturdays, revenue miles increased even faster than boardings and operating costs increased at only half the rate of the increase in vehicle revenue miles – due to the use of 20-seat minibuses, which cost the MDTA \$31 to \$35 per hour to operate, significantly less than the \$51 to \$53 per hour it costs to operate full-size buses. The difference in cost is due to fuel and maintenance costs and to the lower wages paid to minibus operators.

LYMMO

The LYMMO BRT system is very different in application from the Busway operated by MDT. It operates on a 3.0-mile continuous loop through downtown Orlando using a combination of the various types of dedicated running ways including median and same-side travel way configurations. The exclusive running ways are paved with distinctive gray-colored pavers to delineate them from general traffic lanes. They are separated from general traffic lanes either with a raised median or a double row of raised reflective ceramic pavement markers embedded in the asphalt.

Because the LYMMO operates in places and directions contrary to other traffic, all bus movements at intersections are controlled by special bus signals. To prevent motorist confusion, these signal heads use lines instead of the standard red, yellow, and green lights. When a LYMMO bus approaches an intersection, an embedded loop detector in the dedicated running way triggers the intersection to allow the bus to proceed either in its own signal phase or at the same time as other traffic is released when no conflicting traffic movements are permitted.

The LYMMO uses 10 low-floor vehicles fueled by environmentally friendly compressed natural gas. The vehicles use high-quality, modern interiors that incorporate an ITS system know as the Transit TV Network. The Transit TV Network provides real-time information such as Downtown events, weather, and fun and trivia to customers. In addition, public-art exteriors are used on the vehicles to enhance the

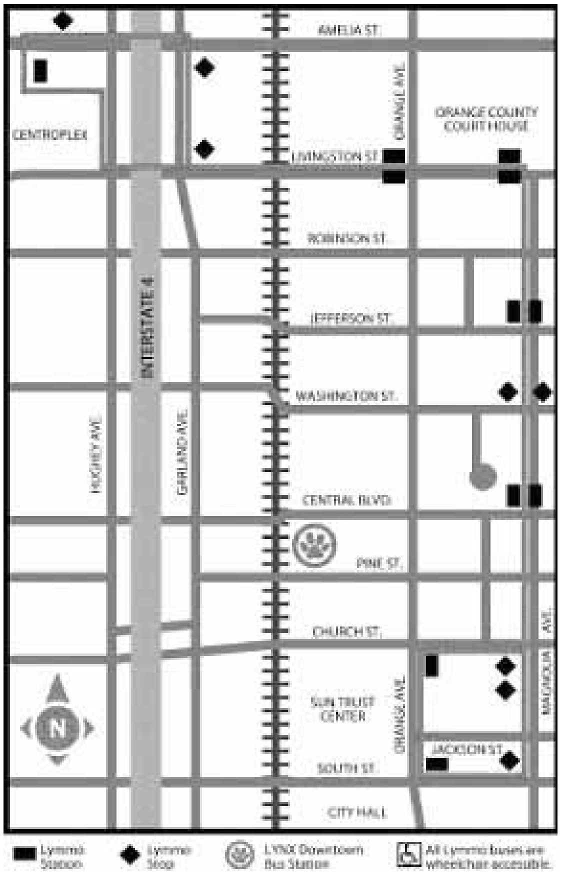

FIGURE 2. Map of the LYMMO System

customer's experience with the LYMMO. The LYMMO system has 11 lighted and computerized stations and 9 additional stops, as shown in Figure 2.

The LYMMO vehicles operate approximately every five minutes during office hours, and after office hours, vehicles operate approximately every 10 minutes. Since the inception of service, the LYMMO has been free to ride during all hours of operation. Operation and maintenance of the LYMMO is 100 percent funded by revenue generated by the City of Orlando's Parking and Enterprise Fund. The LYMMO operates from 6 AM to 10 PM, Monday through Thursday, 6 AM to Midnight on Friday, 10 AM to Midnight on Saturday, and 10 AM to 10 PM on Sunday. The LYMMO's target market is customers who drive to Downtown Orlando and then use LYMMO to get to other Downtown locations, such as the Courthouse, restaurants, shopping, and other land-uses.

For comparison, Table 1 shows the key components of the both the Busway and LYMMO BRT systems.

TABLE 1. Key Bus Rapid Transit Components

Key BRT Attributes	Busway		LYMMO	
	Yes	No	Yes	No
Simple Route Structure	Y		Y	
Frequent Service	Y		Y	
Headway-based Schedules	Y		Y	
Less Frequent Stops	Y		Y	
Level Boarding and Alighting		Y	Y	
Color-Coded Buses		Y		Y
Color-Coded Stations/Stops	Y		Y	
Bus Signal Priority		Y	Y	
Exclusive Lanes	Y		Y	
Modern Vehicle Interiors		Y	Y	
Higher-Capacity Buses		Y		Y
Multiple Door Boarding and Alighting		Y	Y	
Off-Vehicle Fare Payment		Y		Y
Feeder Network	Y			Y
ITS/APTS on Vehicles	Y		Y	
ITS/APTS at Stations		Y	Y	
Coordinated Land-Use Planning	Y		Y	

ON-BOARD SURVEY METHODOLOGY AND STATISTICAL PROCEDURES

The Busway survey instrument was printed in English on one side and Spanish on the other due to the bilingual nature of Miami. It contained 18 questions and provided space for additional written comments by customers. The LYMMO survey instrument was printed in English only and contained a total 13 questions. NBRTI/CUTR and MDT and Lynx staff developed the survey instruments jointly.

The on-board surveys specifically targeted customers riding only those routes that operate along the Busway for either all or a portion of their trips and for all or a portion of their trips in Downtown Orlando on the LYMMO. At least half of all trips on a particular Busway route were selected for surveying. For example, if there were eight trips on a route, four were to be surveyed. If there were nine trips, five were surveyed. The trips selected for survey distribution spanned the service hours, i.e., morning peak, mid-day off-peak, afternoon peak, and evening. For the LYMMO, surveying began at the start of service and concluded at about 7 PM. Given that the typical weekday LYMMO schedule consists of about 186 21-minute round trips (circulations) and the last trip begins at 10 PM, the translates into just over 90 percent of all weekday trip being included in the sample.

Surveyors were instructed to offer a survey form to each customer upon boarding a bus. Every time a customer boarded a Busway or LYMMO vehicle to make a subsequent trip (regardless of the whether the trip was their second, third, fourth, and so on), they were asked to complete another survey. Surveyors were instructed to do the best they could to encourage participation in the survey. If a survey could not be handed directly to a customer, surveyors were instructed to "drop" a survey in each vehicle seat. All surveys were collected on-board buses. No mail back provision was provided for returning the completed surveys.

Once collected, survey data were entered into an Excel spreadsheet for archiving and later analyses. CUTR staff performed the review and data analyses using SPSS (Statistical Product and Service Solutions) software.

Prior to the analyses, survey responses were weighted based on the total weekday ridership and completed surveys for each route to more accurately reflect Busway and LYMMO ridership as a whole. Weighting factors were derived to ensure proper representation of Busway and LYMMO customers. Specifically, weights were calculated by dividing the total weekday ridership (obtained from MDT and Lynx staff) for the survey period by the number of surveys returned. The resulting weight factors were applied to each completed survey's data for statistical analysis. The reader should keep in mind that the survey methodologies involved the survey of willing customers. The methodologies correspond most closely with ridership data that are reported as unlinked trips. Table 2 indicates Busway ridership figures for March 19-23, 2001 and Table 3 shows ridership for the LYMMO for December 20, 2001. The data in Table 2 are representative of the five-day (Monday through Friday) total weekday ridership and the data in Table 3 represent monthly LYMMO patronage. Daily ridership figures were not available for either of the two BRT systems.

The response rates for the on-board surveys of Busway and LYMMO customers ranged from a low of 6.45 percent to 23.7 percent. Although somewhat low, these response rates are fairly usual for surveys of this type where prior experience has shown them to be in the 10 to 20 percent response range. In addition, the point needs to be reinforced that the following results are based on a sample of system users and not a 100 percent census. There is the chance of some customers not choosing these two BRT systems because they felt that additional factors not discussed in the results were more important to their selection of mode choice. In addition, it needs to be made clear to readers that survey instruments were not originally designed to ask customers of these two BRT systems about their satisfaction with the Busway and LYMMO compared to other alternatives such as standard local bus. For example, the Busway survey could have included a question asking respondents to indicate their satisfaction with the travel time on Busway buses versus the travel time on standard local Miami-Dade Transit bus service. It should be understood that everyone has a different approach to determining satisfaction with the various components that comprise a particular mode including travel time and frequency of service, for example. It is only when customers are asked to directly compare the various BRT components to those of other modes that comparable results can be obtained. Nevertheless, the results presented in this paper present the measurement of actual customer satisfaction with the two BRT systems. At present, there are many BRT systems in the planning and design phases as well as in operation that will benefit from the results presented in this paper.

TABLE 2. Weekday Busway Ridership—March 19–23, 2001

Entire Route	Weekday Ridership*	Percent of Ridership on Trips Surveyed
1	8,182	17.4
31/231 (Busway Local)	8,820	18.8
38 (Busway MAX)	17,368	37.0
52	6,619	14.1
252 (Coral Reef MAX)	4,491	9.6
287 (Saga Bay MAX)	1,491	3.2
Total Busway Routes Ridership	46,971	100

* Total weekday ridership for the entire route length.

TABLE 3. Monthly LYMMO Ridership—December 2001

Week (Saturday through Friday)	Ridership	Proportion of Ridership on Trips Surveyed
December 1–7	20,618	27.8
December 8–14	20,592	27.8
December 15–21	19,992	27.0
December 22–28	10,304	13.9
December 29–31	2,541	3.4
Total Ridership	74,047	100

Measuring the Importance of Various BRT Elements

Mean Scores

Questions 17 (Busway) and 13 (LYMMO) on the survey instruments were multi-part questions that asked customers to rate their perception of different aspects of Busway and LYMMO BRT services, using five-point scales (1 = "very dissatisfied" and 5 = "very satisfied"). Each survey included a question that asked about the overall customer satisfaction with the BRT services offered by both systems.

These two questions offered customers an opportunity to rate their individual levels of satisfaction with various service characteristics. Using the five-point rating system's numerical scoring values, an average score was calculated for each service characteristic. The resulting mean scores give a good indication of overall customer satisfaction with each of the service aspects. Since a score of 5 indicates a "very satisfied" rating, the closer to 5 that a characteristic's mean score is, the higher the degree of customer satisfaction is with that particular characteristic.

Table 4 presents all of the weighted average customer satisfaction ratings for the service characteristics included on the surveys. The responses indicate a very high level of satisfaction with the services offered by the Busway and LYMMO; all mean scores fell between "neutral" and "very satisfied," including the aspects travel time and reliability. An analysis of the very high customer mean scores and importance of the service attributes inquired about clearly shows that users regard the Busway and LYMMO BRT systems as premium services.

TABLE 4. Means Satisfaction Scores for Busway and LYMMO

Characteristic	Mean Score	
	Busway	LYMMO
Safety on bus	3.81	4.41
Availability of seats on the bus/comfort	3.60	4.41
Dependability of buses (headway adherence)	3.18	4.47
Travel time on buses	3.63	4.48
Cost of riding buses	3.76	Not asked
Availability of information/maps	3.69	Not asked
Convenience of routes	3.69	Not asked
Satisfaction with recent changes to Busway (traffic signals)	3.68	Not asked
Safety at Busway stops	3.65	Not asked
Hours of Busway service	3.50	Not asked
Frequency of Busway service	3.25	Not asked

STEPWISE Regression

The simplest way to measure the importance that customers of public transit place on specific service characteristics is to calculate mean scores for each characteristic on some type of numeric scale (for example, a scale of 1 through 5). While there are no real discernable drawbacks to this simple method, an alternate and more advanced technique to measure the importance of each service attribute is to derive importance by examining the relationship of each attribute to overall customer satisfaction. This methodology uses STEPWISE regression analysis to estimate the importance of each service attribute. While there is a degree of inter-correlation between each of the service attributes, this method can be used to measure the relative importance of each attribute when determining what elements or combination of elements comprise overall customer satisfaction of these two BRT systems. By using STEWISE regression, the r-squared values can be used as surrogates for customer satisfaction.

The STEPWISE regression analysis enters independent factors (each BRT service characteristic) one at a time, backwards and forwards, to determine which one has the highest correlation with the dependent factor (in this case, overall customer satisfaction). Additional independent factors are entered into the regression equation only when they make a significant contribution to the predictive power of the equation. During the process, if any of the independent factors falls below the specified criterion, it is removed automatically from the equation building process. In this case, the criterion for entering the regression equation was $p < 0.05$, and the criterion for removal from the regression equation was $p > 0.10$.

The STEPWISE regression analysis resulted in all four of the service characteristics entering the regression equation, accounting for 69.3 percent of the customers' overall satisfaction with the LYMMO service. For the Busway, the STEPWISE regression analysis resulted in all eight of the service characteristics entering the regression equation, accounting for 67.3 percent of the customers' overall satisfaction with Busway service. Or, put another way, these service characteristics aided in understanding between almost 64 and 70 percent of overall customer satisfaction with the Busway and LYMMO services, as shown in Tables 5 and 6.

TABLE 5. Results from LYMMO Customer Satisfaction STEPWISE Analysis

Model Depend. Variable	Model Independent Variables	R	R-Square	Adjusted R-Square	Std. Error of the Estimate
Overall customer satisfaction	Comfort	0.750	0.563	0.563	0.473
	Comfort + Travel Time	0.810	0.656	0.656	0.419
	Comfort + Travel Time + Reliability of Service	0.830	0.689	0.689	0.399
	Comfort + Travel Time + Reliability of Service + Safety	0.832	0.692	0.693	0.396

TABLE 6. Results from Busway Customer Satisfaction STEPWISE Analysis

Model Depend. Variable	Model / Service Characteristics	R	R-Square	Adjusted R-Square	Std. Error of the Estimate
Overall customer satisfaction	Frequency of Service	0.694	0.481	0.480	0.734
	Frequency of Service + Travel Time	0.771	0.594	0.593	0.649
	Frequency of Service + Travel Time + Seat Availability	0.792	0.628	0.627	0.622
	Frequency of Service + Travel Time + Seat Availability + Convenience	0.805	0.649	0.647	0.605
	Frequency of Service + Travel Time + Seat Availability + Convenience + Hours of Service	0.814	0.662	0.660	0.594
	Frequency of Service + Travel Time + Seat Availability + Convenience + Hours of Service + Safety on Bus	0.818	0.669	0.667	0.588
	Frequency of Service + Travel Time + Seat Availability + Convenience + Hours of Service + Safety on Bus + Dependability	0.821	0.674	0.671	0.584
	Frequency of Service + Travel Time + Seat Availability + Convenience + Hours of Service + Safety on Bus + Dependability	0.823	0.677	0.673	0.582

Busway

For the Busway, the first three-service characteristic to enter the regression equation were “frequency of service,” “travel time,” and “seat availability” (comfort). These three independent variables accounted for 62.7 percent of the equations overall predictive power, or overall customer satisfaction with the Busway. This finding is not surprising given the results for the simple mean scores for these service aspects where Busway customers rated each highly given the more “rapid” (real or perceived) nature of Busway service compared to MDT local service. Each of these service aspects (independent variables) is an important element of BRT service. The remaining service aspects to enter into the Busway STEPWISE regression model were, in order of entry, “convenience of routes”, “hours of service”, “safety on Busway vehicles”, dependability (on-time performance), and the availability of route information. These remaining five variables added only 4.6 percent to the models overall predictive power. All of the service characteristics are significant at the $p < 0.05$ level.

However, one important Busway service characteristic that did not enter into the regression equation as originally hypothesized. The Busway service aspect in question is “cost of riding the bus.” This result is counterintuitive to what is assumed about the factors that customers weigh in their decision to use local bus service. However, with a premium service such as that offered by a BRT system, it appears that cost is less of a concern than the overall quality of the BRT service and travel timesavings that is offered to customers. By its omission into the regression model, the data seem to indicate that if high quality premium service is offered, persons are willing to pay a little extra for the additional benefits of such a system.

LYMMO

The first service characteristic to enter the regression equation was “comfort of the LYMMO vehicles,“ accounting for 56.3 percent of the equations overall predictive power. This result is not surprising given that customers indicated that they liked the low-floor vehicles and modern vehicle interiors the most, each of these an important “comfort” element and aspect of BRT service. The second service characteristic to enter the regression equation was “travel time on LYMMO vehicles.” The entry of “travel time” into the regression equation increased its overall predictive power to 65.6 percent, a significant increase in predictive power. Again, this result is not too surprising given that LYMMO customers indicated that they elected to use the LYMMO service because it is faster than walking to their destination. This finding is consistent with the “rapid” or “perceived rapid” nature of BRT services such as the LYMMO. The third variable to enter the regression equation was “reliability of LYMMO service.” Interestingly, this service characteristic only marginally increased the overall predictive power of the regression model. This result is somewhat hard to explain, given that customers of public transit systems typically put a high premium on vehicle reliability that includes both on-time performance and vehicle breakdowns. The same holds true for the final service characteristic, “safety on vehicles,” that entered into the regression equation. This service characteristic increased the predictive or explanatory power of the overall regression equation by only 0.004 percent. All of the service characteristics are significant at the $p < 0.05$ level.

DISCUSSION AND CONCLUSIONS

Based on the results of the STEPWISE regression analysis, it appears that an argument could be made for a narrow and comprehensive set of traits as the basis for defining and providing different applications of BRT service. Based on the idea of providing a premium service that is more comfortable, frequent, rapid, and reliable than “typical” local bus service or other modes, BRT could be treated as an attempt to inject new energy and life into stagnant local transit bus services. Building on the results from this analysis, the unique services aspects of BRT that can be added to improve other bus services is good for all concerned.

Much discussion of late in the transit industry has been made about how to make BRT distinct and different from standard local service within an individual transit system. The answer may be found not in the type of vehicles that are provided to riders, but found mainly in the quality of BRT service that is ultimately offered. One only has to look at the success (increased ridership, decreased travel times) of the different BRT applications in Los Angeles; Pittsburgh; Ottawa, Canada; Brisbane, Australia; and Curitiba, Brazil to see the virtue of this statement. All of these BRT systems provide extremely frequent, reliable, easy to use, comfortable, safe, and fast (rapid) service (even in mixed traffic) essentially using conventional-looking buses. The results from the STEPWISE regression analysis seem to suggest that these systems are providing the right mix of service aspects to foster sustained patronage and growth. Perhaps what the customer really wants is to get from home to work and back again in the shortest time with the greatest overall level of comfort and personal safety (and to a degree, the cost of riding may not be an overriding factor). The results from this paper suggest that future customers will rely more on the quality of the BRT service that's offered than any other aspect. Again, the success in terms of ridership gains and public acceptance of the Busway and LYMMO provide ample evidence to support this suggestion.

Based on MDT analysis, the Busway seems to have provided little or no travel timesavings for Busway vehicles compared to existing local service – yet, ridership in the corridor increased by 49 percent on weekdays, 69 percent on Sundays, and 130 percent on Saturdays since May 1998. This increase is mostly explained by the 72 percent increase in weekly revenue miles. This suggests that the MDT management did a good job of listening to customers when deploying and implementing Busway service. The combination of Busway service characteristics including high frequency, both in the peak and off-peak, travel time (real or perceived), and seat availability (comfort) are clearly central factors leading to this success.

Lynx reports that despite exclusive running ways and signal pre-emption, average roundtrip speeds are not as great as expected and are one-third slower on the LYMMO than its downtown predecessor the *FreeBee*. The reasons for this are hard to discern. One possible explanation is that LYMMO buses stop at each station, whether customers are waiting or not. Another possibility is that increased ridership has resulted in additional station dwell time during the boarding and alighting process - despite the use of low-floor vehicles and no fare collection. Despite the slow average system speed, LYMMO ridership has increased dramatically since system implementation – the real measure of success. The other possible sources for increased ridership other than increased service hours is the creation of an overall pleasant and safe riding experience, an aggressive marketing campaign, comfort of the LYMMO vehicles, travel time (whether real or perceived) of LYMMO vehicles, reliability of LYMMO service, and safety on LYMMO vehicles and at stations.

It should be understood that every customer of public transit has a different approach to determining their satisfaction with the various components that comprise a particular mode including travel time and frequency of service, for example, and their decision to use that mode at any given time. There is a chance that factors not present in the two BRT systems analyzed in this paper could have caused customers not to choice the BRT mode for their trip making. For example, the Transit Cooperative Research Program (TCRP) Report 47 (*2*) offers many different potential measures of transit service quality including overcrowding, bilingual signage and system information, quietness of vehicles, fairness of fare structure, announcements of delays, cost of making transfers, absence of offensive odors, ease of paying fare, number of transfer points outside of the downtown core, courteous system staff, physical condition of stations, station access, posted minutes to next bus, and so on in addition to the factors used in this paper. The survey instruments used to gather information for this paper were not originally designed to ask customers of about every possible service characteristic related to the Busway and LYMMO. Nevertheless, the results presented in this paper present the measurement of actual customer satisfaction with important service characteristics of the two BRT systems and those elements that are important to all BRT systems. At present, there are many BRT systems in the planning and design phases as well as in operation that will benefit from the results presented in this paper even using a limited number of service quality measures to determine overall customer satisfaction.

Although the R^2-values are fairly high even with the small number of independent factors (4 for the LYMMO and 10 for the Busway), it is important to note that about 33 percent of with the Busway and 31 percent with the LYMMO service related to overall customer satisfaction remains unexplained. As part of the BRT evaluation processes, a number of focus groups will be conducted that could aid in uncovering the remaining factors related to overall customer satisfaction. Certainly, the four service characteristics included in the regression equation make it clear that they are important factors to customers of this BRT system. However, the unexplained variance also makes it clear that a full understanding behind the dynamics of customer satisfaction may require the inclusion of additional independent variables in futures regression analyses as noted in the preceding paragraph. These service characteristics would certainly include those present in other BRT systems or perhaps psychological factors related to customer satisfaction.

While BRT is the talk of the U.S. public transit industry (and even the global transit industry), there is still a long way to go to make this a successful and publicly accepted mode of public transportation as in other places including Canada, South America, Australia, and Europe. There is still a continued need for marketing, vehicle development, data collection, project evaluation, an updated Alternatives Analysis process to include BRT, revised New Starts eligibility criteria, research, and additional technology transfer. The author supports the statements made by the FTA that no single mode of public transportation is right for all situations. However, given the incontestable merits of BRT, it should receive serious consideration as an important alternative in the planning toolkit.

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The Role of the Street Environment in How People Cross Roads in Urban Settings

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ABSTRACT

This paper models the role of the street environment in how people cross roads in urban settings. Respondents were placed in real traffic conditions at the curbside of street blocks in the Tampa Bay area for a three-minute observation of the street environment. Without crossing the blocks, each respondent stated his crossing preference at each of six blocks. The origin and destination of each crossing were hypothetically set and varied across the blocks. So were the options available: two options for crossing at an intersection and up to four options for crossing at mid-block locations. Within the framework of discrete-choice models, the stated preferences are explained with the street environment, including traffic conditions, roadway characteristics, and signal-control characteristics. All three components of the street environment are considered: mid-block locations, intersections, and the roadside environment. This paper describes survey design and data collection efforts; estimates a nested logit model of pedestrian street-crossing behavior; and discusses its implications to researchers and practitioners.

Key words: nested logit model—pedestrian safety—street crossing—street environment

INTRODUCTION

Street crossing is a critical element of the urban transportation environment for pedestrians. A large body of work already exists on street crossing by pedestrians, including the following by subject area:

- crossing delays (*1*)
- crossing opportunities (*2*)
- pedestrians' behavioral parameters (*3–4*)
- pedestrian compliance (*5*)
- pedestrian perceptions toward specific treatments (*6*)
- determination of level of service (*7–10*)
- engineering parameters such as pedestrian clearance intervals (*11*)
- evaluation of treatments (*12–13*)
- drivers' perspective (*14–15*)
- safety (*16*)
- empirical modeling (*17–19*)

However, little research exists that can help answer questions related to pedestrian planning, engineering solutions to pedestrian crossing safety, and research methods for modeling street-crossing behavior. Below are a few examples of these questions:

Planning Questions

- How can existing planning tools for determining pedestrian level of service for street crossing at mid-block locations and intersections be integrated to determine pedestrian level of service at the block level?

Engineering Questions

- How and when might a pedestrian go to a crosswalk at mid-block locations?
- How and when might a pedestrian go to an intersection?
- Where should transit bus stops be located so that transit users are more likely to choose safe crossing options to access them?

Research Methodology Questions

- What statistical models are most appropriate for modeling the street-crossing behavior of pedestrians so that these planning and engineering questions can be answered?
- What and how should data be collected in order to estimate such statistical models?

This paper models the role of the street environment in how pedestrians cross roads in urban settings. Specifically, 86 participants placed in real traffic conditions at the curbside of 48 street blocks in the Tampa Bay area observed the street environment for three minutes. Without crossing the street blocks, the participants stated their crossing preferences at each of six blocks. The origin and destination for each crossing were hypothetically set and varied across the blocks. So were the options available: two options for crossing at intersections and up to four options for crossing at mid-block locations. Within the framework of discrete choice models, the stated preferences are explained by traffic conditions, roadway characteristics, and signal-control characteristics.

The paper focuses on the street environment so that variables are readily measured for model applications. As an alternative, one could model the role of the direct attributes, such as safety and time, that pedestrians may tradeoff in choosing a crossing option. By focusing on the street environment, the paper assumes that the indirect attributes that characterize the street environment determine the direct attributes and that the street crossing behavior can be modeled with these indirect attributes equally well. As another alternative, one could include the street environment as well as pedestrians' personal characteristics. It is recognized here that these characteristics are potentially important in how pedestrians cross roads. They are excluded solely because data on them are not readily available for model applications. The impacts of these two alternative specifications on model results are reported elsewhere (*20*) and are briefly described in this paper when its research implications are discussed.

The rest of the paper has four sections. They describe: (1) the design of the stated-preference survey, (2) data collection efforts, (3) model estimation results, and (4) shortcomings of the study and its implications to pedestrian planning, engineering solutions to pedestrian crossing safety, and research, respectively.

SURVEY DESIGN

A stated-preference approach was chosen for several reasons. It resulted in wide ranges of variation in the street environment. It allowed solicitation of preferences in real traffic conditions. It also resulted in a manageable number of crossing options for modeling. The design process involved four steps:

1. Identify potential determinants of pedestrian street-crossing behavior.
2. Determine levels of key determinants through the selection of street blocks.
3. Formulate crossing scenarios by defining crossing origins and destinations, crossing options, and temporary mid-block crosswalks.
4. Develop instruments for individual crossing scenarios.

These reasons and design steps differ from those for a standard stated-preference survey (*21*).

Determinants

Two steps were used to select potential determinants that describe the street environment. The first step identified the direct attributes that pedestrians may tradeoff in making a choice: comfort, safety, time, and predictability. Predictability refers to the uncertainty in the amount of time an option may take a pedestrian to cross. The second step identified the indirect factors that may determine the direct attributes.

Comfort and Predictability

Differences in comfort result largely from differences in exposure to unpleasantness (e.g., hot weather) and personal traits that influence comfort sensitivity (e.g., poor health). Such differences are captured with roadside walking and crossing distance. Roadside walking could vary significantly across options. Crossing distance varies when jaywalking is involved or when the choice involves intersections and mid-block locations that have different width. Variation in predictability results from the presence or absence as well as the spacing of traffic signals.

Safety and Time

The amount of time spent walking along a street is determined by the distance involved and speed of walking. Distance is already identified as a potential factor in the paragraph above. The potential factors for safety, crossing time, and waiting time are discussed below for crossing at mid-block locations, crossing at intersections, and roadside walking separately.

Mid-block. Chu and Baltes (*22*) identify potential determinants for pedestrian crossing behavior at mid-block locations, based on supply of gaps, crossing time, and safety margin, which form the three components of the gap-acceptance behavior of pedestrians (*17*). Safety margin is the difference between the time a pedestrian takes to cross the traffic and the time the next vehicle arrives at the crossing point.

Intersections. Crider et al. identify potential determinants for pedestrian crossing behavior at intersections (*8*). These are done separately for safety and delays. Safety consists of conflicts with motor vehicles and pedestrian's exposure to these conflicts. Vehicle movements at an intersection that cross the crosswalk represent conflict volumes. Exposure consists of crossing distance, presence of crosswalks, and presence of curb or sidewalk, and median type. For pedestrian delays, the potential determinants differ between signalized and un-signalized intersections. At signalized intersections, pedestrian crossing delay depends on cycle length for crossing with a pedestrian signal and on the facility's green ratio for crossing without a pedestrian signal. At un-signalized intersections, pedestrian crossing delay is a function of the conflict volumes described above in relation to pedestrian safety.

Roadside. Landis et al. identify potential determinants for pedestrians walking along roadsides (*23*). Through a step-wise regression process, the authors identify factors describing the roadside environment, including the various components of lateral separation between sidewalks and traffic lanes.

Sites

The selection of blocks for the field survey determined the values for most aspects of the street environment and the combinations of these values. The following criteria were used:

- All blocks had two intersecting roads at the two ends with through movement.
- All blocks were on roads that are functionally classified as collector or above in urban settings.
- The blocks were from four different areas of the Tampa Bay region. In order to facilitate survey logistics, the selection was further limited to a circle of 5-mile radius within each of four areas: northeast Tampa, South Tampa, Clearwater, and St. Petersburg.
- A number of potential determinants were considered, including number of lanes, presence and type of medians, signalization and crosswalk marking at intersections, pedestrian signal heads at intersections, sidewalks, lateral separation between sidewalks and traffic lanes, and block length.
- A wide range of combinations of the values of the considered determinants was included. For example, it is desirable to have blocks on a 6-lane road with medians and blocks on a 6-lane road without medians.
- A total of 48 blocks were selected with 12 from each sub-area. The number 48 was chosen because it resulted in 12 blocks in each area. Field surveys were done on different days in the different areas. Furthermore, the 12 blocks in each area were divided into two groups of 6 each. These two 6-block groups were visited by two different groups of survey participants with each group taken by a bus. Based on the survey experience reported by Baltes and Chu (*7*), a single bus was able to visit six sites in a single day.

The actual selection was a manual process with hundreds of miles of driving and several steps:

- Produce GIS maps that showed roads classified as collector or above within each circle.
- Identify blocks in the field that met criterion 1 and record information on the determinants in criterion 4.
- Based on the information from the field, select 12 candidate blocks within each area that met criterion 5.
- Check selected blocks in the field and adjust when needed.

Scenarios

A crossing scenario is what was presented to survey participants for soliciting their preferences. A crossing scenario for a block consisted of the street environment, the origin and destination of the crossing, and the crossing options available to the participants. Much of the street environment for any block was determined once it was included in the sample of blocks. The only exception was crosswalk markings, particularly at mid-block locations. In addition to defining individual crossing scenarios, the design process determined what set of crossing scenarios each survey participant was presented with.

Start and End Points

The origin and destination for any crossing scenario were called the start and end points (Figure 1). Five potential locations for either the start or end point were considered with equal distance between them. For either the start or end point, two potential locations were at the intersections. These potential locations allowed a total of 25 different start-end combinations. Two combinations of start and end points were randomly selected for each block. For ease of reference, the side of a block with the start point was called the nearside and the other the far side.

Crosswalk Marking

Mid-block crosswalks rarely exist in the study area. In fact, none of the 48 street blocks had a mid-block crosswalk. Temporary marking was instead used to define mid-block crosswalks. About half of the sample blocks had a temporary mid-block crosswalk with three in each six-block group. A manual process was used to determine which three blocks in a six-block group got a mid-block crosswalk or where a mid-block crosswalk was placed on a given block. This determination was made visually with simultaneous consideration of all blocks in the same six-block group and with factors considered shown graphically. Factors considered include roadway width, block length (short, medium, long), presence and type of medians, crosswalk marking at intersections, traffic signals, pedestrian signals at intersections, and the two chosen start-end combinations.

Three materials for marking crosswalks were tested on two clear days, on two blocks, on a six-lane road, with 12 participants: pavement tapes, chalk powder, and four orange traffic cones with two on each side of the road. The question for the test participants was: Did the marking adequately represent a marked crosswalk to you during the test? The answers were on a 1-5 scale with 5 being adequate and 1 inadequate. Chalk powder was easily blown off by passing motor vehicles. Both orange cones and pavement tapes were perceived to be adequate to represent real crosswalk marking and orange traffic cones were as effective as pavement tapes. Orange traffic cones were chosen over pavement tapes for logistical, material cost, and safety reasons.

Crossing Options

For a given start-end combination, a set of up to six discrete options was defined that can approximate most of the potentially infinite number of crossing options. These options are labeled as A through F for ease of reference and defined as follows (left and right are relative to the nearside):

- A = Crossing at the left intersection (left intersection)
- B = Crossing at a mid-block start point at a right angle (cross first and walk later)
- C = Crossing with a jaywalk between the start and end points (jaywalk)
- D = Walking on the nearside to the opposite side of a mid-block end point and crossing there at a right angle (walk first and cross later)
- E = Crossing at the right intersection (right intersection)

- F = Crossing at a mid-block crosswalk that is away from a start or end point (mid-block crosswalk)

The phrases in the parentheses may be used to refer to these options. A jaywalk is the straight-line between the start and end points when the start and end points are not across each other. When the start and end points are across each other, the jaywalk option is unavailable. The diagram in Figure 1 shows these options.

The exact options depend on the particular start-end combination. If both the start and end points are located at mid-block locations but not across each other, for example, options A through E would all be available. If the start point is at the left intersection instead, option B would disappear and option A would no longer involve walking along the nearside. If the start and end points are located at the same intersection, only A and E would be available. In general, there are a total of five possible sets of options from the 25 possible start-end combinations discussed earlier. These are: A-E, A-C-E, A-B-E, A-C-D-E, A-B-C-E, and A-B-C-D-E. On the other hand, option F is available only when a mid-block crosswalk is present and located away from a start or end point. All options are available in the diagram in Figure 1.

Group Crossing Scenarios

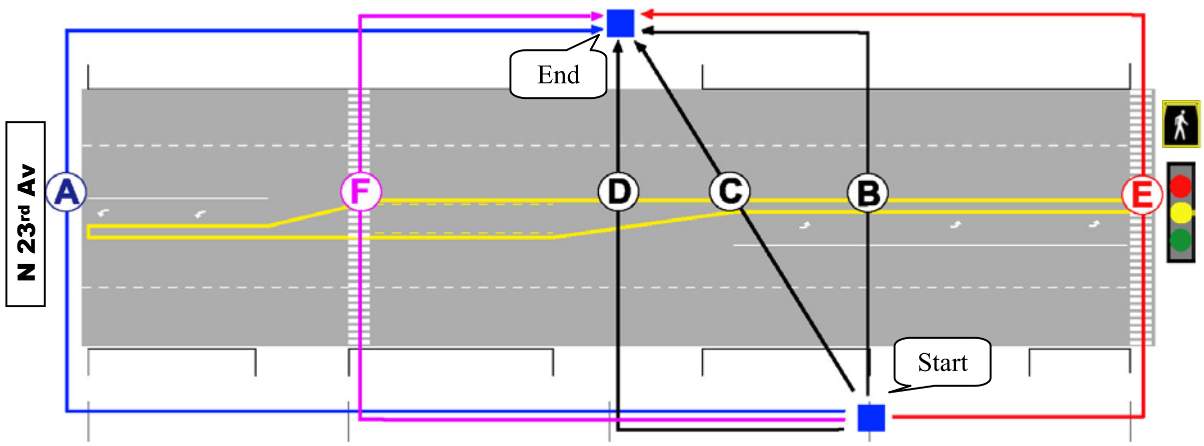
Each participant provided 12 stated-preference responses with 2 responses on each of six blocks in the same circle. As discussed earlier, two start-end combinations were selected for each block. The two responses for a given block from the same participant were for these two different start-end combinations. More is discussed on how these two responses were obtained in the section on field surveys. The particular six blocks within the same area were determined with two considerations. The six blocks would result in a route that is similar in length with the other six blocks in the same area. Each six-block group would have as much variation as possible in key determinants.

Instruments

Ninety-six instruments were developed with each block having two of them, corresponding to the two start-end combinations. Each instrument showed a scaled diagram of the actual block in color. The crossing options, including both the path and the letter label, were coded in colors that were consistent across all instruments to reduce confusion to the participants. For logistical reasons, the start and end points for one of the two combinations were coded in red and the others in blue for a given block. Figure 1 shows an example of the instrument with a start-end combination in blue. Note that the duration of three minutes was chosen so that the participants can observe the street environment for a full signal cycle in most cases. Also the exact order of the options in an instrument depends on where the start and end points are located.

Please enter your PIN here: _______________

The diagram below shows your start point, your end point, and your location options for crossing the street within this block.



Please stand at your start point and observe the block characteristics and traffic conditions for 3 minutes. Based on your observation of the block and evaluation of the options during these 3 minutes, please tell us your choice for crossing this street by selecting **one** from below:

A **F** **D** **C** **B** **E**

FIGURE 1. Sample Survey Instrument for Stated Preferences

DATA COLLECTION

Several aspects of the data collection logistics were discussed earlier. This section focuses on collection of static data and field surveys.

Static Data

Data describing the static aspects of the street environment were collected while the survey instruments were being developed. A form was developed for field collection. It had a section for data related to crossing conditions at each of the five potential start points and a section for data related to the roadside environment. Before any data were recorded, block length was measured and each of the five possible start and end points were marked. In addition, the pre-selected start-end combinations were color-coded into blue or red as designed.

Field Surveys

The final sample of 86 survey participants was recruited through a temporary staffing agency. The initial target sample size was 96 so that a total of 24 would participate on each of the four survey days with 12 on each bus. Ten did not show up for all four days combined. This approach to selecting participants gave greater certainty in the number of recruited participants who actually showed up. Given the fact that completing the field surveys for any given participant took about 5 hours, recruiting volunteers through random sampling of residents in the study area would not have worked as well.

Field surveys were conducted toward the end of April 2002. Prior to departing a central location each day, participants were given verbal instructions and a participant identification number (PIN) at random. The PINs were numbered consecutively from 1. After the briefing, those participants with even PINs boarded one bus and the others the other bus.

At each block, the participants from the same bus were divided into two groups of around 5 to 6 in each group. Two survey workers brought one group to the blue start point and the other to the red point. These two survey workers were supervisors for the two start points. Both of them recorded the PINs of those participants at their start point. Both were also responsible for distributing and receiving the instruments and checking whether the instruments were filled properly. One of them was a timer as well who not only determined when to start and end a particular crossing scenario but also recorded times. In addition, the timer had a sheet with all six blocks that color-coded the locations of the two start-end combinations and mid-block crosswalk marking. At the same time, one survey worker brought one red flag and one blue flag to mark the end points for the two combinations. Another brought the orange traffic cones to the appropriate locations if a mid-block crosswalk was required. Two survey workers got into position for collecting turning movements at intersections and two others for turning movements at mid-block locations (including driveway volumes and u-turns) as well as large vehicles (i.e., trucks, buses, and vans that are larger than regular household vehicles). These were the data collectors, who used pre-developed forms for these dynamic data.

Once everyone was in position, the timer signaled to everyone when to start a crossing scenario. Once started, the participants were given three minutes to observe the street environment as indicated on the survey instrument and were asked to fill the instrument right after being instructed to stop observation. Meantime, the data collectors were recording turning movements and the number of large vehicles. Also, pre-laid traffic counters were recording directional traffic volumes by speed ranges. Four two-way radios were used for communications. Once each group was done with its first crossing scenario for the block, the two groups then switched locations with each other. Once both scenarios were done for a given block, everyone boarded the bus and traveled to the next block.

Data Set

A data set was developed from the survey scenarios, static data, dynamic data, and stated preferences. It contained a total of 1,028 observations (out of 1,032 possible observations) and 38 independent variables. Among the variables, 6 are trip characteristics, traffic characteristics, roadway characteristics for crossing, and traffic control characteristics, respectively. Seven are roadside characteristics that are measured separately for each side of the blocks. The first two columns of Table 1 explain these variables.

Blocks

The 48 blocks had a range of combinations of the potential determinants considered. The average length was 618 feet with a minimum of 232 feet, a maximum of 1,300 feet, and a standard deviation of 314 feet. There were 15 blocks on a 2-lane road, 16 on a 4-lane road, and 17 on a 6-lane road. Sixteen of these blocks were undivided; 20 were with restrictive medians (raised or grassy); and 12 were with painted medians. Crosswalk marking was present at both intersections for 7 blocks, at one intersection for 24 blocks, and at none of the intersections for 17 blocks.

Participants

The participants had more females than males but had a reasonable spread by age and household income. The 65+ age group crossed far fewer roads than the younger groups on the day before survey, ranging from one third of the average crossing by the 25-44 group to one half of the average by the other groups.

On the day before survey, the female participants crossed roads one and half times versus three and quarter times by the male participants. Few of the participants perceived themselves having difficulty with walking at normal speed. These were evenly distributed between the two genders. Far more of them perceived having difficulty with walking at higher walking speeds, however, especially with the 45-64 group. Every one of the 13 participants who reported no difficulty at normal walking speed but reported difficulty at higher walking speeds was female.

Descriptive Statistics

All potential independent variables were examined for correlation. For example, traffic volume is positively correlated with green time and crossing distance for each intersection option with a correlation coefficient slightly over 0.5. This information was then used later in model estimation. In addition, individual independent variables were examined for the reasonableness of their mean, standard deviation, maximum value, and minimum value.

ESTIMATION

Hypotheses

Hypotheses were formulated for a statistical model and expected directions of effects of the independent variables.

Statistical Model

It was hypothesized that the most appropriate statistical model is the nested logit model (*24*). It is natural to view the six potential options for street crossing as two distinctive groups: those related to crossing at intersections and those related to crossing at mid-block locations. That is, the nested logit model has a two-level structure. The top level has two branches: intersections (I) and mid-block locations (M). The bottom level has two options in the intersection branch (A and E) and up to four options in the mid-block branch (B, C, D, F).

Independent Variables

The hypothesized direction of effects of independent variables was based on a basic specification of the utility functions. This specification involved two aspects. First, all variables were to be entered linearly to reduce the complexity of the model. Second, the specific utility functions to which a particular independent variable may enter were determined. Several criteria were used for this purpose.

- Whether an independent variable is constant across the options (e.g., roadside walking varies but not total traffic volume).
- Whether an independent variable is defined for each crossing option (e.g., signalization is defined for intersections only).
- Whether a specific direction of effects could be hypothesized (The width of shoulders or bike lanes is likely to increase the probability that pedestrians choose options that require roadside walking but is likely to decrease the probability that they choose options that do not require such walking).

Based on this specification, hypotheses were formulated for each independent variable. Table 1 also shows the specification and hypotheses.

TABLE 1. Variables and Hypotheses

Variables		Hypotheses							
		Individual Options						Branches	
Description	**Unit**	**A**	**E**	**B**	**C**	**D**	**F**	**I**	**M**
Trip									
Walking distance	Feet along roadsides	-	-	-	-	-	-		
Crossing distance	Feet on travel lanes	-	-	-	-	-	-		
Start and end at mid-block locations	1 if true; 0 otherwise								+
Start at mid-block & end at intersection	1 if true; 0 otherwise								+
Start at intersection & end at mid-block	1 if true; 0 otherwise							+	
Start and end at intersections	1 if true; 0 otherwise							+	
Traffic									
Traffic volume	Vehicles per hour			-	-	-	-		
Mid-block running speed	Miles per hour			-	-	-	-		
Vehicle mix	Percent trucks			-	-	-	-		
Driveway volumes	Vehicles per hour			-	-	-	-		
Mid-block U-turns	Vehicles per hour			-	-	-	-		
Intersection turnings	Vehicles per hour	-	-						
Roadway-Crossing									
Right-turn lane	1 if present; 0 otherwise	-	-						
Left-turn lane	1 if present; 0 otherwise	-	-						
Acceleration lane	1 if present; 0 otherwise	-	-						
Crosswalk marking	1 if marked; 0 otherwise	+	+	+	+	+	+		
Restrictive medians	Width in feet	+	+	+	+	+	+		
Non-restrictive medians	Width in feet	-	-	-	-	-	-		
Roadway-Roadside									
Driveway frequency (nearside)	Number			+	+				
Sidewalk (nearside)	1 if present; 0 otherwise			-	-				
Buffer (nearside)	1 if present; 0 otherwise			-	-				
Barriers in buffer (nearside)	1 if present; 0 otherwise			-	-				
Curbed roadside (nearside)	1 if curbed; 0 otherwise			-	-				
Width of outside lane (nearside)	Feet			-	-				
Width of shoulder / bike lane (nearside)	Feet			-	-				
Driveway frequency (far side)	Number				+	+			
Sidewalk (far side)	1 if present; 0 otherwise				-	-			
Buffer (far side)	1 if present; 0 otherwise				-	-			
Barriers in buffer (far side)	1 if present; 0 otherwise				-	-			
Curbed roadside (far side)	1 if curbed; 0 otherwise				-	-			
Width of outside lane (far side)	Feet				-	-			
Width of shoulder / bike lane (far side)	Feet				-	-			
Control									
Traffic signal	1 if present; 0 otherwise	+	+						
Signal cycle length	Seconds	-	-						
Signal spacing	Feet to next signal	-	-						
Pedestrian signal	1 if present; 0 otherwise	+	+						
Green time	Seconds	-	-						
Green ratio	Unit-less	-	-						

Note: Please refer to hypothesis formulation in the section for Estimation. A = (left intersection); B = (cross first and walk later); C = (jaywalk); D = (walk first and cross later); E = (right intersection); and F = (mid-block crosswalk). I = intersections; M = mid-block. Left and right are determined in terms of the nearside. The nearside of a block is where the start point is.

Model Estimation

Model estimation was a complex process because of the large number of variables and multiple utility functions involved. Model estimation followed two stages and multiple steps.

The first stage resulted in a basic model that included only those characteristics that were explicitly shown in the instruments: traffic signals, pedestrian signals, crosswalks, relative crossing distance, relative roadside walking distance, and the location of the start and end points. These characteristics were highly significant and showed the hypothesized direction of effects in the basic model. This stage followed three steps: (1) estimate a nested logit model of our initial specification; (2) delete variables one at a time that were significant but contradicted our hypothesis; and (3) delete variables one at a time that were consistent with our hypothesis but were insignificant.

One example of the variables that were significant but contradicted our hypotheses was driveway frequency for each roadside. As indicated in Table 1, it is reasonable to expect that people would be more likely to take options that do not require walking along a roadside that has higher driveway frequency.

That is, the coefficients for driveway frequency should be positive as specified. However, they were consistently significant and negative. It is difficult to determine the exact reason for this contradiction. One possible explanation is that driveway frequency is positively correlated with block length. People are less likely to take mid-block options along longer blocks. When block length is not used as an independent variable, the coefficients of driveway frequency may reflect the effects of block length rather than its own effect.

The second stage resulted in our preferred model that included three additional variables: traffic volume, width of a shoulder or bike lane on the nearside, and width of a shoulder or bike lane on the far side. This stage took an opposite approach from the first stage. This was done by starting with the basic model from the first stage and adding one variable at a time that was not already in the basic model. This stage also involved making tradeoffs between certain variables. Signal cycle, for example, made traffic volume become insignificant when both were present although traffic volume worked alone. Since the presence of traffic signals was already in the basic model, it was decided to keep traffic volume rather than signal cycle.

Table 2 presents our preferred model. It contains 10 variables descriptive of the street environment. The model also includes several alternative-specific constants and two inclusive values for the two branches. Note that the columns of coefficients are not in the same order as the options. The coefficients for the two intersection options, A and E, are placed next to each other first. They are followed by the coefficients for the mid-block options. The same order is used in the discussion below. The t-statistics are reported in the parentheses below the coefficients.

The model is well behaved. First, all variables are significant and have the hypothesized direction of effects. Second, it fits the data well. The ρ^2 adjusted for the number of variables is 0.452. In contrast, it is common to see an adjusted ρ^2 below 0.3 in discrete choice models such as mode choice models. Third, the model is consistent with utility maximization (*25*). The scale parameter at the bottom level of the nested logit model was scaled to 1. The estimated coefficients of the inclusive values fall between 0 and 1. Third, the estimated coefficients of the inclusive values are significantly different from 1, indicating that the nested logit model fits the data better than the logit model.

Table 2. Nested Logit Model of Pedestrian Street Crossing Behavior (t-statistics in parentheses)[1]

Variable	Definition	Coefficient							
		Individual Options						Branches	
		Intersections		Mid-block				Inter-sections	Mid-block
		A	E	B	C	D	F	I	M
Alternative-specific constant	1			2.2079 (7.34) [2]		1.7266 (5.55)	1.3875 (3.44)	2.2332 (4.20)	
Walking distance	Feet along roadsides	-0.0034 (-11.65)	-0.0034 (-11.65)	-0.0034 (-11.65)	-0.0034 (-11.65)	-0.0034 (-11.65)	-0.0034 (-11.65)		
Crossing distance	Feet on travel lanes	-0.0027 (-2.31)	-0.0027 (-2.31)	-0.0027 (-2.31)	-0.0027 (-2.31)	-0.0027 (-2.31)	-0.0027 (-2.31)		
Start and end at mid-block locations	1 if true; 0 otherwise								1.5722 (3.14)
Start at mid-block & end at intersection	1 if true; 0 otherwise								0.8415 (2.32)
Traffic volume	Vehicles per hour			-0.0003 (-1.77)	-0.0003 (-1.77)	-0.0003 (-1.77)	-0.0003 (-1.77)		
Crosswalk marking	1 if marked; 0 otherwise	1.0002 (4.30)	1.0002 (4.30)	0.7891 (4.02)	0.7891 (4.02)	0.7891 (4.02)	0.7891 (4.02)		
Width of nearside shoulder/bike lane	Feet if present; 0 otherwise			-0.0728 (-1.22)	-0.0728 (-1.22)				
Width of far side shoulder/bike lane	Feet if present; 0 otherwise				-0.0923 (-1.42)	-0.0923 (-1.42)			
Traffic signal	1 if present;0 otherwise	0.7502 (3.42)	0.7502 (3.42)						
Pedestrian signal	1 if present; 0 otherwise	1.2350 (4.34)	1.2350 (4.34)						
Inclusive value: Intersections	$J_I \equiv Ln(e^{UA} + e^{UE})$							0.7585 (7.05)	
Include value: Mid-block	$J_M \equiv Ln(e^{UB} + e^{UC} + e^{UD} + e^{UF})$								0.8342 (5.87)
Utility function	$\sum$ (Variable * Coefficient)	U_A	U_E	U_B	U_C	U_D	U_F	U_I	U_M

Number of Observations	1,028
Number Cases	4,334
Log likelihood with constants only	-1769.605
Log likelihood at convergence	-963.728
Unadjusted ρ^2	0.455
Adjusted ρ^2	0.453

[1] NLOGIT 3.0 of Econometric Software, Inc. was used to estimate this model with full information maximum likelihood. The RU1 normalization was used for the scale parameters. The nested logit model has two levels with variable options across observations. The top level has two branches: intersections and mid-block locations. The bottom level has two options in the intersection branch (A and E) and up to four options in the mid-block branch (B, C, D, F). A = Crossing at the left intersection (left intersection); B = Crossing at a mid-block start point at a right angle (cross first and walk later); C = Crossing with a jaywalk between the start and end points (jaywalk); D = Walking to the opposite of a mid-block end point and crossing there at a right angle (walk first and cross later); E = Crossing at the right intersection (right intersection); and F = Crossing at a mid-block crosswalk (mid-block crosswalk). I = intersections; M = mid-block. Left and right are determined in terms of the nearside. The nearside of a block is where the start point is.

[2] It is appropriate to determine the significance of the coefficients with a one-sided test because the null hypothesis for each coefficient is either being positive or negative rather than zero. A coefficient would be significant at the 10 percent, 5 percent, and 1 percent level if its t-statistic is at least 1.282, 1.645, and 2.326, respectively. These reported t-statistics do not correct for potential overestimation due to the repeated observations from individual respondents.

One way to understand the model is to look at the implied elasticities, which measure how responsive the choice probabilities are to changes in continuous variables. The model has three of these: crossing distance, roadside walking distance, and traffic volume.

- With respect to crossing distance, the elasticity is -0.099 (A-left intersection), -0.117 (E-right intersection), -0.050 (B-cross first and walk later), -0.584 (C-jaywalk), -0.057 (D-walk first and cross later), and -0.025 (F-mid-block crosswalk). None of the options is responsive to changes in crossing distance. Option C (jaywalk), however, is far more responsive than the other options. That is, pedestrians are far less likely to jaywalk than to take other options when crossing distance increases.
- With respect to roadside walking, the elasticity is -1.547 (A-left intersection), -1.853 (E-right intersection), -0.243 (B-cross first and walk later), -0.345 (D-walk first and cross later), and -0.232 (F-mid-block crosswalk). The probability of an intersection being chosen is highly responsive. An increase of 10 percent in roadside walking could reduce the probability by 15 to 18 percent. In contrast, the probability of any mid-block option being chosen is irresponsive.
- With respect to traffic volume, the elasticity is -0.197 (B-cross first and walk later), -0.273 (C-jaywalk), -0.134 (D-walk first and cross later), and -0.059 (F-mid-block crosswalk). Pedestrians are less likely to choose mid-block options when traffic volume increases. This impact, however, is irresponsive. Furthermore, the elasticity values for options B (cross first and walk later), D (walk first and cross later), and F (mid-block crosswalk) are several times higher in magnitude than those with respect to crossing distance but lower in magnitude than those related to roadside walking distance. For option C (jaywalk), however, the elasticity with respect to traffic volume is only half of that in magnitude as crossing distance.

To present the formula for probability calculations, let U_O (O = A, E; B, C, D, F; I, M) be the sum of the products of all variables in the first column with the corresponding parameter values for option O on the right side columns in Table 1. Note that the inclusive values are $V_I = Ln(e^{UA} + e^{UE})$ and $V_M = Ln(e^{UB} + e^{UC} + e^{UD} + e^{UF})$ for the intersection and mid-block branches, respectively. The probability of a crossing option being chosen is the product of its marginal and conditional choice probabilities. The conditional probability represents the probability of choosing a particular crossing option once the choice has been made between intersections or mid-block options. With intersections being chosen (I), for example, the conditional probability of intersection k (k = L, R) being chosen is given by $P(k / I) = e^{Uk} / e^{VI}$. With mid-block options being chosen (M), similarly, the probability of mid-block option m (m = B, C, D, F) being chosen is given by $P(m / M) = e^{Um} / e^{VM}$. The marginal probability represents the probability of choosing intersections or mid-block options. Specifically, the probability of either being chosen (J = I, M) is $P(J) = e^{UJ} / e^{V}$ where $V = \ln(e^{UI} + e^{UM})$.

DISCUSSION

Limitations

Before discussing potential implications, it is critical to understand the simplifications made as part of the research. One simplification is that the model does not account for the dynamics of traffic conditions and pedestrian's street crossing behavior. The model relates the average traffic conditions during a three-minute period with how a pedestrian may have chosen to cross a street block under such average conditions. Whether safe traffic gaps are available can change quickly over time and across locations along a street block. Such temporal and spatial dynamics in traffic conditions lead to dynamics in the street crossing behavior of pedestrians as well. This simplification falls short for understanding certain crossing behavior, such as mid-block dash, i.e., situations where the pedestrian unexpectedly appeared in front of a motorist while the pedestrian was running and the motorist's view was not obstructed (*26*). Another simplification is that it ignores the role of time constraints. Relative to other direct attributes, time and its predictability would become far more important to a pedestrian when he has a tight time constraint. As a result, he may take riskier crossing options. By excluding time constraints, the usefulness of the model is reduced in understanding the behavior of transit users in trying to catch a coming bus on the other side of the road. The exclusion is made partly because of the difficulty in modeling time constraints.

Implications

Research Methods

A number of implications can be drawn that have both current and lasting value to researchers:

- The results show that pedestrian street-crossing behavior can be reasonably modeled with indirect factors that can be directly measured in practice. In this case, the indirect factors describe the street environment. However, an otherwise similar model based on direct factors alone fits the reported pedestrian street-crossing behavior better. In fact, the adjusted ρ^2 increased from 0.453 to 0.552. The direct factors measure perceived safety, time, and predictability on a scale from 1 (least favorable) to 10 (most favorable). The data were collected from the respondents in the field just after they stated their crossing preference for each crossing scenario.
- Excluding personal attributes from the preferred model appears to have small impacts on the model. An alternative model with added personal attributes was estimated. The addition improved the preferred model with an increase in the adjusted ρ^2 to 0.471. The elasticity with respect to roadside walking was compared, for example, and it increased from –1.547 to –1.593 for the left intersection and from –1.853 to –1.901 for the right intersection.
- The reported results earlier show that the nested logit model fits the stated pedestrian street-crossing behavior better than the conditional logit model.
- The quasi-stated preference approach provides an alternative to the standard stated-preference approach.
- The survey design provides an example of modeling the continuum of street crossing options in real life with discrete methods.

Planning Tools

The existing tools for determining pedestrian level of service are based on simple regression models that predict pedestrian perceptions of quality of service with the street environment. The estimated model from this research could provide a new approach that is based on pedestrians' overall satisfaction with street crossing. Specifically, the estimated utility functions can be combined to provide a meaningful measure of the overall satisfaction from crossing specific blocks: $V = \ln(e^{UI} + e^{UM})$. This concept is similar to using the denominator of a logit destination choice model as an accessibility measure (*27*). More important, this new approach to determining pedestrian level of service is also a behaviorally sound way to measure level of service across different modes equally. The National Corporate Highway Research Program has planned a research project to look for a unified approach for equal measurement of level of service across modes (*28*).

Engineering Solutions

The estimated model may be used to simulate how certain engineering solutions may influence how pedestrians cross streets.

- The model can be used to determine the circumstances under which pedestrians are more likely to go to an intersection or a mid-block crosswalk. With some basic assumptions, curves may be developed to show how different combinations of selected aspects of the street environment influence the likelihood that a typical pedestrian would choose an intersection or a mid-block crosswalk in daytime conditions.
- The model can also be used to determine how marking a mid-block crosswalk may discourage pedestrians from taking risky options.
- Transit stops are often the destination of pedestrians crossing a street. When these stops are located inappropriately, transit users may be more likely to take risky options for crossing. For given origins, the model can help understand how the destination within a block can influence the likelihood of pedestrians to take risky options. The same implication also applies to locating walkways from major activity centers, newspaper boxes, vending machines, etc.

The actual simulation requires additional space to explore and may be carried out in a later paper.

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Conducting a Successful On-Board Survey of Public Transit Customers

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ABSTRACT

This paper presents a best-practices manual that describes the necessary steps in conducting a successful on-board survey of public transit customers. It was specifically developed for the public transit professional that has at least a rudimentary understanding of the purposes and procedures in survey research and is searching for specific guidance on how to "best" conduct an on-board survey of its customers. This how-to manual will help provide public transit professionals with a much better understanding of the total customer surveying process and its importance in planning and ultimately the highest quality service to the riding public. It describes the various components or steps of the on-board transit customer surveying process from specifying and clearly defined objectives, various methods of data collection, questionnaire construction, sample size, appropriate level(s) of analysis, accurate and truthful reporting of results, data entry, report writing, and data archiving.

Key words: customers—data collection—design manual—on-board survey—public transit—questionnaire—sample size

INTRODUCTION

In recent years, there has been a growing awareness of the need to use public transportation resources more efficiently. As a result, it has become very important for public transit systems to carefully evaluate all services so as to provide the most efficient and desirable transit services to the community that it serves. Public transit customer surveys can play an important role in the evaluation of current and planned public transit services. When a public transit system decides to evaluate current or planned services via the use of a customer survey, there are a number of important issues that need to be addressed to facilitate the data collection process and to ensure that reliable and high quality data are collected, analyzed, and ethically reported. In some cases, however, the collection of important information about customers of public transit and the resulting evaluation has not been supported by comprehensive and methodologically valid surveying techniques.

There are a number of important steps that should be followed in the development and conduct of such surveys. This includes simple statistical procedures such as the sampling frame, questionnaire design, and at what level within the transit system information will be collected about customers, for example.

Despite the time and cost associated with such surveying efforts, the results obtained from surveys of public transit customers can be extremely useful to a public transit system's planning and operations functions, as well as to governmental boards, commissions, and councils. Therefore, it is in the best interest of public transit systems to conduct annual periodic surveys of its customers and to make sure that its surveying process is appropriate and correct to meet the desired information needs. The archiving of historical databases should be initiated and used for yearly comparisons of changes in customer demographics, travel patterns, and overall satisfaction with services provided, at a minimum.

SURVEY ELEMENTS

In order to fully identify the many aspects of public transit customer surveying, it was important to examine, review, and summarize the various types of survey instruments (questionnaires) and final reports from as many sources as possible. To achieve the project objective of producing a how-to manual for surveying public transit customers, a review of literature related to surveying, in general, and actual surveys of public transit customers was conducted.

In order to gather and summarize as much literature as possible about surveying public transit customers, the American Public Transit Association (APTA) was contacted and solicited to provide its complete transit system membership mailing list. APTA is a membership organization charged with serving and leading its diverse membership through advocacy, innovation, and information sharing to strengthen and expand public transit in North America. APTA graciously agreed to the use of its complete membership mailing list.

A thoughtful letter was crafted that specifically asked public transit systems on APTA's mailing list for assistance in gathering information about the various types of customer surveys that they have conducted over the past decade or so. The letter asked, if possible, for each of the transit systems to provide a hard copy of all of the on-board or other survey reports that it has or has had completed for the system either internally, by a private consultant, or other entity. The letter stated that it is of particular importance that the research team obtains complete reports that list, in detail, all of the specifics about the customer surveys such as survey design, survey distribution technique(s), and any other relevant methodological issues.

Using APTA's membership mailing list, the letter was sent to over 400 transit systems in both the US and Canada. This effort resulted in a total of 100 reports and other types of important information being sent for review.

The results from the review of literature illustrated the different techniques, methods, and elements used by various transit systems operating various modes such as buses, heavy and light rail, and paratransit, in both urban and rural environments.

Perhaps the most surprising finding from the literature was the absence of a comprehensive source(s) that specifically addressed the unique aspects of the public transit customer surveying. While there are many sources such as textbooks and the like that cover statistical and other general methodological issues of surveying, none cover the unique aspects surrounding surveying customers of public transit.

The results from the literature review made it clear that there are a number of important issues that must be addressed and logical steps followed in the design and conduct of surveys of public transit customers. As noted previously, many important issues such as the statistics related to sampling and the like, are covered extensively in textbooks and other similar publications. The statistical and other methodological issues related to surveying in public transit will be briefly touched on in this how-to manual for reference and general understanding by the end user.

What Is a Survey?

A survey is a system for collecting information to describe the characteristics or attitudes of a particular group of individuals, in this case, public transit customers. Outside of the arena of public transit, surveys are conducted to determine political and consumer preferences and the opinions and beliefs of just about every conceivable issue.

The literature notes that all surveys have the following features in common, regardless of the group or topic being surveyed:

- specific and clearly defined objectives
- method of data collection
- questionnaire construction
- sample size
- appropriate level(s) of analysis
- accurate and truthful reporting of results

From this list of survey basics above, there are many interrelated steps involved in the design, planning, and administration of a survey of public transit customers. Before a survey can be conducted, important and sometimes subtle decisions must be made about the objectives and the purpose of the survey as well as its unique characteristics. A survey of public transit customers usually originates when transit system planning staff are confronted with an information need for real data about existing customers, their preferences, and overall satisfaction with the transit system. It is often the case that information about customers is either insufficient, outdated, or does not exist at all.

Survey Design Steps

Developing, administering, and reporting the results of an on-board survey or other survey of public transit customers includes a number of important steps that should be followed to ensure the highest degree of success with the survey effort. The steps to follow are provided in an outline format. The intent of presenting the steps in this manner is so the end-user of the how-to manual can refer back to previous sections for detailed guidance.

Step 1: Define and Clarify Objectives

- Define expectations
- Define what information needs to be gathered about customers
  - Demographics
  - Travel Patters
  - Satisfaction
  - Other
- Confirming all related costs
  - Staffing, materials, equipment, travel, etc.
- Establishing a reasonable timeline
- Investigate any previous survey efforts
- Coordinate with persons knowledgeable about surveying including academics, consultants, etc.
- Familiarize with this how-to manual

Step 2: Identify Sample

- Decide sampling method
- Probability or non-probability
  - Simple random sample
  - Stratified random sample
  - Cluster random sample
  - Systematic Sample
- Representative of the whole population
- Degree of accuracy
  - Confidence Level
    - 95 percent or other
- Minimize sampling error
- Minimize sampling bias
- Influenced by available resources
- Randomly selected routes
  - Inbound and outbound

Step 3: Data Collection Methodology

- Data collection methodology
  - Self-administered
    - On-board survey
    - Seat drop
    - Driver assist
    - Surveyor assist
    - Intercept

- Mail
- Telephone
  - CATI
- Personal interview
  - CAPI

Step 4: Questionnaire Design

- Develop clearly worded and simple questions
- Keep the questionnaire as short as possible
- Avoid question writing problems
  - Double negatives
  - Probing questions
  - Hypothetical
  - Acronyms
  - Bias
  - Ambiguous wording
  - Double-barreled
  - Cryptic
- Don't assume customers know terminology
- No way to word a question perfectly
- Questions should be relevant to objectives
- Short questions are best
- Avoid slang, jargon, and technical terminology
- Develop consistent response methods (i.e., checks, circles)
- Make questions as impersonal as possible.
- Sequence questions from the general to the specific
- Closed-ended questions should use exhaustive and mutually exclusive response choices
- Place questions with similar content together (i.e., demographics and so on)
- Make the questions as easy to answer as possible
- Provide clear and concise directions
- Define unique and unusual term
- Use an attractive questionnaire format that conveys a professional image
  - Landscape
  - Folded
  - Front-back
  - Paper selection
    - Card stock
    - Copier
    - Color
- Coding of response choices
- Coding schemes
- Numeric or alpha
- Research design
- Consecutive numbering
- Distribution logs

Step 5: Surveyors

- Obtain surveyors (if necessary)

- Extensively train surveyors
    - Refer to checklist in how-to manual

Step 6: Conduct Pre-test of Questionnaire and Survey Methods

- Conduct a pre-test of draft questionnaire using actual persons who will be surveyors
- Fine tune questions and overall survey methods
- Ensure surveyors are trained properly
- Solicit information from the surveyors that participated in the pre-test
- Conduct focus groups and/or brainstorming sessions with surveyors
- Make appropriate modifications

Step 7: Conduct the Survey

- Undertake the survey
    - Operator assistance
    - Surveyor assistance
    - Other assistance
- Collect surveys
    - Operator assistance
    - Surveyor assistance
    - Drop boxes on vehicles
    - Mail return provision

Step 8: Data Processing and Analysis

- Data entry type
    - Manual
    - Electronic
- If manual, then data entry staff
- Data weighting
- Template design
- Software
    - Spreadsheet
        - Microsoft Excel™, Lotus 1-2-3™
    - Database
        - Microsoft Access™
    - Other
        - SPSS™, SAS™
- Data cleansing
    - Transcription errors
    - Logic checks

Step 9: Reporting Results and Other

- Consider audience
    - General or academic
    - Organize tightly
    - Do not overwhelm reader
    - Consider sensitive topics

- Use simple bar graphs, pie charts, tables
- Accuracy of reported results
- Describe survey design
- Describe sampling methods
- Ethical reporting of results
- Data Archiving
- Historical Comparisons
- Consistency between instruments
- Multi-year comparisons
- Length of time between surveys

OTHER CONSIDERATIONS

In 1991, the Federal Transit Administration began a project to implement a transit performance monitoring system (TPMS). The TPMS was designed to collect data on transit customers through an ongoing, systematic program of on-board surveys. The long-term goal of the TPMS initiative is to standardize the collection of data and, thereby, provide a basic, but comprehensive analysis of the performance and benefits of transit service.

The project has consisted of two rounds of surveys, each involving 14 transit systems. The Round 1 surveys were performed in 1997 and 1998. The Round 2 surveys were conducted in 1999 and 2000.

Twelve core questions were included on the survey questionnaire during rounds one and two. The survey form shows that the 12 core questions are 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 12, and 15. The rationale for inclusion of these questions is as follows:

- Questions 1, 3, 8, and 15 are used to determine trip purpose, automobile availability, and income. The responses to these questions are used to define the functions or benefits provided to the customer such as congestion management, low cost mobility, and livable communities. These functions or benefits are discussed in the chapter entitled Survey Results.
- Question 5 addresses trip frequency and also is used to estimate the number of people in the community that use transit service. For example, each response of one day a week might be given a weight of 7.0 to estimate the number of people using transit service one day a week.
- Question 6 is used to assess the degree of turnover in transit ridership.
- Questions 7 and 9 help assess the level of added mobility that transit provides to customers.
- Questions 2 and 4 provide information on access and egress modes.
- Questions 10 and 12 are used to examine the survey responses in terms of age and gender.

When conducting an on-board survey of public transit customers, serious consideration should be given to the inclusion of as many of these twelve core questions and exact response choices as possible.

SUMMARY

It has become very important for public transit systems to carefully evaluate both current and planned services in order to provide the most efficient and desirable public transit services to the community that it serves and funds its existence. Surveys of public transit customers can play an important role in the evaluation of current and planned services. When a public transit system decides to evaluate current or planned services through the use of a customer survey, there are a number of important issues that need to be addressed to facilitate the data collection process and to ensure that reliable and high quality data are collected, analyzed, and ethically reported. In some cases, however, the collection of important information about customers of public transit and the resulting evaluation has not been supported by comprehensive, thorough, and methodologically valid surveying techniques.

This paper presents a how-to manual that describes the steps to follow when conducting an on-board survey of public transit customers. It was specifically developed for the public transit professional that has at least a rudimentary understanding of the purposes and procedures in survey research and is searching for specific guidance on how to “best” conduct such a survey. It is hoped that this how-to manual will help provide public transit professionals with a much better understanding of the total customer surveying process and its importance. This how-to manual describes the various components or steps of the on-board transit customer surveying process

U.S. 83 Texas Study Corridor—FHWA Economic Development Highway Initiative: Border Crossings and Rural Communities

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ABSTRACT

Congress, in fiscal year 2000, appropriated $4.5 million for the Economic Development Highway Initiative, administered through the Federal Highway Administration. This mandate targeted economically disadvantaged regions to determine what highway improvements were necessary to serve as a catalyst for economic development. The U.S. 83 Texas study corridor (Webb, Dimmit, and Zavala Counties) represents approximately 120 miles of highway with “Super Two” characteristics. Economic growth and development characteristics for these counties vary significantly. In 2000, the Laredo port of entry accounted for roughly 41.2 percent of the total value in overland trade with Mexico, making it one of the busiest ports of entry in the hemisphere. This has created a strong demand for trucking, warehousing, staging areas, and support service industries in the region. Dimmit and Zavala Counties to the north are at the other end of the development spectrum. They are very rural and have higher rates of unemployment, poverty, and declining population. With the ever-increasing demand for trade, increasing truck traffic from the west coast, lower land costs in the northern region of the corridor, and interest from industry in locating to the northern region, the study corridor north of Laredo is poised for economic development as well. This growth potential has a direct relationship with transportation, so continuing to make the necessary improvements on U.S. 83 to enhance transportation safety and efficiency is essential for economic development.

Key words: border crossing—economic development—transportation corridor

INTRODUCTION

Congress, in FY 2000, appropriated $4.5 million for the Economic Development Highway Initiative, administered through the Federal Highway Administration (FHWA). This mandate targeted economically disadvantaged regions to determine what highway improvements were necessary to serve as a catalyst for local and regional economic development. The objective of this study was to determine the potential economic development benefits to the region from highway improvements. The possibility of funding highway improvements based on economic development impacts was a radical departure from basing improvements primarily on traffic volume and safety. Economic development is contingent upon a number of requirements that include income levels, education, housing, healthcare, quality of life and infrastructure. These requirements pose major challenges to the economic growth and expansion of the study corridor and are the basis for the corridor's selection.

The U.S. 83 Texas study corridor was one of 12 corridors in nine states selected for this initiative. The U.S. 83 study corridor (Webb, Dimmit, and Zavala Counties) represents approximately 120 miles of highway with "Super Two" characteristics. U.S. 83 runs through the heartland of the United States, from Mexico to Canada, and has a strategic alignment to the major consumer markets in Mexico such as Monterrey and Mexico City.

METHODOLOGY

Due to the significantly different degrees of economic activity between the northern and southern portions of the study area, project initiation meetings were conducted with community stakeholders in both regions. Community leaders, business owners, local government representatives and other stakeholders were invited to participate in these meetings. As a result, two Advisory Committees were established, one representing Webb County (Southern Corridor) and the second representing Dimmit, and Zavala Counties (Northern Corridor). The purpose of the Advisory Committees was to provide feedback and oversight; encourage participation by local interests; provide access to previous studies and analysis; and facilitate input from local, regional and state interests. The study work plan was divided into four primary segments: socioeconomic profile, industry sector analysis, transportation improvement concept, and economic development implications. Study analysis was based on empirical and anecdotal information collected through the course of the study. Study insights and additional information to support existing empirical data were gathered from interviews with the advisory committee, business and community leaders, and stakeholders.

Socioeconomic Profile

The existing socioeconomic conditions of the study area, its people and its businesses, were profiled to formulate a picture of the current socioeconomic situation and their relationships with transportation corridors. Socioeconomic characteristics for the counties in the study area vary significantly. Webb County, which includes the Laredo port of entry, has experienced economic growth due to location. In 2000, the Laredo port of entry accounted for roughly 41.2 percent of the total value in overland trade with Mexico, making it one of the busiest ports of entry in the hemisphere. The phenomenal growth that Laredo has seen in the past ten years is without question a result of the Laredo border crossing and the North American Free Trade Agreement (NAFTA), as demonstrated in Table 1 (*1*). This has created a strong demand for trucking, warehousing, staging areas, and support service industries in the area. Dimmit and Zavala Counties to the north are at the other end of the development spectrum. They are very sparsely populated and have higher rates of unemployment, poverty and declining population.

TABLE 1. Laredo Border Crossing Activity North and South

2000 Amounts	Increase from 1990
9.1 M pedestrians	27.5%
16.8 M vehicles	24.4%
2.9 M trucks (8,000 daily)	314.3%
336 K loaded railcars	242%
459 M lb. landed weight at airport	897.8%

Source: Laredo, Texas, Chamber of Commerce.

Because of the varying degree of economic vitality in the study area, stakeholders view transportation investment from an economic development standpoint differently. In Webb County, transportation improvements are viewed to improve economic development by accommodating more trade related economic stimulation. In Dimmit and Zavala Counties, stakeholders view transportation improvements from the perspective of increased market access, improved competitiveness for attracting businesses and tourists, and improved quality of life.

Dimmit and Zavala Counties are very sparsely populated with 7.7 and 8.9 persons per square mile, respectively. This compares to Webb County with 57.5 persons per square mile and the state and national average of 79.6 persons per square mile (*2*). There are 254 counties in the state of Texas. From 1999 data, Webb County ranked 21, Dimmit County ranked 164, and Zavala County ranked 154. Laredo has approximately 91 percent of Webb County's population and Nuevo Laredo on Mexico's side of the border has a population of more than 660,000. Webb County's population growth rate from 1990 to 2000 of 44.9 percent was approximately double the state's growth rate of 22.8 percent, considerably greater than the national average of 13.1 percent, and ranked ninth in the nation. This compares to –1.8 percent and –4.6 percent growth rates for Dimmit and Zavala Counties, respectively (*3*).

The poverty level for the study corridor is of concern. Webb County has a poverty level of 31.2 percent for people of all ages, Dimmit County 33.2 percent, and Zavala County 41.8 percent. The study area poverty levels are more than double the state average of 15.4 percent and for Zavala County more than triple the U.S. average of 12.4 percent (*2*). The educational attainment of persons 25 years and older also raises concern for social issues and economic development initiatives. The percentage of the population that is 25 years and older that has a high school degree or better is 53 percent of Webb County, 54.3 percent of Dimmit County, and 43.4 percent of Zavala County (*2*). The study corridor is well below the state average of 75.7 percent. This is alarming when economic growth and personal incomes are a direct reflection of the overall labor pool's educational achievement.

Per capita personal incomes (PCPI) for the study area are well below state and national averages as identified in Table 2 (*4*). Average annual unemployment rates for the study area are considerably higher than the state and national averages. From 1990 to 2000, Webb County had a high of 15.3 percent and a low of 7.0 percent. Dimmit County had a high of 20.0 percent and a low of 12.7 percent. Zavala County had a high of 26.1 percent and a low of 15.3 percent. The state for the same period had a high of 7.7 percent and a low of 4.2 percent and the U.S. had a high of 7.6 percent and a low of 4.1 percent (*5*).

The industry sector analysis was conducted to predict trends within industry segments and to identify and target economic development opportunities and their relationship/dependence on U.S. 83. The primary industry sectors identified were transportation and border related activity, agriculture, retail trade, and oil and gas.

TABLE 2. Per Capita Personal Incomes

	Per Capita Income	State Ranking	% State Average	%National Average
Webb County	$14,112	239	53%	49%
Dimmit County	$12,789	247	48%	45%
Zavala County	$11,351	250	42%	40%

Source: Regional Economic Information System, Bureau of Economic Analysis.

Industry Sector Analysis

Transportation and Border Related Activity

Goods movement between the U.S. and Mexico has increased steadily and dramatically over the past decades. The growth rate accelerated during the 1990's to nearly double the growth rate of the 1980's as demonstrated in Table 3 (*6*).

TABLE 3. Average Annual Percent of Change in Volume of Goods Traded

	1980	1990	2000	1980–1990	1990–2000
U.S.-Mexico trade	$28 B	$58 B	$207 B	7.6%	13.6%

Source: Laredo Development Foundation.

The growth in trade volume has had the most dramatic effect in Webb County, where earnings growth in the transport and utilities sector has averaged over 11 percent per year. This is well above the growth rate of Texas at 9.1 percent and U.S. at 6.1 percent. However, the growth in Dimmit and Zavala Counties lags far behind at 2.1 percent and 1.9 percent, respectively (*4*). Laredo with good crossings and good transportation on either side, provides a clear competitive advantage to the region. The advantage is being exploited through close cooperation with Nuevo Laredo in cross-border maquiladora development, outreach to industrial centers in Mexico such as Monterrey, and development of industrial parks, warehouses, and value-added distribution facilities.

Agriculture

Agriculture is a major industry and employer in the region, and is particularly significant in Dimmit and Zavala Counties, which have an extensive winter vegetable garden industry. Farm earnings in Zavala County have far surpassed Texas and U.S. growth rates at 6.0 percent compared to 3.4 percent and 1.3 percent, respectively (*4*). Del Monte, located on U.S. 83, is one of the larger employers in Zavala County. Since 1995, full time employment has risen to 130, a 47.7 percent increase. The plant also supports 800 part time employees who typically work 42–52 weeks per year. They typically receive 10 to 20 truckloads of raw vegetables and ship 40 to 50 truckloads of finished product per week. Another business significant to the corridor is Dixondale Farms located in Carrizo Springs on U.S. 83. Dixondale Farms produces and distributes approximately 60 percent of all the onion seed plants in the U.S. Twenty percent of their distribution is through mail order, primarily to Texas, California, New York, and Ohio, utilizing UPS and FedEx services. They also serve Wal-Marts in 17 states.

Ranching has continued at a fairly steady pace, but many of the ranches now rely on hunting permits for substantial income. According to the Laredo Chamber of Commerce, deer hunting brought in an estimated $40 million in revenue to Webb County in 2000. Along U.S. 83, the local economy depends heavily on income generated during the hunting season, as hunters are good customers for the small retailers.

Retail Trade

Located near the Mexico border and the thriving city of Nuevo Laredo, and with one of Mexico's best roads leading from Monterrey to Laredo, Webb County in particular enjoys a diverse and vibrant retail trade sector. Nuevo Laredo has a population of more than 660,000, and Monterrey, Mexico, with a population of 3.9 million, is only two hours away, by good roads. Many people cross the border on foot or by car on a regular basis to take advantage of the prices and selection in U.S. stores. The retail trade sector has grown annually by 4.8 percent for Webb County and Dimmit and Zavala Counties have shown a steady growth rate of 5.2 percent and 4.9 percent, respectively (*4*). The Laredo Wal-Mart is the busiest store for its size in the entire United States—thanks largely to shoppers from Mexico. Average annual retail employment growth in Webb County out paces the state's growth rate.

Oil and Gas

Oil and gas extraction has clear boom and bust cycles, with volatility ranging from wildcat discoveries of new sources to tensions in the Middle East affecting oil futures and prices. Texas is the nation's leader in oil and gas extraction, producing approximately 58 percent of all U.S.-derived oil and gas. Three years ago the oil industry in Dimmit County started to turn around, and is in a period of recovery. As a result, the Eastern Oil Well Service Company, which services existing wells, has grown during this period. The company has 50 employees, and as such is a major employer in Dimmit County.

Transportation Improvement Concept

The transportation improvement concept for the U.S. 83 corridor was developed to identify current and planned improvements that may lead to changes in transportation travel demand and patterns within the corridor. The improvement concept would set the stage for evaluating the relationship between transportation and economic development. Desired transportation improvements in the study area were identified based on safety, mixed traffic use, and economic development opportunities. For the study corridor, the main issue was traffic mix that included tractor trailers, RVs, delivery trucks, oil drilling rigs and equipment, oil tankers, and local traffic and its relationship to traffic safety and continued road damage as a result of the heavy trucks. U.S. 83 for the most part does not have sufficient lanes for acceleration or deceleration for the large heavy equipment coming on and off U.S. 83 from local farm access roads (oil drilling and exploration equipment, oil tankers, agricultural machinery, etc.) and businesses, which becomes a safety issue when combined with fast moving trucks. As a result, highway improvements would facilitate highway safety and economic development expansion based on increased transportation efficiencies.

Traffic Flows/Routes

During the course of the study, traffic flows/routes based on traffic type became apparent. U.S. 83 serves as an important transportation route and link between economic development and transportation. Well-defined traffic flows and patterns were identified for

- westbound border trade—a two-hour time savings
- wholesale deliveries to local businesses—the necessary goods to support retail
- Lucky Eagle Casino in Eagle Pass—a major regional tourism draw
- RVs traveling north and south—snowbirds to and from the upper Midwest

- students going to and from the Junior College in Uvalde—continuing education

As a result of U.S. 83 being a more direct route and bypassing the heavy San Antonio traffic, trucks going to or coming from the Laredo Border crossing use U.S. 83 as an option for West Coast origins or destinations. Total trip time can be reduced by two hours when taking U.S. 83 north to Carrizo Springs, 277 west to Del Rio, and 90 to I-10. In addition, heavy loaded trucks will detour onto U.S 83 when the scales are open at Moore, on I-35. At times one can almost tell by the increased number of trucks on U.S. 83 when the scales are open.

Traffic Counts

The annual average daily traffic (AADT) counts were tallied in five-year intervals from 1980 to 2000 to identify traffic volumes and patterns and provide percentage change and average annual growth rates to demonstrate the traffic impacts of NAFTA on the study region (*7*). The AADT increase from Carrizo Springs west to Eagle Pass on U.S. 277 supports conversations with retailers and truck drivers that the majority of the truck traffic on U.S. 83 is going to or coming from the west coast. The traffic flows in the area have demonstrated solid growth and there is evidence in most cases of accelerated traffic growth on U.S. 83 in the post NAFTA (1995–2000) period. It is expected that this strong growth rate will continue, resulting from increased trade liberalization and the interaction between and the integration of the North American markets.

U.S. 83 and I-35 are one and the same for approximately 20.5 miles from Laredo heading north. This section had a 167 percent increase in AADT from 5,100 to 13,600 trips between 1980 and 2000. From 1995 to 2000, a 49 percent growth in AADT occurred. When expressed in terms of average annual change, there was a 5 percent yearly growth over the 20-year period, and an 8.4 percent per year growth between 1995 and 2000.

From the U.S. 83 and I-35 juncture, continuing north on I-35 to Encinal at the Webb County line, there was an AADT increase of 232 percent from 3,700 to 12,300 trips between 1980 and 2000. This is equivalent to a 6.2 percent average annual growth. From 1995 to 2000, a 45 percent growth in AADT occurred, which represents an average annual increase of 7.7 percent.

From the U.S. 83 and I-35 juncture, continuing north on U.S. 83 to Carrizo Springs, there was an AADT increase of 59 percent from 1,700 to 2,700 trips between 1980 and 2000. This represents a 2.3 percent average annual growth rate. Between 1995 and 2000, there was a 54 percent increase, which represents a 9.1 percent average annual growth.

On 277 heading west to Eagle Pass from U.S. 83 in Carrizo Springs, the 1980 to 2000 period showed an increase in AADT of 71 percent from 1,750 to 3,000 trips, with a 30 percent increase from 1995 to 2000. The 20-year average annual change on this segment was 2.7 percent, while the post 1995 increase was at the rate of 5.5 percent. This AADT increase from Carrizo Springs to Eagle Pass on U.S. 277 is similar to the 59 percent increase from the I-35 break away to Carrizo Springs, which supports conversations with retailers and truck drivers that the majority of the truck traffic on U.S. 83 is going to or coming from the West Coast.

Current and Planned Improvement Costs

Planned transportation improvements have included new pavement, additional pullover lanes, four-lanes through the larger towns, curbing and guttering, sidewalks and designer retention walls. Cost figures provided by the Laredo District Office of the Texas DOT, showed the Texas DOT having spent approximately $10.9 million dollars on U.S. 83 in the study corridor in FY 2001 and 2002. No funding has been allocated for improvements in FY 2003 and 2004 and $10 million has been earmarked for FY 2005. However, from February 1998 to December 2012, approximately $76.6 million has been spent or earmarked for projects in the U.S. 83 study area.

Improvement Concept Recommendations

Recommendations from advisory committee members and stakeholders advocate the future development of U.S. 83 as a four-lane highway from the U.S. 83 and I-35 junction to Uvalde to provide for improved safety due to the user mix on the existing road, adequate entry and exit access for heavy equipment, and for facilitating economic development. The majority of advisory committee members and individuals interviewed recommend that U.S. 83 should be four-lane to Uvalde with improved shoulders to connect with rail and I-90 to San Antonio and points west. Such an upgrade, according to a preliminary estimate, would cost approximately $225 million. (Figures were provided by the Texas Department of Transportation's Laredo District Office and WSA's Houston Office.)

Other options include additional pullover lanes as expressed by numerous users. Crystal City would like to see signs on U.S. 83 pointing to downtown Crystal City indicating a business route designation. This would encourage more U.S. 83 drive-bys to go into Crystal City and hopefully stop. Other relatively inexpensive recommendations include Scenic Highway and Texas Wildlife Trail signs and historic markers. More signs and other guides to attractions such as birding trails or natural and tourist attractions would take advantage of the eco-tourists.

Economic Development Implications

The economic development implications of the improvement concept were shaped based on the information gathered during the course of the study. Much of this was based on the understanding of the relationship between the success of the local industry sectors (existing and potential) and U.S. 83. It is apparent that opportunities exist for developing distribution facilities in Dimmit and Zavala Counties geared towards Mexican trade, and land is much cheaper than near Laredo. By continuing to improve upon and building better roads, there would be a substantial increase in west coast traffic business, due to the two-hour timesaving alone. Distribution centers could take advantage of the bilingual work force and location, providing jobs in receiving, disbursing, assembling and distributing. The focus would have to be on logistics and transportation, with value added in services such as break-bulk and customs facilitation. It may be possible to develop a lower-cost warehouse/value added niche, possibly in combination with a truck service and inspection center, and perhaps even a truck driving school.

A number of business start-ups have occurred since 1995 (post NAFTA) that have contributed greatly to the local economies in the northern portion of the study corridor. Recent new business start-ups were identified based on interviews, which are not all-inclusive, with a conservative estimate of 160 new jobs.

In January 2002, the U.S. Department Agriculture announced the selection of the Middle Rio Grande FUTURO Communities to receive the empowerment zone (EZ) designation in Round III (*8*). The EZ designation encompasses portions of Dimmit and Zavala Counties in the study corridor. EZ benefits include specific tax advantages that accrue to existing and potential owners, businesses and developers. Tax advantages for Round III designations contain wage credits of up to $3,000 per qualified employee and rural zones can issue up to $60 million each in new tax-exempt bonds. These bonds can be used for shell buildings and financing the construction of a new building or the renovation of an existing building. The Round III Empowerment Zone provides an excellent marketing tool for the communities on the U.S. 83 corridor in combination with its proximity to the border crossing and east and westbound traffic from Eagle Pass and U.S. 83 juncture.

Crystal City Prospects

A number of potential job increases are expected to result from the attraction of businesses to the Middle Rio Grande FUTURO Communities Empowerment Zone and the various incentives it offers. Crystal City, Texas is actively trying to draw into the area three establishments, all of which would heavily rely on utilizing the U.S. 83 Corridor in Southern Texas for their shipments. First, a Mexican cheese processor, currently negotiating with the city, is anticipated to open a processing facility in 2003, employing about 200 individuals when fully operational. Second, the abundance of high-quality water in the Carrizo Wilcox Aquifer has attracted interest from Californian and Mexican entrepreneurs to build a water bottling plant around the Aquifer. Such a plant would generate about 20 direct new jobs in the area. Third, there is a potential for the development of a transportation staging facility along U.S. 83 in the area. This facility would handle approximately 400 containers per month, and create 10 new jobs. The potential impact of these additional jobs would provide a gain in earning power from $7.4 to $7.9 annually as identified in Table 4.

TABLE 4. Potential Economic Impact of Prospects

	Estimated Impact
Potential new positions	230
Conservatively expected increase in job creation	30%–40%
Total employment impact	299–322 new jobs
Average weekly industry wage	$474
Annual earning power increase	$7.4–$7.9

Source: Regional Economic Information System II, Bureau of Economic Analysis.

SUMMARY

U.S. 83, as rural as it may be, has a dynamic role in the development and growth of the small communities along its path. The border trade dynamics of Laredo will continue to increase as a result of NAFTA, which will continue to provide development opportunities along the U.S. 83 corridor. Highway improvements on U.S. 83 will continue to have a positive impact on economic development in supporting its key industries. Local communities are in a good position to take advantage of their empowerment zone incentives and their proximity to U.S. 83 and the border.

Local officials and stakeholders believe with justification based on the current and future economic potential of the region that the U.S. 83 study corridor should eventually be improved to a four-lane highway. The Texas DOT, as demonstrated during the course of the study period, is committed to improving U.S. 83 and recognizes that continued improvements will help the local communities in the corridor and region recognize their potential. However the costs of providing the four-lane highway are too great to be able to implement in the near future, especially during a time of limited resources and competing interests. The Texas DOT will continue to provide additional selective improvements as identified in the transportation improvement plan. This level of improvement should not impede any new economic development activity anticipated by local officials.

Challenges do exist in regard to income levels, education, housing, and healthcare. The local leadership, stakeholders and development agencies in the study corridor recognize these challenges and are leading efforts to address them. They also realize that they are in a unique position to make positive change and to take advantage of their natural resources, cultural diversity, U.S. 83, proximity to Laredo, and NAFTA.

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A Structural Model for the Rapid Analysis of Concrete Pavement Systems

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ABSTRACT

Pavement design is a decision making process which uses pertinent information available to make required judgments. One of the tools used in the design process is analysis of the pavement system. To be of value, it may be necessary to make many analyses of several pavement systems with different loading conditions. With the more complicated models, such as the finite element models (FEM), this may require considerable time on the part of the designer. Furthermore, many consulting firms and designers do not have the necessary background and/or computational tools needed to make many of the required analyses. This paper specifically focuses on the rapid analyses of Portland Cement Concrete (PCC) pavements using artificial neural network (ANN) models.

As part of the research activities of the Federal Aviation Administration Center of Excellence (FAA-COE) at the University of Illinois, artificial neural networks (ANNs) have been successfully used to develop structural models to predict the critical concrete pavement responses (maximum strains, deflections and stresses) for various slab thicknesses, load locations on the slab, subgrade support conditions, and load transfer efficiencies of the joints. These ANN models based on factorials of finite element solutions offer an attractive alternative to the direct use of finite element analysis for determining the critical pavement responses needed for mechanistic based concrete pavement design.

The development and validation of the neural network model for the rapid analysis of the concrete pavement systems will be discussed in this paper. This valuable tool is intended to aid pavement engineers in the investigation of "what if" scenarios before making a final design decision in a relatively very short amount of time (several thousand analyses can be performed in one second using today's typically available personal computers). Potential benefits of using an ANN-based analysis tool and design methodology in concrete pavement design will also be discussed. Examples of pavement analyses using the ANN based tool will be given. Important findings of this application will be outlined in the paper.

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Key words: artificial neural networks—dimensional analysis—elastic layered programs—finite element analysis—jointed concrete pavements—mechanistic-empirical pavement design—principle of superposition

Note: Preparation of this paper was still in progress at the time of publication; final results will be presented at the symposium.

Construction Scheduling for Urban Freeway Renewal Projects: A Case Study

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ABSTRACT

The reconstruction of Interstate 235 (I-235) is the largest and the most expensive project in the history of the Iowa Department of Transportation. In order to accomplish this urban freeway renewal project, the Iowa Department of Transportation personnel need to know corridor-level information, such as durations, predecessors, and successors of construction projects, as they manage the design and construction process. They also need to know project-level information when communicating with the contractors, utility companies, and governmental agencies that are using detailed construction schedules. An appropriately customized corridor schedule, when updated consistently and distributed regularly, provides the necessary information. This customized schedule facilities management decisions and helps the corridor project keep within budget while keeping it on schedule. This paper discusses the corridor schedule development, how it has been customized, updated, and distributed, as well as the research results and recommendations. The reconstruction of I-235 corridor is used as a case project in this research to better develop a corridor scheduling system that can be used elsewhere for other urban freeway renewal projects.

Key words: corridor schedule—heavy/highway construction—Interstate 235—scheduling—urban freeway renewal project

INTRODUCTION

The reconstruction of Interstate 235 (I-235) is a budgeted $426 million project managed by the Iowa Department of Transportation (Iowa DOT). As one of the largest and most expensive road projects in the Iowa DOT's history, construction began in 2002 and is scheduled to be completed in 2007. The I-235 project is located near downtown Des Moines, Iowa (see Figure 1). The 14-mile corridor will either be rehabilitated with hot-mix asphalt or totally reconstructed and paved with portland cement concrete (PCC). Seventy-one bridges and 21 interchanges on this corridor need to be rebuilt, which involves about 175 separate construction contracts. During the 2003 construction season, approximately 10 concurrent projects are under construction.

The general construction timeline for the reconstruction is as follows:

- 2002 ~ 2004: Utility relocation, overpass bridge replacements, temporary paving, interchange reconstruction.
- 2005 ~ 2007: Mainline paving, mainline bridge widening (except Des Moines River Bridge, which is constructed in 2003).

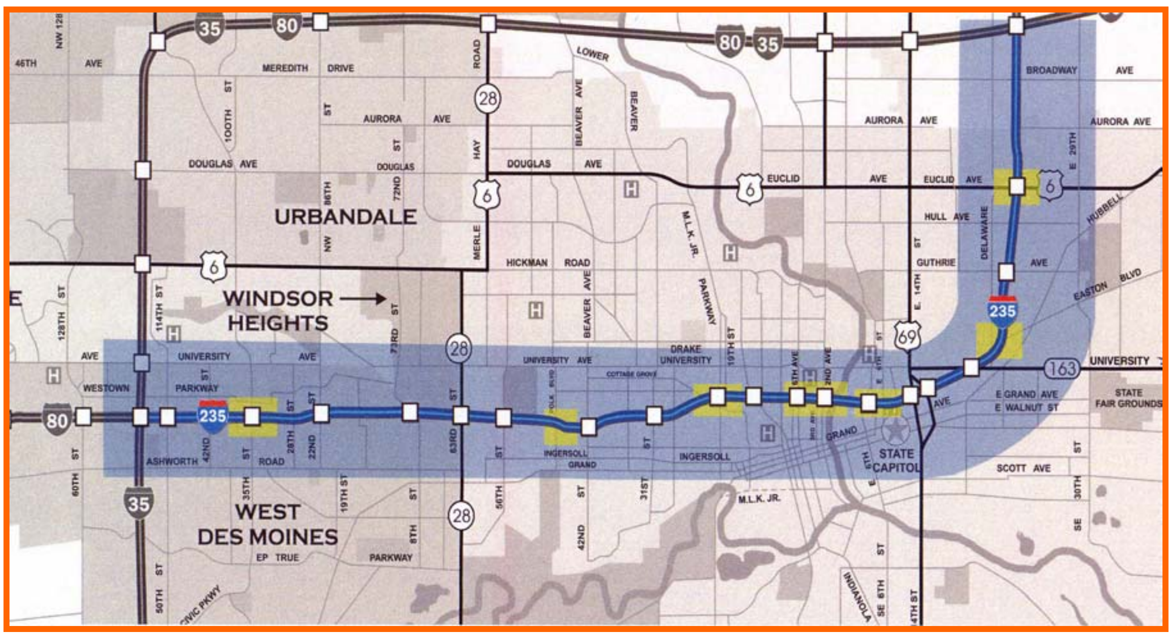


FIGURE 1. Map of I-235 Corridor (Des Moines Downtown Area)

The reconstruction of I-235 is a comprehensive project with multiple contractors and multiple tasks. The entire project needs to be accomplished in limited working spaces, within tight deadlines and budget. In order to accomplish this urban freeway renewal project, project participants need to know corridor-level information, such as durations, predecessors, and successors of proposed construction contracts, as they develop detailed plans and manage construction. They also need to know project-level information when communicating with the contractors, utility companies, and government officials who are using detailed construction schedules. An appropriately customized corridor schedule, updated consistently and distributed regularly, provides necessary information. This customized schedule facilities management decisions and helps the corridor project keep within budget while keeping it on schedule. Therefore, the objective of this research is to better develop a corridor schedule system that can be generally applied to other urban freeway construction projects that are administered in way similar to the I-235 project.

RESEARCH METHOLOGY

In order to accomplish the I-235 project, the development of a schedule that is able to provide an appropriate level of detail for the entire corridor is an important part of the research (*1*). Not only can this corridor schedule be used by the Iowa DOT and contractors for making decisions and controlling the work, but it can also potentially be used to help resolve conflicts when multiple resources and jobs (e.g., personnel, materials, and administrative effort) may be required at a given time. The working flow of the research is shown in Figure 2.

Since August 1999, Iowa State University (ISU) researchers have been working on the developing, updating, customizing, and distributing the I-235 corridor schedule. Researchers developed a conceptual schedule for I-235 based on input from Iowa DOT personnel. Then the schedule was further developed by adding logical relationships (see Figure 3). Since January 2002, the schedule has been refined to the extent that it can be updated and published regularly, which is an important part of the system in order to make it useful.

Developing the Corridor Schedule

In order to simplify and standardize the scheduling process, scheduling templates were developed for each major work category, including asphalt cement concrete/PCC paving, concrete/steel girder bridge, retaining walls, concrete box culverts, and utility work (*2*). When fully developed, the templates provide the following:

- identification of critical work activities that involve each major work category in determining construction duration
- identification of construction sequences between work activities and other considerations such as the lag time between the activities

A great time savings can be achieved as the templates are customized to any project and, in the case of a larger project, inserted into the larger project's overall schedule.

With the experiences built upon 55 field visits, the knowledge of the staging plan of each section, as well as the information obtained from the Iowa DOT regarding available state and federal budgets for upcoming fiscal years, logical relationships among separate projects (e.g., 35th Street Bridge construction and 35th Street Interchange construction) can be developed. Integrating these 175 projects, their durations, and relationships leads to the I-235 corridor schedule.

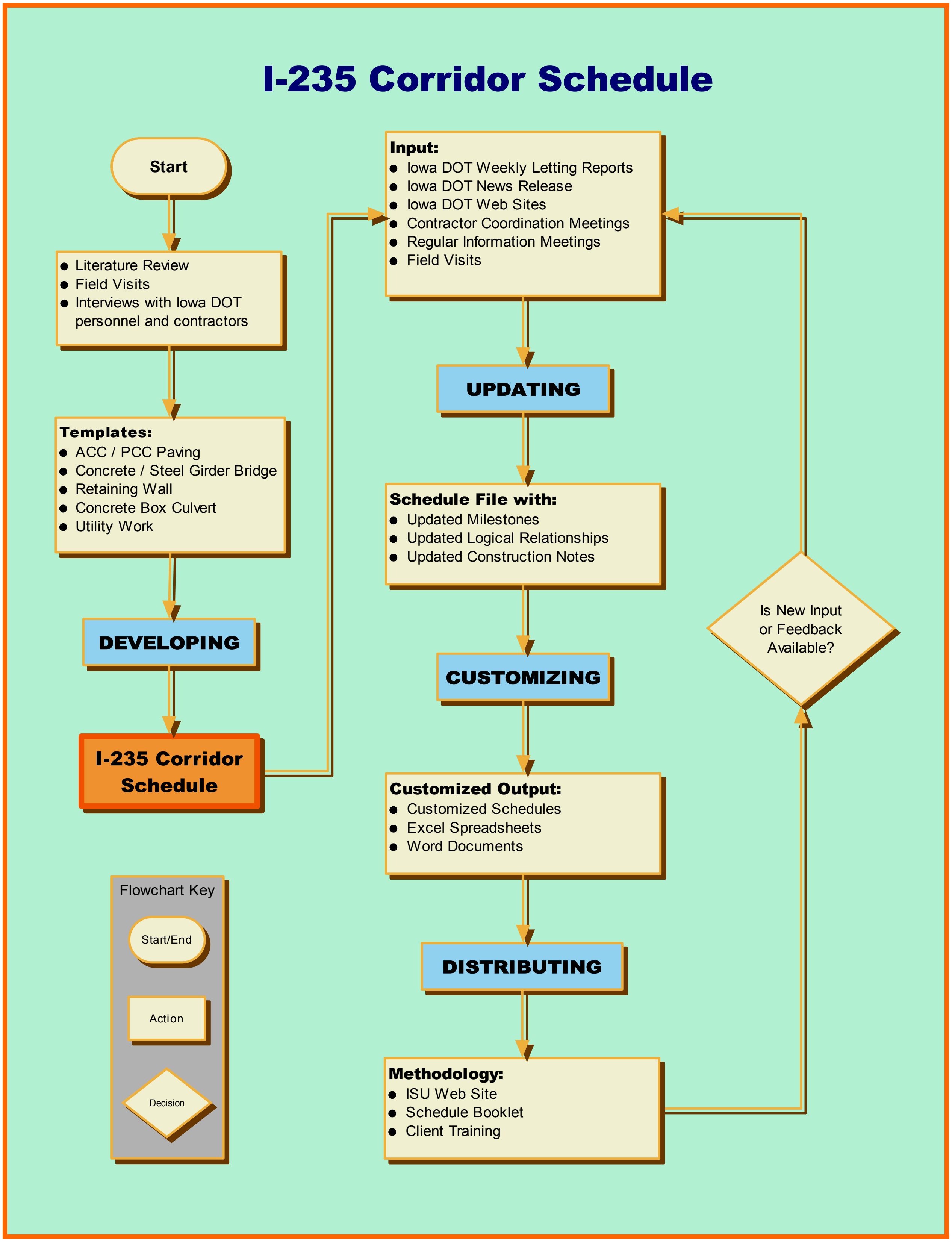


FIGURE 2. Flow Chart of I-235 Corridor Scheduling Work

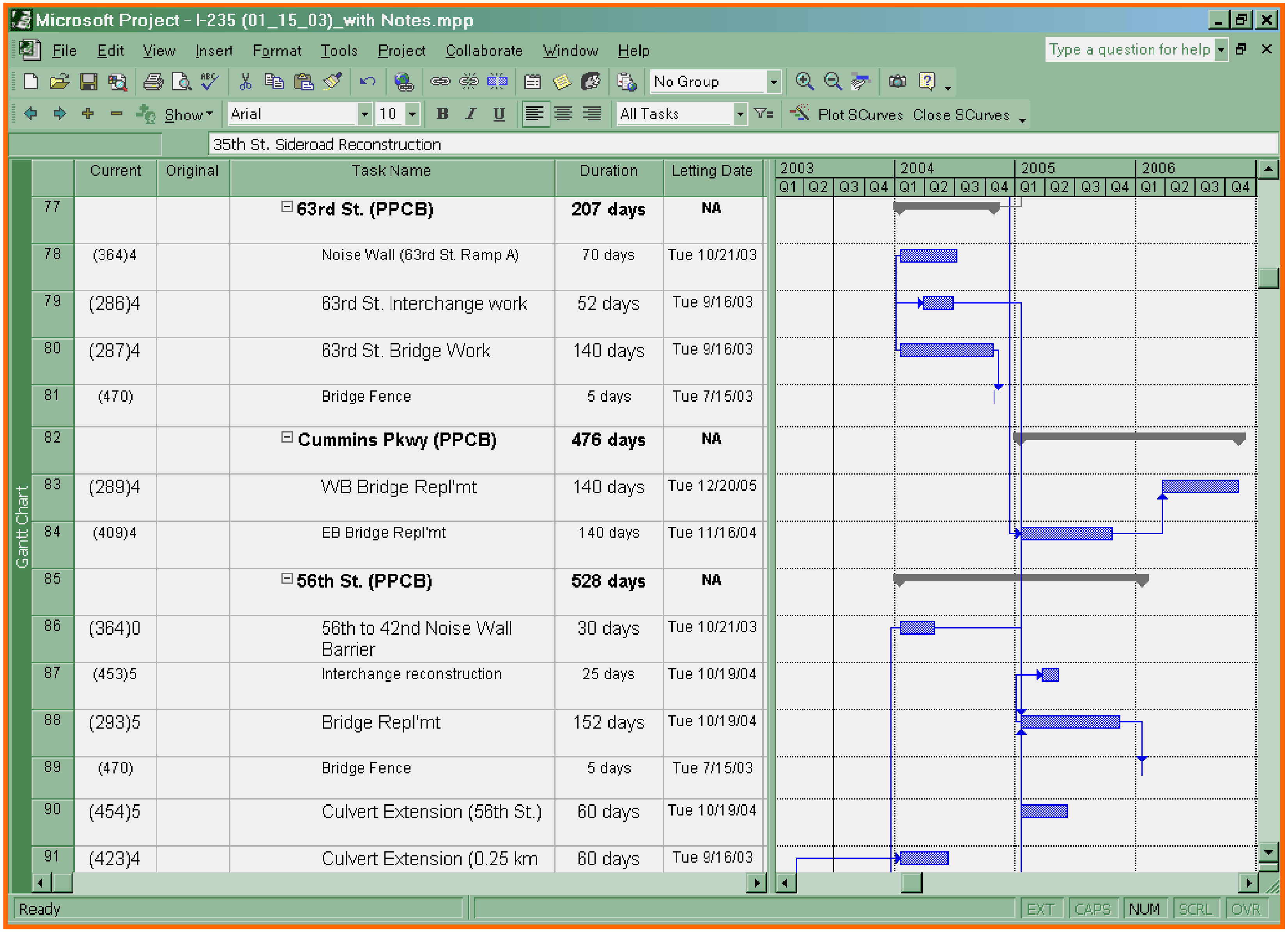


FIGURE 3. I-235 Corridor Schedule—Gantt Chart View

Updating the Corridor Schedule

In order to make the I-235 schedule an effective and valuable managing tool, it was updated every week since March 2002, a total of 63 times as of June 30, 2003. Relevant information about actual project progress is collected and recorded in the schedule file.

In a large project like I-235, it is difficult to collect information on project progress. Researchers developed a method of collecting information that involved reviewing Iowa DOT documentation, attending construction coordination meetings, and obtaining information from key people who have direct knowledge of the work.

From this weekly information collection effort, the following is abstracted: letting date changes; actual start/finish dates; and notes about changes and construction progress. Then milestones and logical relationships are updated, not only as data format, but also as construction notes recorded into the schedule's database (see Figure 4). If the comparison against the programmed schedule shows resource conflicts that can potentially affect meeting deadlines, users are informed.

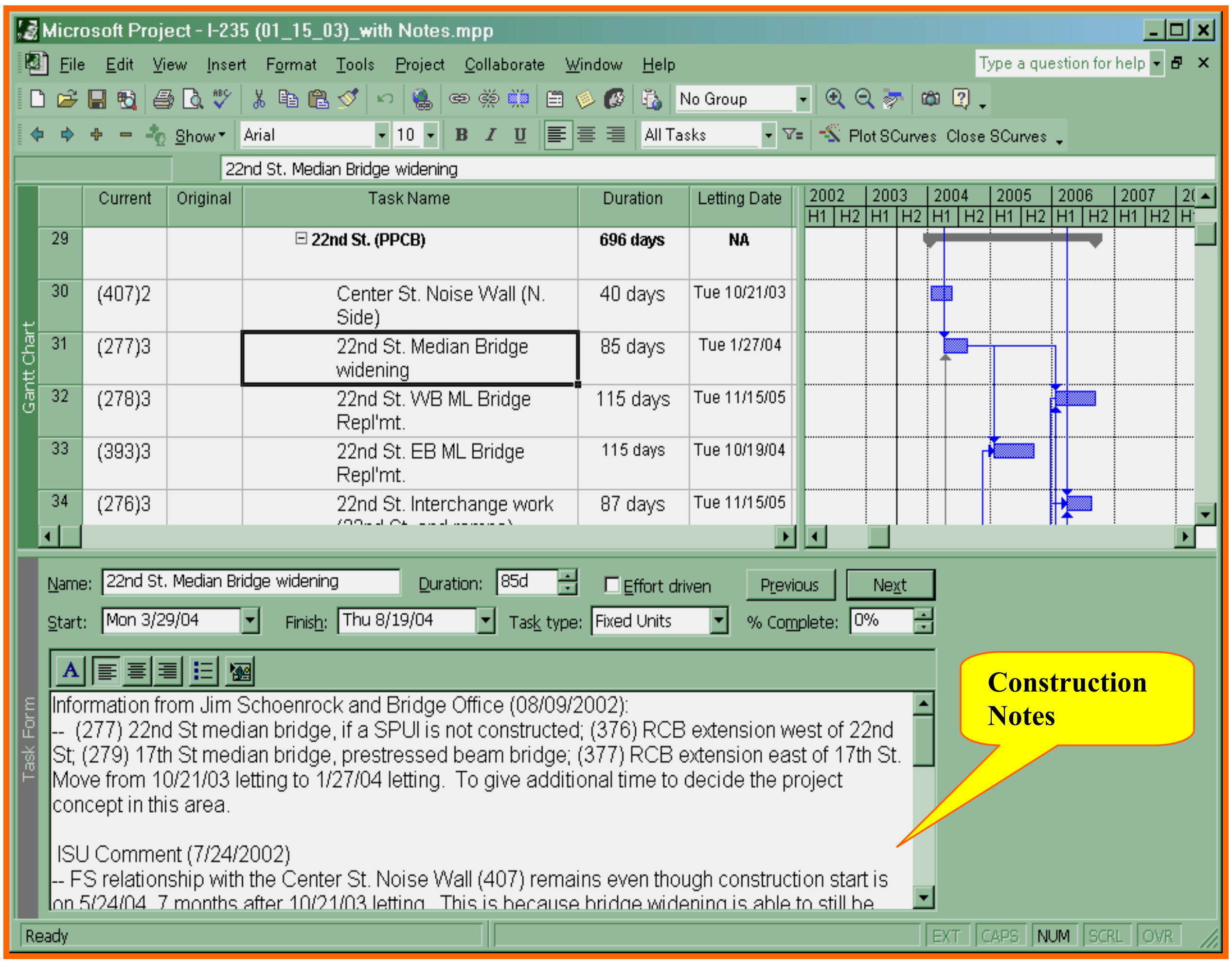


FIGURE 4. Updating the Corridor Schedule

Customizing the Corridor Schedule

Before distributing the schedule to users, it is important to customize the schedule according to the needs and expectations of various users:

- The Iowa DOT management team needs to view summary information, unencumbered with detail.
- Other project participants require detailed information concerning their areas of interest.

Using the many tools Microsoft Project provides, the research team is able to customize the corridor schedule to show the information that needs to be presented. For example, the schedule was customized so it could be used by Iowa DOT Office of Design for personnel management purposes (see Figure 5).

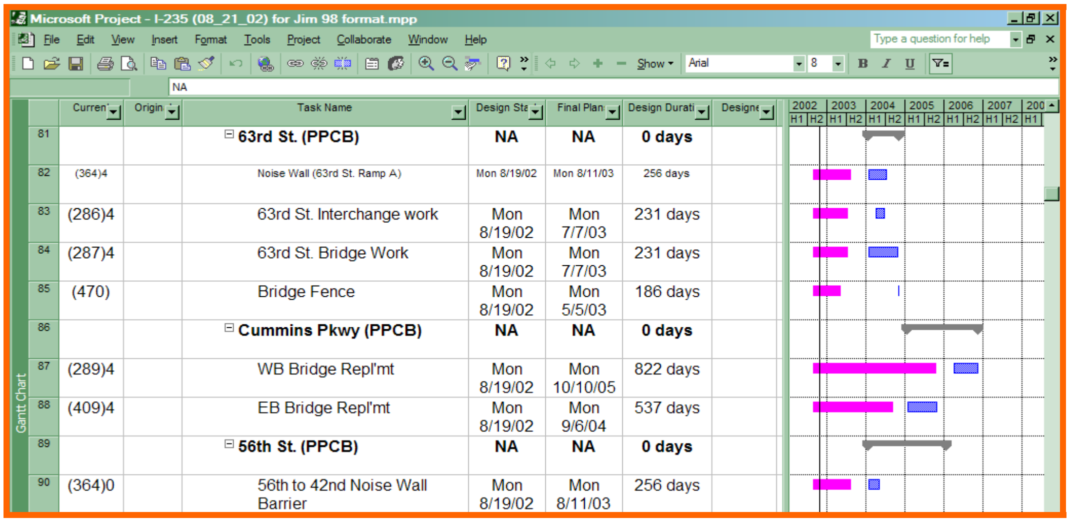


FIGURE 5. Customized Corridor Schedule for Iowa DOT Office of Design

Distributing the Corridor Schedule

When the success of the project depends on a group working together and interdependently, it may be vital that each user is able to communicate with one another in a timely manner. Users can waste valuable time when schedule distribution systems are slow or restrictive (e.g., if the schedule is not published as expected), or they don't know how to use the software, which is required to access the schedule.

Recognizing this, researchers publish the schedule via a website on a weekly basis (see Figure 6), publish schedule booklets (Figure 7) every three months, and provide training to users from time to time, as follows:

- *Corridor Schedule Publication.* Every Wednesday, the corridor schedule is updated and posted on the website (http://erl.cce.iastate.edu/I235). Notification of the update is sent out to every user via mail. Non-public information is protected by a user name and password.
- *Corridor Schedule Booklet.* The schedule booklet is a collection of summarized information from the scheduling work. It is published every three months (or less if there is a major change) and includes the following:
  - projects sorted by letting dates and project numbers (see Figure 8)
  - construction contact list (Iowa DOT and contractor contact information from ongoing projects)

- o corridor schedule
- o summary map (MicroStation map graphically shows project location, letting dates, plan turn-in dates, design numbers, etc.) (see Figure 9)

- *Client Training.* In order to make users of the schedule, researchers presented workshops to users regarding how to use the above products.

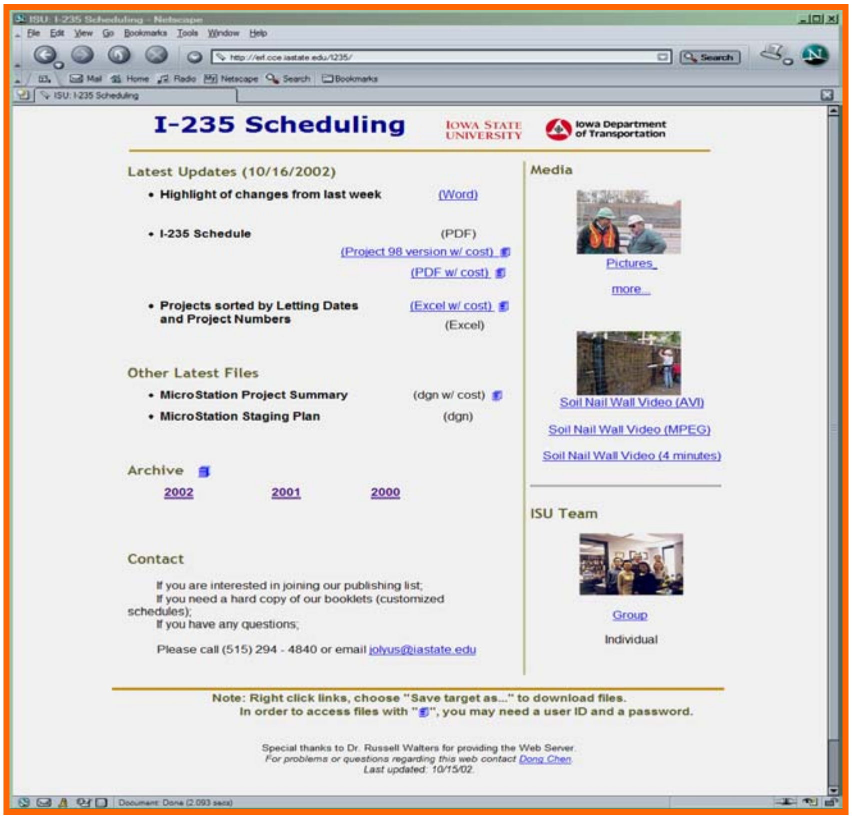


FIGURE 6. I-235 Scheduling Website

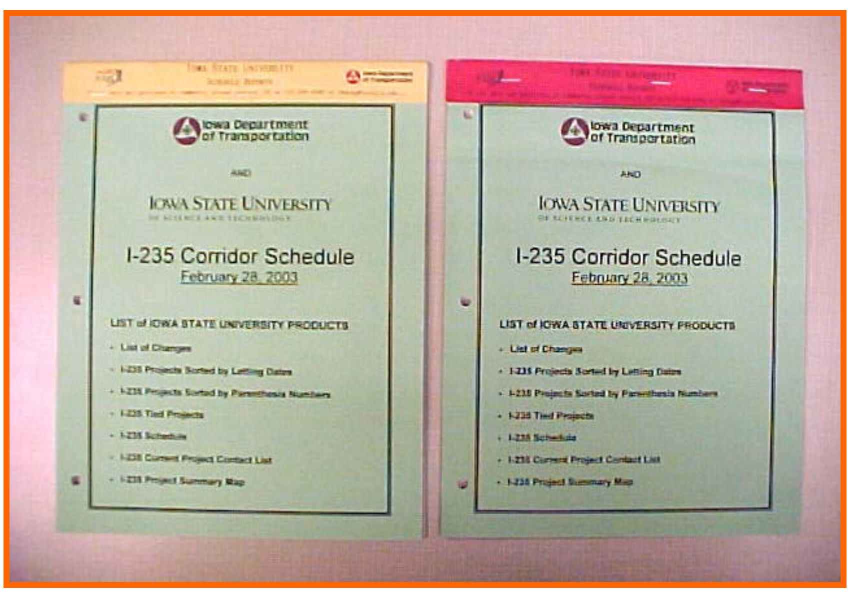
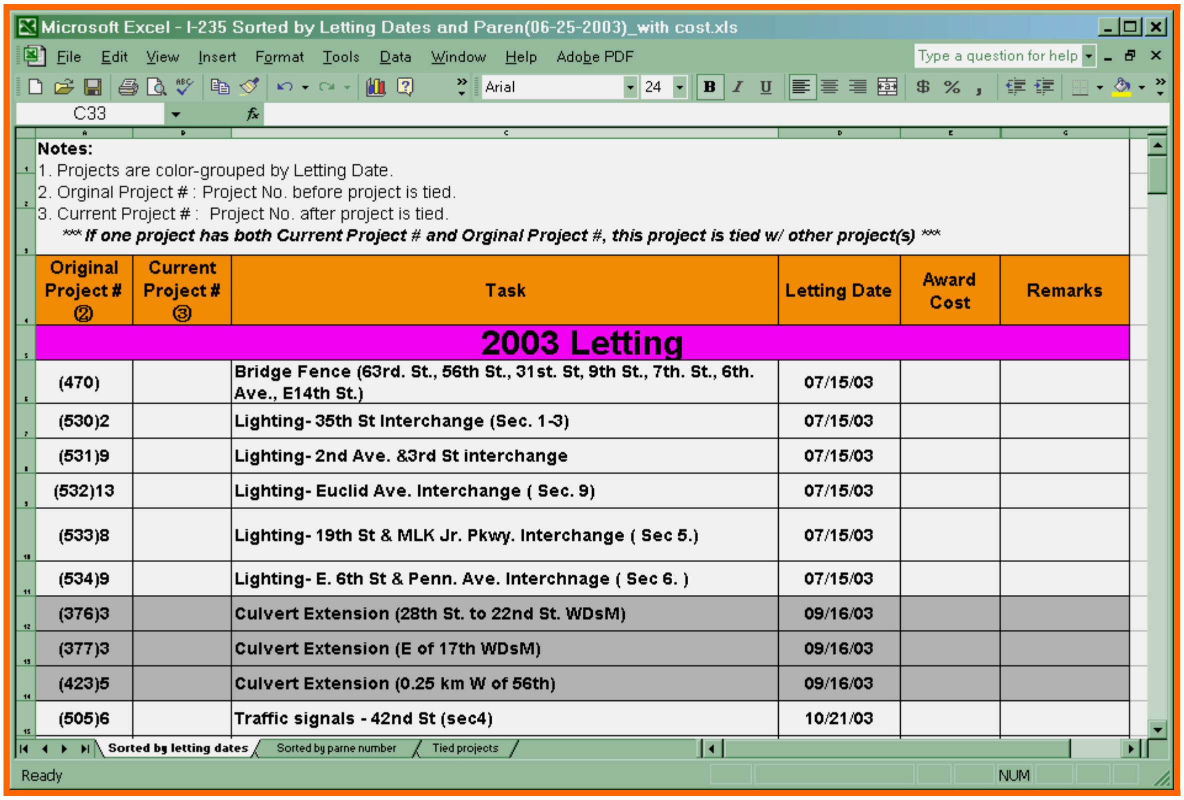


FIGURE 7. I-235 Scheduling Booklet


Microsoft Excel - I-235 Sorted by Letting Dates and Paren(06-25-2003)_with cost.xls

Notes:
1. Projects are color-grouped by Letting Date.
2. Orginal Project # : Project No. before project is tied.
3. Current Project # : Project No. after project is tied.

**** If one project has both Current Project # and Orginal Project #, this project is tied w/ other project(s) ****

Original Project # ②	Current Project # ③	Task	Letting Date	Award Cost	Remarks
2003 Letting					
(470)		Bridge Fence (63rd. St., 56th St., 31st. St, 9th St., 7th. St., 6th. Ave., E14th St.)	07/15/03		
(530)2		Lighting- 35th St Interchange (Sec. 1-3)	07/15/03		
(531)9		Lighting- 2nd Ave. &3rd St interchange	07/15/03		
(532)13		Lighting- Euclid Ave. Interchange (Sec. 9)	07/15/03		
(533)8		Lighting- 19th St & MLK Jr. Pkwy. Interchange (Sec 5.)	07/15/03		
(534)9		Lighting- E. 6th St & Penn. Ave. Interchnage (Sec 6.)	07/15/03		
(376)3		Culvert Extension (28th St. to 22nd St. WDsM)	09/16/03		
(377)3		Culvert Extension (E of 17th WDsM)	09/16/03		
(423)5		Culvert Extension (0.25 km W of 56th)	09/16/03		
(505)6		Traffic signals - 42nd St (sec4)	10/21/03		

Sorted by letting dates / Sorted by parne number / Tied projects

FIGURE 8. I-235 Scheduling Booklet—Project Sorted by Letting Dates

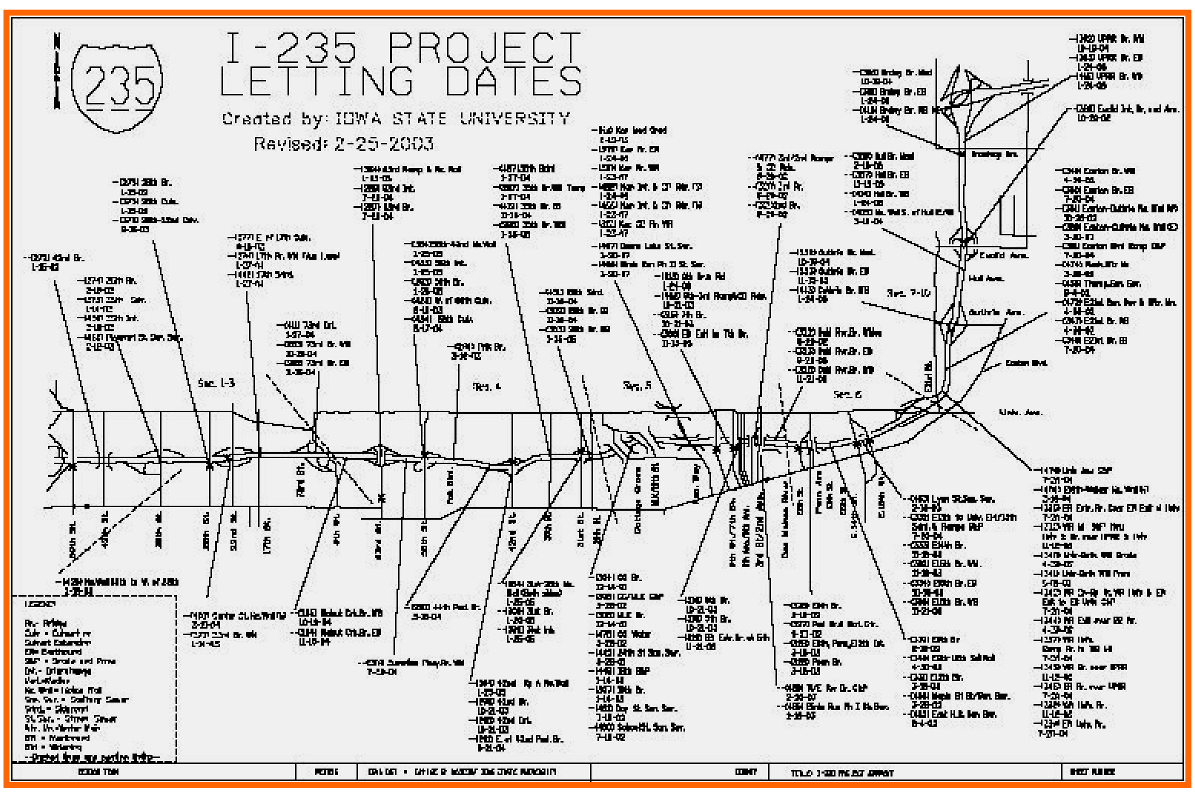


FIGURE 9. I-235 Scheduling Booklet—Summary Map

RESEARCH RESULTS

Over the past three years, the scheduling assistance to the Iowa DOT for the reconstruction of I-235 has been provided through a variety of activities. The following was accomplished:

- Assistance was rendered to the Iowa DOT personnel as they made management decisions and fine-tuned the I-235 project. Iowa DOT personnel are facing challenges in managing this project. Various assumptions are made after the review of limited information. Selections need to be made among options quickly. Such challenges are typical for an urban freeway renewal project. Detailed schedule reviews have resulted in significant savings. For example, according to researchers' advice, the Iowa DOT and the contractor adjusted the staging plan for Martin Luther King area, reducing detour construction and ensuring project completion in one construction season (*3*).
- Researchers helped fill in the gap between the Iowa DOT conceptual plans and contractors' detailed schedules (see Figure 10). Iowa DOT personnel develop conceptual plans that define the construction sequence and funding requirements. Contractors develop detailed schedules for each of their specific projects. How should the gap be filled between the Iowa DOT's conceptual plans and the contractor's detailed schedule? The flow of information is more efficient when there is a central repository to which information can be fed and extracted. The I-235 corridor schedule is designed to be such a repository, or database, which has an appropriate level of detail to bridge the gap between conceptual plans and detailed schedules. For instance, the logical relationships shown in the schedule connect corridor-level projects and can be used to show how several detailed contractors' detailed schedules fit together. Historical milestones of each task can be recorded as notes to the schedule; this helps both parties remember what happened without an extensive review of their own documents.

RECOMMENDATIONS

On the basis of research conducted by ISU, the following sets forth researchers' recommendations:

- In similar situations, the corridor scheduling system can be used on nationwide urban freeway renewal projects. America has been seriously under-investing in needed freeway and bridge repairs, and has failed to even maintain the substandard conditions it currently has (*4*). Substantial increases of urban freeway preservation and system expansion investment would be required to prevent both average physical conditions and operational performance from becoming severely degraded (*5*). The system developed by ISU provides a protocol for scheduling an urban renewal project. When analyzed individually, it can be easily applied to other urban projects.
- Productivity analysis of recent urban freeway projects should be performed. Production rates that the Iowa DOT currently has are more than 5 years old. Some of them came from rural projects. Researchers found that they are not always compatible with the I-235 corridor. From the beginning of this project, 12 bridges (including ramps) have been finished, four bridges are being constructed, and one paving project is underway. Historical data of these projects should be analyzed to obtain the latest production rates, which can then be used to provide more accurate scheduling services for the rest of the projects on I-235. Furthermore, these production rates can be standardized and thus be applied to other freeway reconstruction projects.

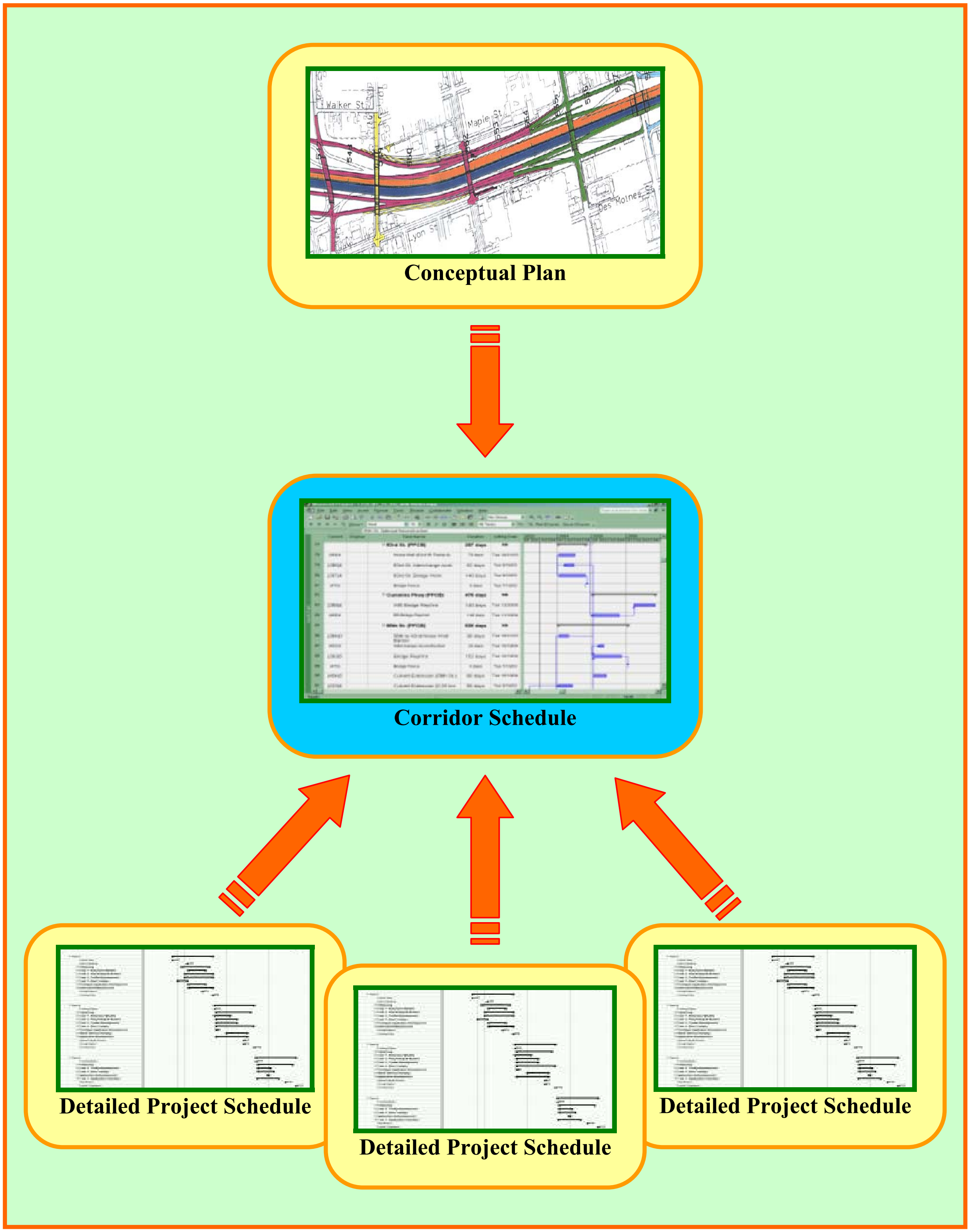


FIGURE 10. Function of the I-235 Corridor Schedule

ACKNOWLEDGMENTS

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Methodology to Measure the Costs and Benefits of Technology to Improve Hazardous Materials Transportation Safety and Security

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ABSTRACT

The Hazardous Materials Safety and Security Technology Operation Test is a project funded by the U.S. Department of Transportation's, Intelligent Transportation Systems Joint Program Office and the Federal Motor Carrier Safety Administration. The project is testing commercial off-the-shelf technology on vehicles transporting hazardous materials by highway. The goal of the project is to demonstrate the effectiveness of these technologies to enhance both safety and security with the goal of speeding up deployment within the industry. The independent evaluation will quantify the costs and benefits of implementing these technologies in the industry.

Key words: costs and benefits—deployment—hazardous materials transportation—safety and security

Proceedings of the 2003 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2003.

INTRODUCTION

Project Objective

The objective of the Hazardous Materials Safety and Security Operational test is to demonstrate an approach using commercial, off-the-shelf technological solutions to enhance both the safety and security of hazardous materials transportation by highway, with the goal of speeding up deployment by the industry. The evaluation methodology presented in this paper will quantify the benefits and costs of implementing these technologies in the hazardous materials transportation industry.

Background

Following the September 11, 2001, terrorist attacks on the United States, the U.S. Department of Transportation (U.S. DOT) was asked to identify areas within the transportation that were vulnerable to terrorist attack. Hazardous materials transportation was identified as a major area of concern. There are over 800,000 shipments of hazardous materials in the United States each day. These shipments range from high hazard shipments such as explosives and toxic by inhalation shipments, to small packages of corrosive or flammables. Nearly 300,000 of the daily shipments in the United States are petroleum products by truck, amounting to nearly 3 million tons (*1*).

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

Overview

This test is focusing on four different highway transportation scenarios are being tested each with 25 vehicles: (1) bulk petroleum transportation; (2) bulk chemical transportation; (3) less-than-truckload transportation; and (4) truckload explosives transportation. The scenarios were chosen based on the results of a hazardous materials risk and threat assessment that was conducted as the initial phase of this project, combined with a desire to test in different industry types. The risk ands threat assessment methodology was used to identify the types of materials that were of highest concern, as well as the most likely attack scenarios (theft of a material, interception/diversion and legal exploitation). Specific vulnerabilities were also identified during this phase of the project, and this was the basis for selecting the technologies within each scenario.

A wide variety of commercial off-the-shelf technologies are being tested. These technologies are in various stages of deployment within the hazardous materials motor carrier industry. They include wireless satellite and terrestrial communications with global positioning systems (GPS), enhanced digital phones, untethered trailer tracking, routing and geo-fenced mapping software, panic button, biometrics and smart cards, driver authentication with global login, electronic shipping documentation, intelligent on-board computers with vehicle disabling and cargo locking, and e-seals. The technologies being tested are grouped together into several packages within each scenario. This was done to address a wide range of vulnerabilities identified in the risk/threat assessment, and also to test several different cost tiers.

The test design is very important to the results that will be achieved. For example, the packaging of technologies into cost tiers will provide a wide range of information to the benefit – cost analysis. During this portion of the study, two areas will be analyzed. First, the macroeconomic/societal component of the analysis will look at the benefits to society from increased safety and security (through reduction of vulnerabilities) of hazardous materials shipments and compare that to the costs of deployment. Secondly, the study will consider the private sector benefit-cost ratio achieved through the use of technology to gain operational efficiency improvements.

The development of a methodology to evaluate the costs and benefits of these technologies presents some unique challenges. Traditional measures of safety risk (probability and consequence) cannot be directly transferred to address the security problem. Limited data on the frequency of terrorist events is the primary contributor to this problem. The general framework for looking at reductions in security vulnerabilities used looks at three factors; threat, vulnerability and the consequence when determining the overall costs of a given terrorist attack scenario.

Evaluation of the security benefits and safety benefits will be combined to conduct the macroeconomic analysis and the operational efficiency gains provided by the technology will used to provide to determine the benefits to the private sector. These are not the only areas that will be addressed by the evaluation, however. Additionally an overall cost assessment will be included in the evaluation as well as a survey of the use of technology in the hazardous materials transportation industry. Finally, the evaluation will consider customer satisfaction with the technology, institutional challenges that were identified during the test, an analysis of the security gaps and potential for deploying the technology.

Teaming Partners

The operational test is being managed led by FMCSA and funded largely by the U.S. DOT, Intelligent Transportation Systems Joint Program Office (JPO). The JPO is also funding the independent evaluation of the project. A multi - agency working group provides assistance on the project and includes the FMCSA, Research and Special Programs Administration, Federal Highway Administration, U.S. DOT Office of Intermodalism and the JPO.

Lead by Battelle, the operational test team includes the American Transportation Research Institute, Commercial Vehicle Safety Alliance, Total Security US, Qualcomm, Savi Technology and Biometric Solutions Group. Significant cost share is being provided by the team members in addition to the government funding for the project. There will be participation from six public sector agencies in four states (New York, Illinois, Texas and California), and the test will involve ten motor carriers and seven shippers. The independent evaluation is being conducted Science Applications International and Cambridge Systematics.

HAZMAT SAFETY AND SECURITY OPERATIONAL TEST – PLANNING AND DESIGN

The first nine months of the project involved the preliminary analysis, system design, and project planning necessary to conduct a major field test. This section of the paper outlines the results of these early activities. This includes the following: (1) a risk/threat assessment of hazmat transportation (*2*) to identify major vulnerabilities, and (2) a concept of operations (*3*) that identifies technologies, materials, and operational scenarios that will be tested.

Risk/Threat Assessment of Hazardous Materials Transportation

DOT initially identified 25 functional requirements for the test. These requirements were organized around the pick-up, enroute, and delivery phases of a hazmat shipment. However, before the developed the final approach to test these requirements, an assessment of the risks and threats of various hazmat operations and hazmat supply chains was conducted. The purpose of this effort was to ensure that the areas of greatest concern (operations with the highest risks and/or vulnerabilities) in hazmat transportation were being addressed by the functional requirements and technologies to be tested.

Figure 1 illustrates the assessment process developed to conduct this task. The assessment began with a broad look at hazmat transportation including factors such as: type of material, quantity of shipment, shipment frequency, type of operation, and routing. These factors are then considered from two different perspectives: intentional (e.g. terrorist) vs. unintentional (e.g. accidental) releases. Then reference components were established for intentional releases. The primary purpose for defining reference components was to organize, identify, and represent typical vulnerabilities. Each reference component was defined not to represent industry best practices related to security but to reflect the combination of vulnerabilities that can be readily found throughout industry.

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

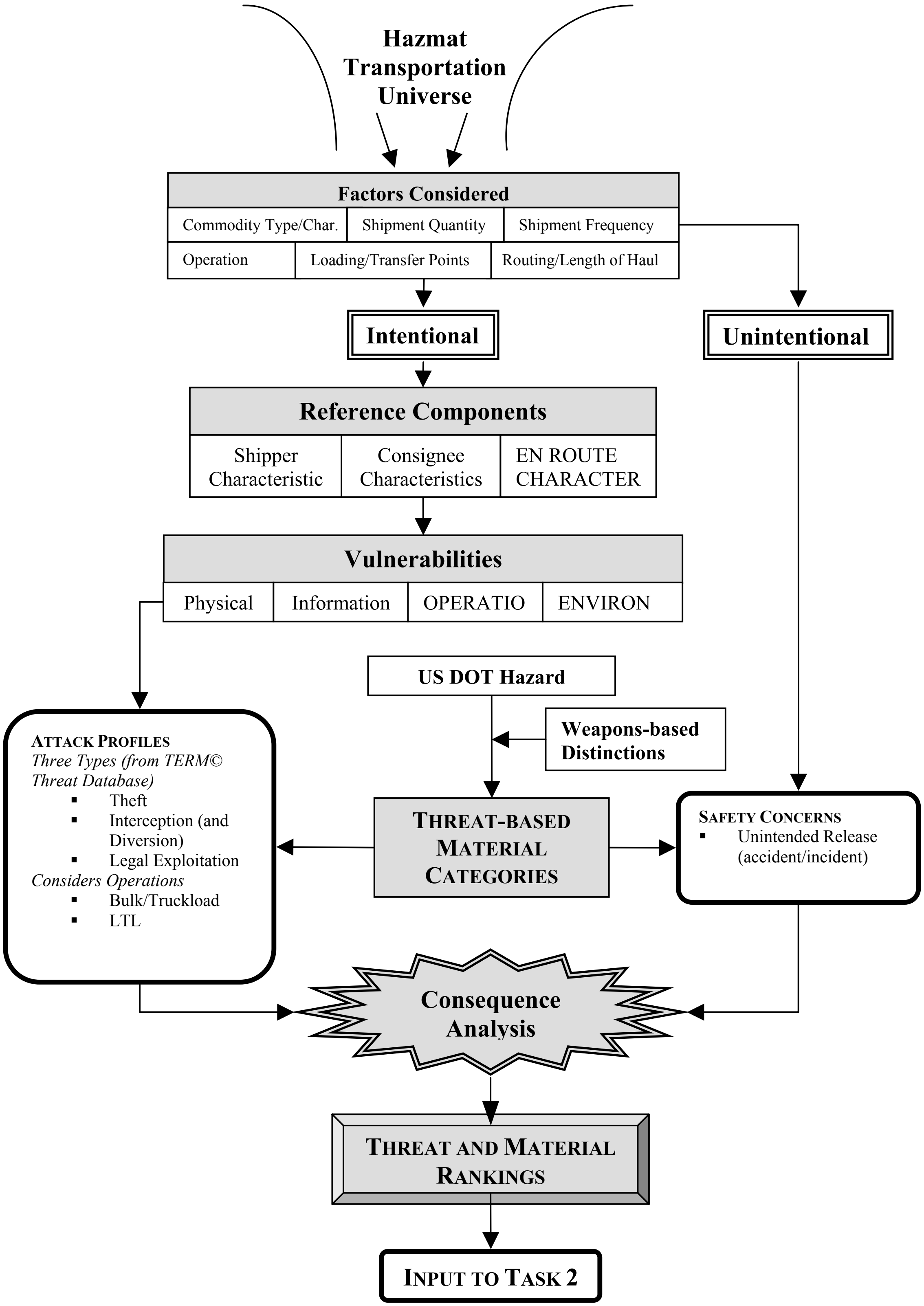

FIGURE 1. Assessment Process

Concept of Operation for Conducting the Hazmat Operational Test

Technology Selection

One of the key aspects in the selection of the technologies to be tested as part of this Field Operation Test (FOT) was the requirement to utilize "…commercial-off-the-shelf technologies that can be implemented rapidly by the motor carrier industry." In essence, this FOT is not a technology development activity, rather an integration of existing technologies that can address the specific functional requirements laid out in the SOW.

Listed below are the major technological components included in the FOT and a brief description of the functionality of each component.

Wireless Satellite or Terrestrial Communications (with GPS) Trucks will receive wireless tracking and communications systems, with integrated GPS working in conjunction with the dispatch systems that provide for load/cargo positions and status. The system will also include a Driver Interface Unit for two-way text communications. Positions are automatically displayed to the carrier's dispatcher at a regular frequency determined by the carrier.

Digital Phone (without GPS): This technology provides integrated work order assignment and status messaging between a carrier's dispatch and a driver using a low-cost digital cellular handset with specialized operating software. Store-and-forward guaranteed messaging ensures message delivery upon returning to digital cellular coverage areas. Along with messaging, ancillary services such as mapping and directions, for example, are also available.

Panic Buttons: The Panic Buttons will provide real-time emergency alert message notification by the driver to the dispatcher. An emergency alert message will be generated via the use of a panic button, which comes in two configurations:

a. A panic button mounted inside the vehicle to send an emergency alert.
b. A wireless panic button that can be carried by the driver to remotely send an emergency alert and/or use the remote panic button to disable the vehicle.

Driver Authentication: Driver authentication is necessary to make sure the only authorized drivers are operating hazmat vehicles and picking up hazardous materials shipments. This FOT will test two separate technologies designed to authenticate drivers.

Driver Authentication with Global Login: Similar to a username and password on a computer system, Global Login is an authentication feature of the Wireless Communications System being used for this test. Through the use of a driver login process, the login information (user id and password) that the driver enters into the truck-based interface is verified both locally (on the truck) and over the air using the wireless communication system. If this verification fails, various configurable alerts and resulting actions can be triggered, up to and including vehicle disabling with the aide of an on-board computer (if installed).

Driver Authentication with Biometric Identification: This technology will require having a biometric verification unit on the vehicle. This will be a customized system designed to satisfy the environmental and usage characteristics required for installation in a trucking rig. The biometric system consists of a Central Processing Unit (CPU) with proprietary firmware which controls an attached smart card reader and fingerprint scanner, and which performs biometric verification. The biometric system will be customized to communicate with the on-board computer.

Electronic Supply Chain Manifest (ESCM): The ESCM system provides technologies that allow positive identification of the person responsible for the cargo and tracking capabilities for cargo movement within a hazardous materials shipment. Combining biometric verification, smart-cards, Internet applications and the on-board wireless communications, the system insures proper chain-of-control for the hazardous materials throughout the lifecycle of a hazardous materials shipment. It also provides visibility into the location and status of the shipment to the shipper, carrier and consignee, thus enhancing both security and customer service.

Electronic Supply Chain Manifest system security is achieved using:

a. Biometric fingerprint readers to restrict unauthorized system access and validate driver identification. Biometric log-ins are required at all access points to create, modify, send, receive, or view data and information within the enclosed test system.

b. Smart Cards that integrate data encryption and biometrics to enhance security of the ESCM system. Encrypted smart cards containing shipper, cargo and driver data are used throughout the ESCM supply chain to transfer and validate essential supply chain information.

Intelligent Onboard Computers (OBC): The OBC will be integrated with the wireless communications and vehicle operating systems to allow a variety of security related functions, based on configurable input. The OBC can be used to control the disabling of the vehicle in a variety of means. These methods include blocking fuel, or sending proprietary system instructions via the wireless communications system directly to the vehicle's data bus. The primary mode of disabling for this FOT will be retarding the vehicle into a limp mode where the vehicle still has electrical power but little throttle response past idle. This unit will also be configured to shut the vehicle down if there is a loss of satellite signal strength (i.e., cut the feed cable). The driver also will be able to call the monitoring center and inform them that the vehicle needs to be disabled (in case of theft, for example). At that time the dispatcher could send an over-the-air command to disable the vehicle.

A cargo door lock that requires the driver to request authorization from the carrier's dispatcher to lock or unlock the trailer door will also be demonstrated. This lock is a rugged unit that is bolted to the inside door of the trailer. Using over-the-air communications, a message requesting the doors to be unlocked/locked is sent to the dispatcher. The dispatcher then sends a message to the vehicle OBC device, which sends a command to the door, allowing the driver to unlock/lock the cargo door.

Electronic Cargo Seals: This technology includes a cargo E-seals that automatically generates an alert if the seal is broken without proper authorization. The seal uses short-range wireless communications to interface with a mobile E-Seal reader (located in the vehicle). The mobile reader is interfaced to the on-board wireless communications device and the cargo "alerts" will be given to the driver and/or forwarded automatically to the dispatcher. These alerts will include the date, time and location where the seal was breached.

Routing and Geo-fenced Mapping Software: This technology will deploy specialized software that allows the operator to define a risk area or a route to monitor. An "electronic fence" is set around any given route or point on a displayable map. The dispatcher can define a risk area (e.g., the White House) and if the vehicle enters the risk area or deviates from its route, an alert is sent to the carrier's dispatch center. A safe-haven can also be setup as a geo-fenced area and notifications can be configured if a vehicle leaves the area.

The geo-fencing capability interacts with frequent positioning and the on-board wireless communications system. If the geo-fence application has received a security breach, the system will automatically increase the positioning reports to a configurable time interval.

Untethered Trailer Tracking: The trailer tracking subsystem provides trailer position information to the dispatcher on a regular basis. The collection of untethered trailer positioning information is accomplished through the installation of devices on the trailers. Through the use of various sensors, these devices monitor the trailer to which they are attached. In response to physical or temporal events, these devices will report details of the event, including position, time, status, and identity data. Using the tethered device, connect and disconnect events are captured and transmitted as alerts to the dispatcher. This will notify the dispatcher that a trailer has been connected or disconnected from the tractor.

Technology Tiers

It was recognized early on in the FOT that the unique operational characteristics of many of the hazardous materials carriers around the country would not lend to a full-scale deployment of all the technologies described above on every vehicle. While it may be prudent (and the market may bear the cost) to deploy more technologies on certain types of shipments (i.e., explosives), other carriers operate in a situation where the marginal cost of deploying some of these technologies in their vehicles would be prohibitive. To represent these concerns of the market, the FOT team has separated the various technology components into six technology tiers, ranging from a low-end cost of approximately $250 per vehicle to a high-end of approximately $3,500 per vehicle. Table 1 provides a brief summary of each technology tiers.

TABLE 1. Technology Tiers

Tier (Cost)	Description
1 ($250)	Include a digital cellular phone with pickup and delivery software with on-phone/on-board directions/mapping. This option would also include on site vehicle disabling with the wireless panic remote. This would not be able to send a panic message but would give the ability to shut it down remotely. This would not include positioning until position location is turned on to national networks.
2 ($800)	Includes terrestrial communications with in-dash panic button.
3 ($2,000)	Includes satellite communications with an in-dash panic button and Global Login.
4 ($2,500)	Includes all of what is in tier 3 but adds the additional OBC. The other variant includes satellite communications with an in-dash and wireless panic button with Biometric authorization, and E-manifest.
5 ($3,000)	Includes satellite communications with an in-dash and wireless panic button with Biometric authorization, E-manifest and an additional OBC. The other variant is swapping the OBC for an untethered trailer-tracking device.
6 ($3,500)	Includes satellite communications with an in-dash and wireless panic button with Biometric authorization, E-manifest and E-Seals.

The price estimates by tier reflect only the hardware installed on the truck in commercial quantities. It does not reflect the price of servers and dispatch systems amortized over the number of vehicles since this can vary widely depending on customer set up. In addition, the price estimates reflect the cost of an initial install (assuming no technology previously installed on the truck).

Scenario Development

The final step in developing the concept of operations for the FOT was to match up each technology component with a testing scenario. The scenarios were developed to address the functional requirements in the DOT's request for proposals, the threats and vulnerabilities identified in the threat/risk assessment. The overall goal of the FOT was to test technologies installed in 100 vehicles. Each scenario will test a total of 25 vehicles, with various combinations of technology installed on each vehicle. In selecting the scenarios, attempts were made to match the shipment types – bulk, less than truckload (LTL), and truckload (TL) with specific hazardous commodities. Scenario 1 will involve the delivery of Class 2 Flammable Gas and Class 3 Flammable Liquids in the bulk delivery environment, Scenario 2 will involve the delivery of high hazards in the LTL environment, Scenario 3 will bulk chemical (Class 2.2 and Class 3 with inhalation hazard) delivery vehicles, and Scenario 4 will involve the truckload of Class 1.1–1.6 Explosives. Table 2 provides a summary of each scenario and the technology components to be tested by scenario.

TABLE 2. Technology Components by Scenario

Scenario	Description	Technology Components
1	Bulk Fuel Delivery	• Wireless Satellite Communication • Global Login • In-Dash Panic Button • Wireless Panic Button • Digital Phone • Terrestrial Communication • On-Board Computer
2	LTL High Hazard	• Wireless Satellite Communication • Global Login • In-Dash Panic Button • Wireless Panic Button • Terrestrial Communications
3	Bulk Other	• Wireless Satellite Communications • Biometric Authentication • In-Dash Panic Button • Wireless Panic Button • Electronic Supply Chain Manifest
4	Truckload Explosives	• Wireless Satellite Communication • Biometric Authentication • In-Dash Panic Button • Wireless Panic Button • Electronic Supply Chain Manifest • On-Board Computer • Wireless Electronic Cargo Seal • Geo-Fencing • Untethered Trailer Tracking

INDEPENDENT EVALUATION

Purpose of the Evaluation

The purpose of the evaluation (*4*) effort is to independently assess the test methods for leveraging technology and operations to improve hazmat transport security, safety, and operational efficiency. As such, it is expected that technologies will be demonstrated that result in decreases in the existing vulnerabilities of hazmat shipments to terrorist activities. To achieve this goal, the technologies will be combined with changes in shipper/carrier operations and tested across four distinct hazmat operational scenarios. The evaluation will also focus on safety and operational efficiency impacts.

Cost-Benefit Analysis

The three main evaluation impact categories examined by the evaluation team will be security, safety, and operational efficiency. These impact categories will then feed the benefit-cost analysis according to macroeconomic/societal (macro) public sector benefit-cost results (stemming from security and safety benefits) and microeconomic/industry (micro) private sector benefit-cost results (derived from operational efficiency improvements). The evaluation team will examine test technology systems users' perspectives in a Customer Satisfaction Study. An Institutional Challenges Study will document any information and communication improvements necessary to facilitate implementation of these test technology systems on a larger scale. The macro/societal and micro/industry benefit-cost measurements analysis will determine the following:

- Are the industry operational efficiency benefits significant enough to drive widespread industry deployment of test technology systems, or their equivalent?
- If not, are the macro benefits large enough to warrant government subsidization/new regulation to bring about wide-scale national deployment?

Evaluation Methodology

The evaluation team will measure the impact of technology solutions on the security, safety, and operational efficiency of hazmat movements from shipper to en-route transport to final delivery. The FOT will be administered by separating the test into four operational scenarios to allow each scenario to address a distinct segment of the hazmat industry. In some cases, the same technology will be tested in each scenario. In other instances, the testing of technologies will be limited to specific scenarios.

Each scenario will deploy a unique set of technology solutions to account for the unique operational characteristics for a particular sector of the hazmat market. The selected technological solutions for each scenario will seek to improve security, safety, and operational efficiency at several cost levels, depending on the comprehensiveness of the deployed technology set. The four scenarios were all scrutinized against security risk profiles that categorize and prioritize risk based on: the potential tactics terrorists might use; the most likely hazardous materials that could be involved; and by the type of shipment – bulk/truckload or less-than-truckload (LTL). The rationale for this risk analysis was to determine potential security gaps that might exist for each scenario. See Figure 2.

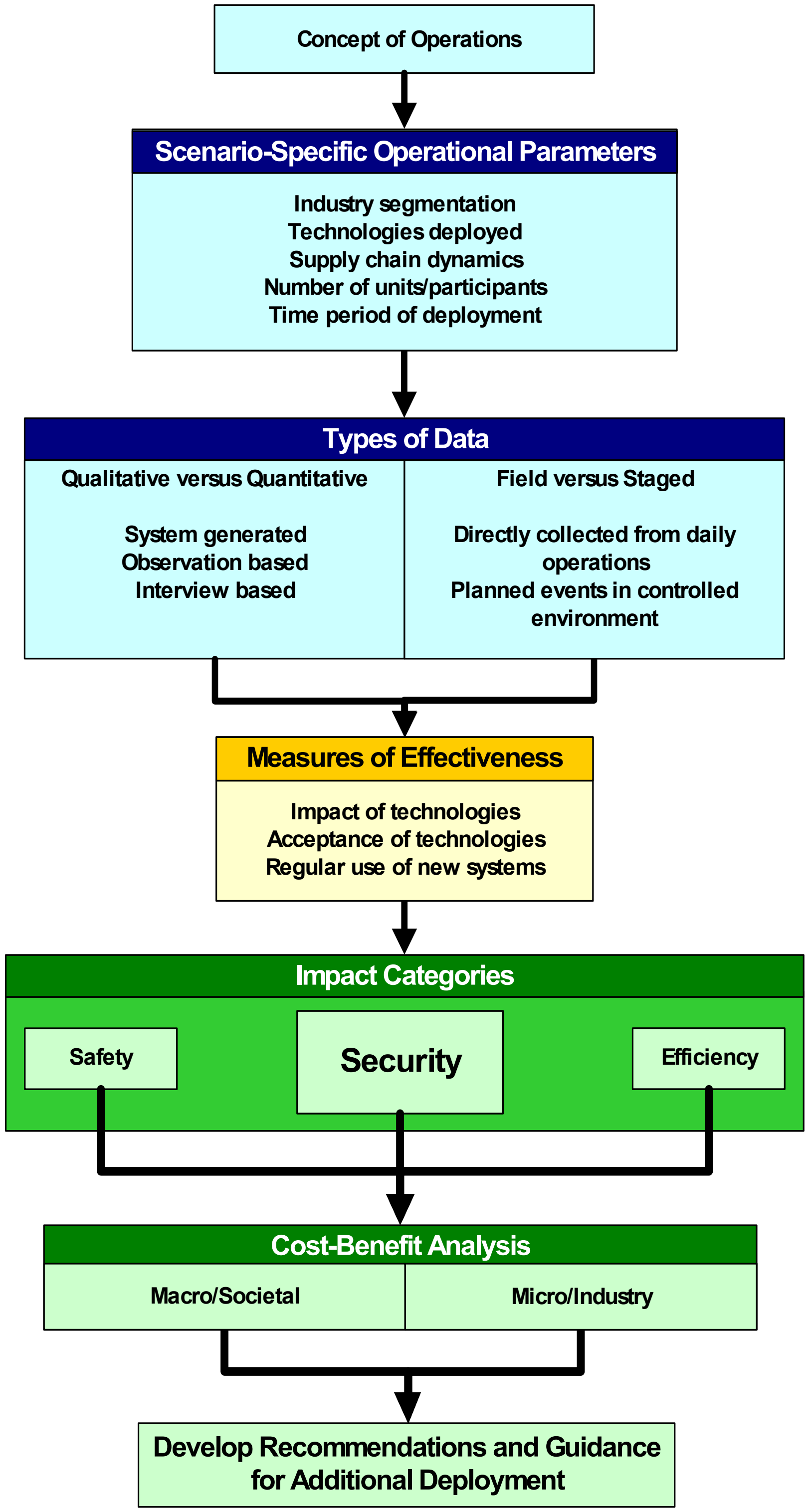

FIGURE 2. Evaluation Framework

Data Collection

Two types of data will be collected to support the technology-based and system-based evaluations: qualitative and quantitative. Qualitative data will be collected via on-site observations and personal interviews during the FOT. Interview guides will be designed and developed to collect data on operational effectiveness, customer satisfaction, and institutional challenges. For example, drivers will be asked about the ease of use of the various technologies, and how by adding the technology in question impacted their daily operations. These data will be critical for documenting the participant acceptance of the technologies. Also, if the quantitative data collected falls short of providing the necessary volume of data points for conclusive findings, the qualitative data will be available to verify and confirm the findings.

The quantitative data will be collected through primarily through the QualComm system-generated archived reports, which the operational test team will make available to the evaluation team on-line in a secure manner. The evaluation team will work closely with the Operational Test Team to establish types of data and criteria for archiving data to ensure these data are available. Additionally, data archiving of event logs will provide ongoing data collection of use and performance of technology applications throughout the FOT. For example, activating a panic button will generate a log containing time, date, and location of the event, which can be linked to subsequent response activities by the dispatch.

Evaluation Study Areas

Eight evaluation study areas where identified to frame the relevant issues, drive the data collection effort, facilitate the analysis and produce meaningful recommendations at the conclusion of the test period. Not all evaluation study areas will be applied to each of the four operational test scenarios.

Evaluation Study Area 1: Security Benefits Assessment and Evaluation Study Area 2: Safety Benefits Assessment will derive the information necessary to calculate the macro/societal benefits portion of the benefit-cost analysis. The security benefits are going to be measured primarily in terms of the measured reduction of the vulnerability, keeping in mind the formula for vulnerability assessment: Threat x Vulnerability x Consequence = Cost.

Threat is a function of terrorist aims and operating procedures and is not impacted by the technologies applies in this test. This factor will be based on the work completed during the initial threat/vulnerability assessment. The reduction in the vulnerabilities will utilize an expert panel that will evaluate the data elements generated during the test, including changes in response time, reliability of panic buttons and other factors. Consequences will then be taken from the threat assessment document to complete the equation.

The safety benefits assessments will utilize traditional risk assessment methodology developing probabilities and costs will be derived from existing databases and risk assessments. The basic formula is Probability x Consequence = Cost.

Evaluation Study Area 3: Operational Efficiency Assessment will provide the information to determine the private sector micro/industry benefits derived from the use of the technology. Return on Investment (ROI) will be examined by considering the benefits to a company of they were deploy and of the technologies in the test, and will look at the component technologies and the integrated systems being test. The ROI = Total Benefits Achieved divided by Total Investment Costs.

Evaluation Study Area 4: Cost Assessment will provide current test system expense measures as well as future costs under full HAZMAT industry deployment

Evaluation Study Area 5: Industry Technology Survey will explore viable, commercial alternatives with similar functionality to the test technologies.

Evaluation Study Area 6: Customer Satisfaction will provide specific user perspectives on the test technology systems.

Evaluation Study Area 7: Institutional Challenges will identify information and communication barriers that need to be alleviated to ensure full cooperation between system users.

Evaluation Study Area 8: Deployment Potential Assessment will determine if industry operational efficiency benefits are at a level to drive widespread HAZMAT industry deployment of test technology systems or their equivalent, or if the macro benefits are enough to cause government subsidization or implement new regulation to bring about wide-scale HAZMAT national deployment.

The final report will present the findings, conclusions, and recommendations from the independent assessment, and is expected to include results for the following areas:

- Identify the effectiveness of the operational test technologies to improve security throughout the HAZMAT movement chain.
- Identify safety benefits by assessing the effectiveness of the operational test technologies to improve safety throughout the HAZMAT movement chain.
- Identify operational efficiency benefits by assessing the effectiveness of the operational test technologies to improve operational efficiency throughout the HAZMAT movement chain.
- Detail costs for individual test system components and integrated HAZMAT systems leading to a cost analysis that estimates the cost of deploying these technologies (both individual components and integrated systems) throughout the freight industry.
- Present a relevant benefit-cost analysis for individual test components and integrated test systems at both a micro-level focusing on industry operational efficiency gains, and a macro-level focusing on improved security and safety benefits to society.

CONCLUSIONS AND PROJECT STATUS

Through the process of conducting the initial risk and threat assessment and developing and evaluation plan, concept of operations and system design, there are several important lessons that have been learned. First, conducting a risk and threat assessment to set the stage for the technology testing portion of the project was critical to the success of the project. This assessment drew a roadmap for the rest of the test by identifying the threats and vulnerabilities that existed, and which of those could be addressed by a technological solution. Secondly, the risk and threat assessment led the team to package the technologies in new and unique ways to address a wide range of vulnerabilities to gain the maximum security benefit as well as to test a variety of business types. Finally, in addition to packaging these technologies to maximize the security benefits, the packaging of technologies in different cost tiers will provide a robust evaluation that could not have been expected without a tiered approach.

Currently final preparations are being made for running a single prototype vehicle for a period of one to two weeks during the month of July 2003. This prototype will test two of the four scenarios that are part of the overall test, but will include all technology combinations that will be used in the four scenarios. Following completion of the prototype preparations will begin for full scale testing to begin in August of 2003. Each scenario will run for a period of approximately six months. Analysis results and final reports will be completed in the summer of 2004.

ACKNOWLEDGMENTS

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Access Management Techniques to Improve Traffic Operations and Safety: A Case Study of a Full vs. Directional Median Opening

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ABSTRACT

Access management applications, which could effectively be used to improve traffic operations and safety, are getting increased popularity. While there are some studies providing important information on various access management methods and techniques, questions still remain surrounding the effects of specific access management treatments on roadway operations and safety. This study provides the results of a case study in one of such areas, where the operational and safety characteristics of a full median opening are compared with those of a directional median opening, in the form of a before-and-after study.

Field data were collected at the site by using video camera technique. Before period consisted of one week of field data where the intersection operated as a full median opening. The median was then converted into a directional median opening by using temporary physical barriers and field data were collected again in the same way for another week. Only the data collected during daytime, under good weather conditions with no special events were considered in the analysis.

Operational characteristics were measured in terms of weighted average delay and weighted average travel time experienced by the left turning vehicles from the stop controlled approach (driveway), where safety performance was measured in terms of number of conflicts and rate of conflicts. According to the findings, the total weighted average travel delay was significantly reduced after the median opening was made to function as directional. However, changes in the travel times were not statistically significant. The average number of conflicts per hour was reduced by almost 50 percent whereas conflict rate per thousand involved vehicles was also significantly reduced. Additionally, the severity of conflicts measured subjectively was also found to be reduced during the after time period. Accordingly, both traffic operations and safety situations were found to be improved when the full median opening was converted to a directional opening.

Key words: access management—medians—traffic operations and safety

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INTRODUCTION

Access management helps achieve the necessary balance between traffic movement and property access by careful control of the location, type, and design of driveways and street intersections *(1)*. Applications in the area of access management are getting increasingly popular in many of the states throughout the United States. While there are some studies providing important information on various access management methods and techniques, questions still remain surrounding the effects of certain specific access management treatments on roadway operations and safety. One such area is operational and safety characteristics of full median openings versus those of directional median openings.

Current practice adopted by the Florida Department of Transportation (FDOT) is to have directional median openings on major arterials where the design speed is greater than 40 mph. Accordingly FDOT is considering the replacement of some of the full median openings with directional median openings, which sometimes receives criticism from the general public and the abutting commercial developments. Lack of more quantified results and documentation available for presenting the benefits of directional median openings in such situations was evident at public hearings, which was the main motivation towards this project. This paper describes a case study on the subject in the form of a “before and after” study, which was a part of a bigger project in which right turn followed by U-turn from driveways was considered as an alternative to direct left turn from driveways.

The main objective of this case study was to compare safety and operational impacts of converting a full median opening located on a major arterial into a directional median opening. As a result of this conversion left turning traffic from the driveway was forced to make right turns and then U-turns at the next available median opening or signalized intersection.

METHODOLOGY

The Site

The site considered in this study is located on U.S. 19 and 115th Avenue, in Pinellas County, Florida. Layout of the study site is illustrated in Figure 1. U.S.19 is a major arterial oriented in the north-south direction, with three and four lanes on the southbound and northbound, respectively. The northbound and southbound lanes are separated by a raised median. The posted speed limit on this segment of the road is 55 mph. This site was the subject of a geometric improvement related to left-turn movements. Initially, during the before period, the median was a full median opening that allowed vehicles to turn either to the left directly or to make the right turn first and then make a U-turn at a median opening located 400 feet from the street in order to travel northbound. The median opening was approximately 120 feet in length, which allowed three or more vehicles to wait on the median storage while impending the movement of other vehicles. Then, the median opening was closed and converted into a directional median opening, so that vehicles departing from the side street could only turn right. A descriptive analysis of the traffic conflict data collected at this site was performed because this site allowed the possibility to examine and evaluate the implications of changing a full median opening to a directional median opening in the frame of a before and after study *(2)*.

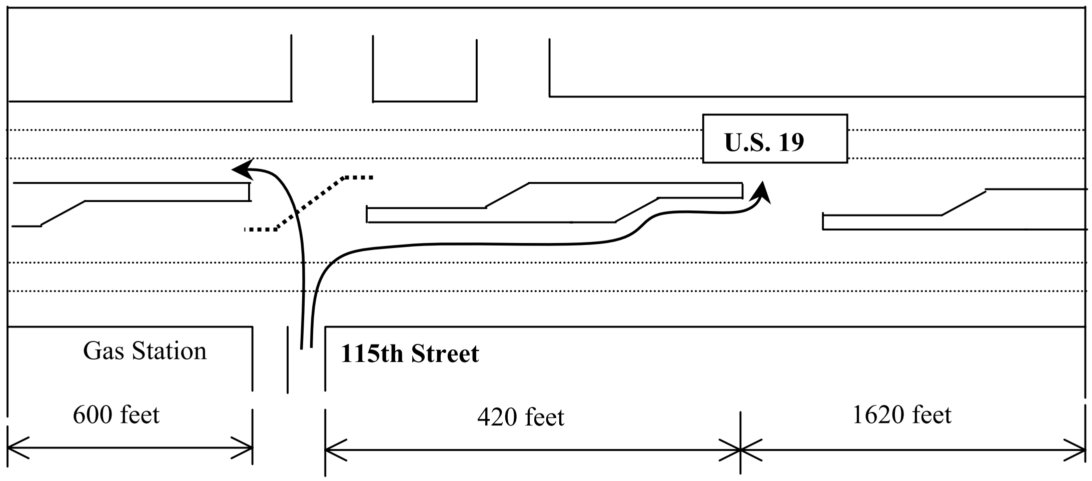

FIGURE 1. Layout of the Study Site: U.S. 19 at 115th Street

Change

During the before period, the opening at the location was a full median opening allowing all possible movements. Accordingly some of the left turning vehicles made direct left turns while some other drivers selected indirect left turns. During this time field data was collected for a period of one week. The median was then converted into a directional median opening by using temporary physical barriers and field data were collected again in the same way for another week. This time period provided the information on the after period in which case the left-turning drivers coming from the driveway were expected to do it indirectly.

Data

Operational and safety evaluation of the two types of median openings were conducted by using the field data collected during before and after periods. All traffic movement data required for the operational analysis and conflict data required for the safety analysis were collected by using 3video cameras covering the whole intersection and the weaving distance. Cameras were installed at a height of approximately 15 ft above the ground so that the vehicles traveling on middle or outer lanes would not cover the vehicles traveling in the inner lane. Two reasons supported the idea of using video cameras. First, it would be extremely difficult to manually record traffic conflicts, exact timings and traffic volumes at the same time because observers would have to track and identify the maneuver of each vehicle when it departed from the side street. Second, on-site observations could not easily be verified, which is of especial consideration when observers have to record several different types of conflicts *(3)*.

Traffic volumes on the arterial were recorded by using an automatic data recorder ADR-1000 from Peek Traffic™ installed on the pavement. Right turn, direct left-turn, left-turn in, and right turn plus U-Turn maneuver volumes were obtained reviewing the videotapes at the laboratory. All cameras and the counter were synchronized before the start of the data collection, which eliminated tedious matching of vehicles, especially when 3 or 4 cameras were involved simultaneously.

Data were collected during weekdays under normal traffic conditions, good weather, and dry pavement conditions. Weekdays in this study were considered from Monday through Thursday where normal conditions were expected to prevail. Also, data was collected during peak and non-peak periods. The morning peak hour was considered between 7:00 AM and 9:00 AM, and the afternoon peak period was considered between 4:00 p.m. and 6:00 p.m.

Analysis

Traffic and conflict data obtained through the field data collection were analyzed to compare operational and safety effects during before and after time periods, which represented full and directional median openings, respectively.

Operational Analysis

Traffic operational comparison of the two types of median openings was carried out by using two parameters, total delay and total travel time of left turning vehicles from the driveway *(2).* Total waiting delay of the direct left turning vehicles consisted of delay at the driveway and the delay at the median opening. Total waiting delay for indirect left turns at the directional median opening consisted of delay at the driveway and delay at the downstream median opening where the U-turn was made. Total travel time was obtained by adding the total running time to total delay. Since the before period could accommodate both direct and indirect left turns, Weighted average travel delay and weighted average travel time were used for comparison purposes.

Safety Analysis

Conflict data were analyzed in this study to evaluate the safety impacts of the two types of median openings *(3)*. A total of 371 conflicts were recorded before the full median opening was converted to a directional median opening, and a total of 327 conflicts after the improvement. Hourly number of conflicts, daily number of conflicts and conflict rates (conflicts per thousand involved vehicles) were estimated for comparison purposes.

RESULTS

Operational Analysis

Results of the operational analysis using weighted average delay and weighted average travel time are given in Figures 2 and 3, respectively. It was found that the differences in delay during before and after periods were statistically significant for both peak and non-peak periods. As for the travel time, even though there is a slight reduction during the after period the difference was not statistically significant. Accordingly, even if the directional median opening yields lesser delay for left-turning vehicles there is no change in the total travel time since it requires longer distance to be traveled.

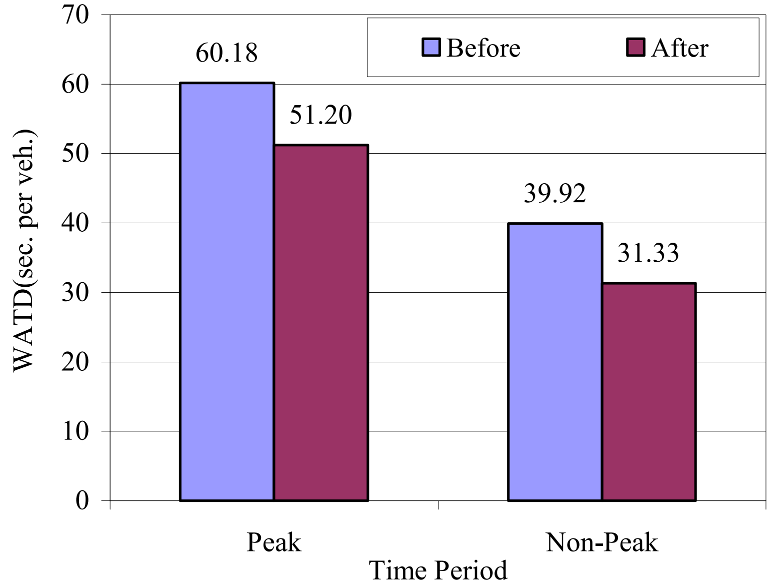

FIGURE 2. Before and After Comparison of Weighted Average Travel Delay

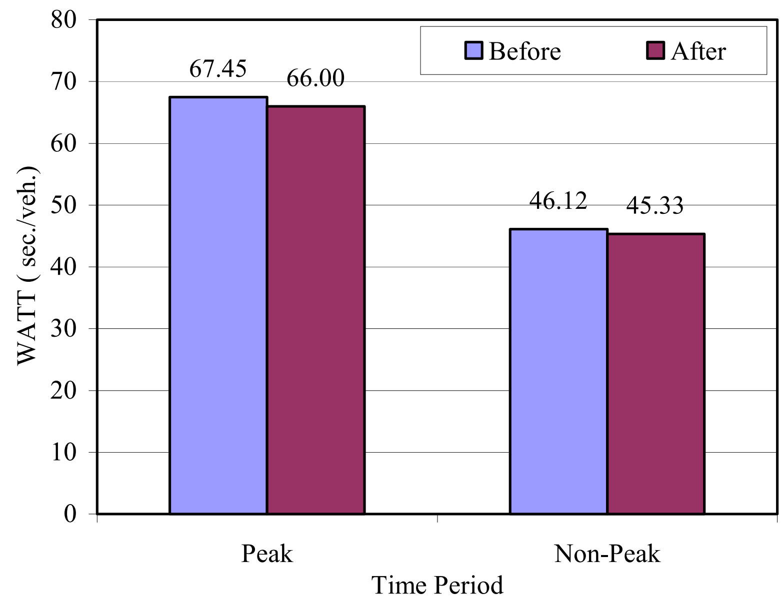

FIGURE 3. Before and After Comparison of Weighted Average Travel Time

Safety Analysis

Results of the conflict analysis to evaluate the safety effects of the two median opening types are given in Tables 1 and 2, by considering total number of conflicts per hour and conflicts per thousand involved vehicles. It was seen that the number of conflicts per hour and conflicts per thousand involved vehicles were both significantly reduced due to the change in the median opening type to directional. Percentage reductions in conflicts per hour and conflicts per thousand involved vehicles were 49.9 percent and 46.3 percent, respectively. In a detailed analysis of severity of reported conflicts (that is not described in this paper), it was found that the severity of conflicts was also less for the after period.

TABLE 1. Comparison of Total Number of Conflicts per Hour During Before and After Time Periods

Number of Conflicts/Hour During the Before Period			Number of Conflicts/Hour During the After Period			Reduction %
Due to DLT	Due to RTUT	Total	Due to DLT	Due to RTUT	Total	(Before – After) x 100 / Before
Peak Period						
23.92	3.20	27.12	0	13.14	13.14	51.5%
Non-peak Period						
19.00	4.25	23.25	0	12.06	12.06	48.1%
Total Average						
21.46	3.72	25.18	0	12.60	12.60	49.9%

TABLE 2. Average Number of Conflicts per Thousand Involved Vehicles

Time	Conflicts per Thousand Involved Vehicles	
	Before	After
07:00–08:00	118.71	26.82
08:00–09:00	97.94	27.85
09:00–10:00	54.68	37.08
10:00–11:00	55.31	53.63
11:00–12:00	51.00	61.96
12:00–13:00	73.64	27.48
13:00–14:00	62.43	32.82
14:00–15:00	55.62	44.86
15:00–16:00	72.22	34.49
16:00–17:00	77.54	46.28
17:00–18:00	65.59	28.04
Average	71.33	38.30
Average Reduction = 46.3%		

CONCLUSIONS AND RECOMMENDATIONS

This case study proved the fact that access management treatments could be used effectively to improve traffic operations and/or safety. Even though studying only one location does not statistically provide sufficient evidence, this case study was conducted as a supplement to a bigger project that studied the subject. Using the same site in the form of a before and after study accounted for the argument that often arises of differences in site characteristics when multiple sites were used.

By converting a full median opening into a directional median opening, the weighted average delay experienced by left turning vehicles was significantly reduced even though the reduction in total travel time was unaffected. However, safety effects of the conversion was highly significant where the conflicts per hour and conflicts per thousand involved vehicles were reduced 49.9 percent and 46.3 percent, respectively. Accordingly the conversion was highly successful under the prevailing conditions.

ACKNOWLEDGMENTS

This case study was completed as a part of a much larger project, which was funded by the Florida Department of Transportation.

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Young Drivers and Run-Off-the-Road Crashes

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ABSTRACT

Motor vehicle crashes are one of the leading causes of death among young Americans. They also experience higher percentage of single vehicle, Run-off-the-road (ROR) crashes compared to other drivers. When looking at the methods of improving the alarming death rate of young drivers, it is important to identify the determinants of higher crash and injury severity. With that intention, the study described in this paper developed the models to identify the factors that are influential in making an injury severity difference to young drivers involved in ROR crashes.

Since the outcome of the models, crash severity, is of discrete nature, logistic regression was identified as the most suitable approach and a set of sequential binary logistic regression models was developed to identify the influential factors. The intention was to predict the crash severity outcome of single vehicle ROR crashes involving young drivers. The developed models were validated and the accuracy was tested by using a set of crash data that were not utilized in the model development, where results were found to be satisfactory. Factors influential in making a crash severity difference to young drivers involved in ROR were then identified through the models. Factors such as influence of alcohol or drugs, ejection in the crash, point of impact, rural crash locations, existence of curve or grade, and speed of the vehicle were significantly important towards increasing the probability of having a more severe ROR crash. Restraint device usage and being a male clearly reduced the tendency of high severity and some other variables such as weather condition, residence location, and physical condition were not important at all.

Key words: run-off-the-road crashes—safety—young drivers

INTRODUCTION

Motor vehicle crashes are one of the leading causes of death among young Americans. Young drivers have also been identified as a high-risk group among several special population groups *(1).* The group of young drivers 15 to 20 years old had the highest fatality involvement rates per hundred thousand drivers, where the rate was 64.6. Fatality crash involvement per hundred thousand drivers for the 21 to 24 year old driver age group was 45.0 and it was the second largest among all the age groups *(2)*. Poor safety performance by young drivers could in general be attributed to three major factors, inexperience, risk taking behavior and immaturity, and greater risk exposure *(3).* Driving a motor vehicle is a complex task that requires many skills and knowledge that are gathered through experience. Young drivers who start with little technical ability, lack of proper judgment, and inexperience are therefore more likely to be involved in highway crashes. They are also more likely to be engaged in high-risk behaviors such as not wearing seat belts, speeding, impaired driving, and inattention. Factors that tend to increase the crash risk such as nighttime driving and having other young passengers are also more common among young drivers. The crash experience of young drivers is different from that of other drivers. Young drivers are involved in more single vehicle fatal crashes than any other driver age group *(3).* In this type of fatal crashes, the vehicle usually leaves the road and overturns or hits a roadside object such as a tree or a pole.

When looking at the remedies to reduce the alarming fatality crash involvement rates of young drivers, it is important to identify the factors that are influential in making a crash severity difference. With that intention, the study described in this paper developed a set of sequential binary logistic regression models to identify the roadway, driver, environmental, and vehicle related factors influencing crash severity. Through the use of sequential model structure, models are able to capture the differences between several severity levels.

The main objective of this study was to identify the roadway, driver, environmental, and vehicle related factors influencing the severity of run-off-the-road (ROR) crashes involving young drivers. The factors that tend towards increased crash severity could also be expected to contribute towards the occurrence of crashes as well.

METHODOLOGY

In this study, young drivers were considered as drivers from 16 to 25 years old. It concentrated only on single-vehicle ROR crashes due to two main reasons: (1) Proportion of single vehicle ROR crashes was very high among young drivers, (2) It was necessary to identify the factors that were solely associated with young drivers. In the case of two or multi vehicle crashes contributions from drivers other than the young driver might affect the crash severity, which could be avoided by considering only single vehicle ROR crashes. The size and weight of the vehicle might also be expected to have an influence on the crash severity. In order to control for that, only passenger cars were considered in the crash severity modeling. Passenger cars for the purpose of this study were considered as automobiles, passenger (mini) vans, and light pick up trucks with two rear tires.

Since the outcome of the models, crash severity, is of discrete nature, logistic regression was identified as the most suitable approach to identify the important factors. Two types of logistic regressions, ordinal response logistic regression and sequential binary response logistic regression, were tried and the best models were selected. Since the ordinal response logistic regression equations developed to model the young driver crashes failed to meet the common slope parameter assumption associated with the predictor variables, they were not considered further. Instead, the modeling process was continued with the sequential binary logistic regression modeling. SAS version 8.1 was used in the model development mainly utilizing the capabilities of the PROC LOGISTIC procedure *(4, 5)*.

Fitting a Binary Logistic Regression Model

In the case of binary logistic regression models, the relationship between a binary response variable and one or more explanatory variables are modeled. The logistic regression model uses the explanatory variables to predict the probability that the response variable takes on a given value. The response variable takes one of the two binary values (0 and 1) in the case of binary logistic regression models.

For a binary response variable y, the linear logistic regression model has the form,

$$\begin{aligned} Logit\,(p_i) &= \log\left[p_i \,/\left(1 - p_i\right)\right] \\ &= \alpha + \beta' X_i \end{aligned},$$

where

p_i = Prob. $(y_i = y_1 \mid X_i)$ is the response probability to be modeled, and y_1 is the first ordered level of y

α = intercept parameter

β' = vector of slope parameters

X_i = vector of explanatory variables

This logistic regression equation models the logit transformation of the i^{th} individual's event probability, $p_{i,}$ as a linear function of the explanatory variables in the vector, $X_{i.}$ A more general class of models share the feature that a function $g = g(\mu)$ of the response variable is assumed to be linearly related to the explanatory variables *(6)*. The function g is known as the "link function". Since the logit function has advantages like being able to be more easily interpreted, it was used in developing the crash severity models for young drivers.

Data

The data for the crash severity model development involving young drivers were extracted from the Florida Traffic Crash Database obtained from the State Data Program *(7)*. This database contains the details of all the police reported crashes in Florida available in three sub-files—accident file, personal file, and vehicle file—which could be combined by using the common variable CASENO that refers to the identification number of each crash. Crash data available for the most recent two-year period (1997–1998) was used in the modeling process and 1996 data was used in the validation. Crash severity, which was defined as the most severe injury to a person involved in the crash, was defined under five major categories: no injury, possible injury (a complaint of pains or claims of injury that are not apparent), non-incapacitating injury (an injury that seriously affect the normal functioning), incapacitating injury (a visible injury that does not seriously effect the normal functioning), and fatal (within 90 days). The law enforcement officer who completes the crash report determines this injury severity based on his/her judgment. Frequencies of crash severity for the dataset used in the logistic regression modeling are given in Table 1, under different crash severity categories.

TABLE 1. Frequencies Distribution of Crash Severity in the Data Sample

Crash Severity Level	Frequency
No injury	3,443
Possible injury	1,509
Non-incapacitating injury	2,250
Incapacitating injury	1,044
Fatal (within 90 days)	136
Total	8,382

Modeling Procedure

As the difference in crash severity at each level was considered important, four sequential binary logistic regression models were developed as follows:

i. Fatal (Binary response = 1) and At Most Incapacitating Injury (Binary Response =0)
ii. Incapacitating Injury (Binary response = 1) and At Most Non-incapacitating Injury (Binary Response =0)
iii. Non-incapacitating Injury (Binary response = 1) and At Most Possible Injury (Binary Response =0)
iv. Possible Injury (Binary response = 1) and No Injury (Binary Response =0)

For each model, the relationship between the binary response variables was modeled with a set of independent variables. From all the variables included in the Florida Traffic Crash Database, the variables that could in some way be assumed to be related to severity were sorted out while eliminating the strongly correlated ones. These selected variables were then coded into the binary form except for the travel speed. The considered independent variables and the meanings of their binary values are given in Table 2. Only the variables with significant influence on the crash severity outcome were included in the models. The regression model coefficient of each significant independent variable provides and explanation of the type of influence that variable has on the model outcome. The logistic regression model with the selected set of independent variables and estimated model coefficients could be used to predict the probability that the response variable takes a given value.

TABLE 2. Independent Variables Considered in the Logistic Regression Modeling

Variable	Interpretation	
	Binary Response = 1	**Binary Response = 0**
AL_DRG	Driver was under influence of alcohol or drugs.	Driver was not under influence of alcohol or drugs.
PERDEF	Physical condition of the driver was a factor.	Physical condition of the driver was not a factor.
PEJECT	Driver has ejected in the crash.	Driver has not ejected in the crash.
MALE	Driver was a male.	Driver was a female.
IMP_SIDE	Impact point was side of the vehicle.	Impact point was not side of the vehicle.
IMP_FRNT	Impact point was front of the vehicle.	Impact point was not front of the vehicle.
FAULTC	Driver was at fault for the crash.	Driver was not at fault for the crash.
RESTR	A restraint device was used.	A restraint device was not used.
RURAL	Crash occurred in a rural area	Crash was not in a rural area.
GR_CUR	Curve or grade exists at the crash location.	Crash location was straight and level.
FREEWY	Crash occurred on a freeway.	Crash was not on a freeway.
VFOLT	Vehicle was at fault for the crash.	Vehicle was not at fault for the crash.
CNTY_RES	Driver was a resident of the same county.	Driver was a non-resident of the county.
BDWTHER	Weather was not clear.	Good weather conditions.
DAYLIGHT	Crash occurred during daylight conditions.	Crash was not during daylight conditions.
SPEED	Actual speed of the vehicle at the time of the crash.	

RESULTS

Model Coefficients and Odds Ratios

Results of the crash severity models for young drivers involved in single vehicle, fixed object passenger car crashes are presented in Table 3 together with their corresponding odds ratios. Independent variables that were significant at 95 percent level were included in the model. Odds ratio measures the odds of the outcome being increased if the value of that independent variable is subjected to a unit increase. It is therefore a good indication of the strength of each of the independent variable towards the crash severity related to young driver crashes considered in this study.

The parameters known as Somers' D, Gamma, Tau-a, and c, were calculated by the association of predicted probabilities and observed responses in order to evaluate the model fitness. In a general sense, the higher the values of these indices are, better the model fitness with the actual data is *(5)*. The parameters provided reasonably satisfactory values in the case of these binary logistic regression models.

TABLE 3. Crash Severity Model Coefficients and Odds Ratios for the Second Model Format

Variable	Coefficients in Model (Odds Ratio)			
	1. Fatal and at Most Incapacitating Injury	**2. Incapacitating Injury and at Most Non-incapacitating Injury**	**3. Non-incapacitating Injury and at Most Possible Injury**	**4. Possible Injury and No Injury**
INTERCEPT	–4.795	–2.530	–1.286	–1.068
AL_DRG	1.346 (3.84)	NS*	NS	–0.436 (0.65)
PERDEF	NS	NS	NS	NS
PEJECT	1.736 (5.67)	1.528 (4.61)	0.802 (2.23)	1.48 (4.37)
MALE	NS	–0.292 (0.75)	–0.338 (0.71)	–0.736 (0.48)
IMP_SIDE	0.851 (2.34)	NS	0.203 (1.23)	–0.242 (0.79)
IMP_FRNT	NS	NS	0.284 (1.33)	NS
FAULTC	–3.328 (0.04)	NS	NS	NS
RESTR	–1.360 (0.26)	–0.875 (0.42)	–0.645 (0.53)	–0.319 (0.73)
RURAL	NS	0.484 (1.62)	0.554 (1.74)	0.512 (1.67)
GR_CUR	0.629 (1.88)	0.229 (1.26)	0.324 (1.38)	0.187 (1.21)
FREEWY	NS	NS	NS	NS
VFOLT	NS	NS	0.125 (1.33)	NS
CNTY_RES	NS	NS	NS	NS
BDWTHER	NS	NS	NS	NS
DAYLIGHT	–0.572 (0.56)	0.219 (1.25)	NS	0.202 (1.22)
SPEED	0.026 (1.03)	0.02 (1.02)	0.012 (1.01)	0.020 (1.02)

Note: NS = The variable is not significant.

Influential Factors

Factors influential in making a crash severity difference to young drivers involved in single vehicle ROR crashes are obtained using the model. The probability of getting a certain crash severity under a given situation could be estimated by substituting the corresponding explanatory variables into the model. The model with the estimated coefficients would be as follows:

Logit p_1 = - 4.795 + 1.346(AL_DRG) +1.736(PEJECT) + 0.851(IMP_SIDE) –3.328(FAULTC) – 1.36(RESTR) + 0.629(GR_CUR) – 0.572(DAYLIGHT) + 0.026(SPEED)
Logit p_2 = -2.53 + 1.528(PEJECT) – 0.292(MALE) – 0.875(RESTR) + 0.484(RURAL) + 0.229(GR_CUR) +0.219(DAYLIGHT) + 0.02(SPEED)
Logit p_3 = - 1.286 + 0.802(PEJECT) – 0.338(MALE) + 0.203(IMP_SIDE) + 0.284(IMP_FRNT) – 0.645 (RESTR) + 0.554 (RURAL) + 0.324(GR_CUR) + 0.125VFOLT+ 0.012(SPEED)
Logit p_4 = - 1.068 + 0.436(AL_DRG) + 1.48(PEJECT) – 0.736(MALE) - 0.242(IMP_SIDE) - 0.319 (RESTR) + 0.512(RURAL) + 0.187(GR_CUR) + 0.202(DAYLIGHT) + 0.02(SPEED)

where
p_1 = Probability of occurrence of a fatality given that a crash has occurred
p_2 = Probability of occurrence of an incapacitating injury given that at most incapacitating injury has occurred
p_3 = Probability of occurrence of a non incapacitating injury given that at most non-incapacitating injury has occurred
p_4 = Probability of occurrence of a possible injury given that at most possible injury has occurred

According to the way that the models were developed the coefficient of an independent variable is directly related to the probability of having a more severe crash. Therefore, a positive coefficient indicates a variable that increases the probability of having a more serious crash outcome and vice versa.

CONCLUSIONS AND RECOMMENDATIONS

This study developed the models to estimate the severity of young driver ROR crashes using sequential binary logistic regression modeling and thereby identified the factors influential in making a crash severity difference. It should be noted that the basis of data for these model developments were the police crash reports and therefore the commonly raised concern of the accuracy of such information would affect these results as well *(8).* Particularly, identification of the severity level at certain levels might have been subjective in nature and therefore might affect the accuracy of the models. Sequential binary logistic regression modeling provided a reasonable way of predicting the crash severity with fairly acceptable accuracy. Use of alcohol or drugs, ejection in the crash, gender, impact point of the vehicle, restraint device usage, urban/rural nature and grade/curve existence of the crash location, lighting condition, and speed were the most important factors affecting the severity of young driver single vehicle fixed object crashes involving passenger cars.

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Railroad Flatcar Bridges for Economical Bridge Replacement Systems

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ABSTRACT

The use of Railroad Flatcars (RRFCs) as the superstructure in low-volume bridges has been investigated in a research project at Iowa State University. These alternative bridges will enable county engineers to replace old, inadequate county bridges for less money and in a shorter construction time than required for a conventional bridge.

A feasibility study completed in 1999 by the Bridge Engineering Center at Iowa State University determined that RRFC structures have adequate strength to support Iowa legal traffic loads. In a follow-up research project, two RRFC demonstration bridges with different substructures and types and lengths of RRFCs were designed, constructed, and tested to validate the conclusions of the feasibility study.

Bridge behavior predicted by grillage models was supported by data from field load tests, and thus, design recommendations were developed for determining live load distribution in the RRFC bridges. Moreover, it was determined that the engineered RRFC bridges had live load stresses significantly below the steel yield strength and deflections significantly lower than the AASHTO Bridge Design Specification limits. Finally, it was proven that RRFC bridges can be constructed for considerably less money and in a shorter construction time than required for a conventional bridge. Based on the results of this research, it has been determined that through proper RRFC selection, connection, and engineering design, RRFC bridges are a viable, economic alternative for low-volume road bridges.

Key words: field testing—replacement structures—RRFC (railroad flatcar) bridges

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INTRODUCTION

Approximately eighty-one percent of Iowa's 25,000 bridges are on secondary roads, and thus, are the responsibility of the counties. The number of bridges in Iowa ranks it 6^{th} in the nation while Iowa's population ranks 30^{th}. Therefore, the state's tax base is limited, and as a result, Iowa county engineers have inadequate funds to properly address the secondary road bridge problems. To address this problem, the Bridge Engineering Center at Iowa State University (ISU) investigated the feasibility of using railroad flatcars (RRFCs) as the superstructure on low-volume bridges. Railroad flatcars offer several attractive characteristics that make them desirable for superstructures; they are easy and quick to install, can be used on existing or new abutments, are available in various lengths, require low maintenance, and are relatively inexpensive. In 1999, results from a feasibility study indicated that properly designed RRFC bridges can carry Iowa legal loads (*1*).

Addressing the recommendations from the 1999 ISU feasibility study, a follow-up research project was initiated in 2000 to design and construct two RRFC demonstration bridges (See Figure 1) (*2*). Buchanan and Winnebago Counties in Iowa expressed interest in using the RRFC bridge concept since it was envisioned that a RRFC bridge could be constructed for less than one half the cost of a conventional bridge. Therefore, the objectives of the follow-up research were to 1) develop a process for selecting structurally adequate flatcars, 2) develop design and construction guidelines for these alternative LVR bridges, and 3) design, construct, and test two demonstration bridges. The following tasks were undertaken to meet the research objectives:

1. Thorough inspection and selection of readily available, decommissioned RRFCs.
2. Construction and testing of a laboratory connection specimen that simulated a connection between RRFCs.
3. Design and construction of two RRFC demonstration bridges with different types of flatcars, span lengths, and substructures.
4. Field load testing of the RRFC bridges before and after the flatcars were connected.
5. Comparison of analytical and experimental results.
6. Development of a simplified design methodology.

RRFC INSPECTION AND SELECTION

The bridges to be replaced in Buchanan and Winnebago counties were 39 ft (11.9 m) and 56 feet (17.1 m) long (out-to-out), respectively, and thus, it was desired to find RRFCs of similar size. While searching for flatcar suppliers, it was found that most railroad salvage yards own or have access to decommissioned flatcars. Erman Corporation, who supplied the RRFCs for the project, searched for decommissioned RRFCs of the desired lengths and located 56-ft (17.1-m) v-deck RRFCs, 85-ft (25.9-m) RRFCs, and two types of 89-ft (27.1-m) RRFCs. Selection of adequate RRFCs is critical to the success of RRFC bridges, and therefore, five criteria were developed and used to evaluate each type of flatcar.

(a) Buchanan County Bridge

(b) Winnebago County Bridge

FIGURE 1. Photographs of the Completed RRFC Demonstration Bridges

(1) Member Sizes, Support Locations, and Load Transfer Capabilities

To support Iowa legal loads, the RRFC members must be of sufficient size to meet strength and serviceability criteria. The structural elements of each RRFC can be categorized by: decking, girders, secondary members, and transverse members. The decking is the top steel plating on each RRFC; griders and secondary members are oriented longitudinally on the RRFC. Girders are the largest members in the RRFC, and thus, support most of the loads since they are supported at the piers and/or abutments. Each typical RRFC has three girders – two exterior and one interior. The transverse members are connected orthogonally to the longitudinal members, and thus, function to transfer loads from the secondary members to the girders. Secondary members are the remaining longitudinal members that typically transfer load from the decking to the transverse members.

When inspecting RRFCs, it is important to identify regions that are adequate to serve as support locations at the piers and/or abutments. For most RRFCs, the girders will be supported at the piers and/or abutments on the bolsters, or wheel locations on the RRFC. If supported at areas other than the bolster areas, the members need to be checked for adequate strength and stability in the bearing area.

(2) Member Straightness/Damage

While many flatcars have been decommissioned because new designs have made them obsolete or their net worth has depreciated to essentially zero, some flatcars have been removed from service because they have been damaged. Deformed members will not adequately carry or distribute loads, and in addition, they may have been yielded or buckled. Therefore, visual inspection, use of string lines, etc., should be used to determine that all structural members are straight; flatcars with deformed or buckled members, or those with obvious signs of repair, should be rejected.

(3) Member Connections

RRFC members are either connected by rivets or welds. Rivets, however, can lose significant strength over time primarily due to corrosion. Even if all rivets appear to be in good condition at the time RRFCs are inspected, to avoid future problems, it is recommended that only RRFCs with welded member connections be used. Although fatigue issues should not typically be a concern with bridges on low-volume roads, all welded connections need to be visually inspected to ensure that fatigue cracks are not already present.

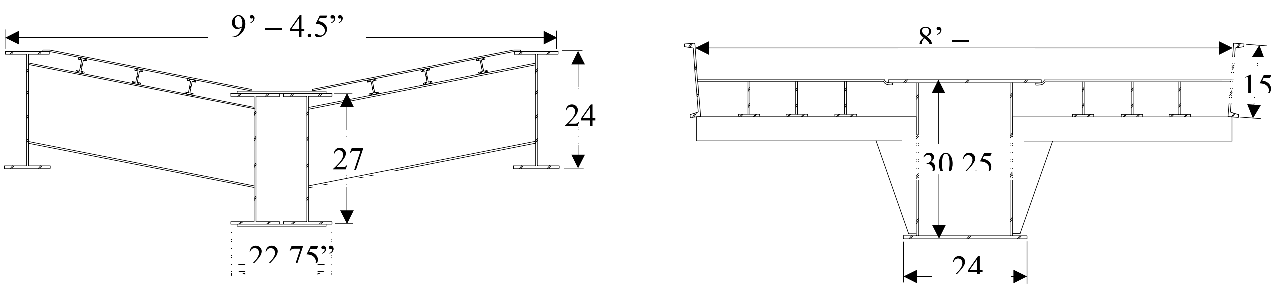
(4) Uniform and Matching Cambers

Since several RRFCs must be connected transversely to provide the desired bridge width, careful attention should be given to the camber of each RRFC. Although it was found that cambers are essentially the same for most RRFCs of a particular type, some flatcar cambers do vary from the norm. Not only will significantly differing cambers cause problems when connecting the flatcars, but it could also cause difficulties when trying to construct a smooth driving surface.

(5) RRFC Availability

The use of RRFCs on low-volume bridges is obviously subject to the availability of decommissioned flatcars. Flatcars are removed from service because new designs make them obsolete or because their net worth has depreciated to essentially zero. However, it is recommended that flatcars be selected that have been removed from service because of obsolescence. In addition, if possible, select a type of RRFC that is abundantly available so that bridges may be constructed repetitively, and thus, not requiring new designs.

Using these five criteria and a simplified grillage analysis to evaluate each type of RRFC, it was determined that the 56-ft (17.1-m) v-deck style RRFC and the 89-ft (27.1-m) style RRFC shown in Figure 2 were the best flatcars for the Buchanan County Bridge (BCB) and the Winnebago County Bridge (WCB), respectively.



(a) 56-ft RRFC **(b) 89-ft RRFC**

FIGURE 2. Cross-sections of the RRFCs Used

BRIDGE DESIGN AND CONSTRUCTION

In addition to investigating two different RRFC superstructures, it was possible to design, construct, and investigate different substructures as well as RRFC connections in the demonstration bridges. With the exception of the piling driven for the abutments of each bridge, all construction was completed with local maintenance personnel and equipment. Details of each bridge are presented in the following sections.

Buchanan County Bridge

The flatcars for the BCB were supported at their ends, which were checked for strength and stability, on 3-ft (314 mm) square, reinforced concrete cap beams with backwalls; each cap beam was supported by five HP 10x42 steel piling (See Figure 3a). The use of reinforced concrete in the substructure allowed for an integral abutment at one end of the bridge with an expansion joint at the other end. Longitudinal flatcar connections consisting of reinforced concrete beams with transverse threaded rods spaced 24 in. (610 mm) on center were installed between the flatcars for distributing live loads efficiently among the three RRFCs (See Figure 3b and Figure 3c). To ensure that the longitudinal connections supported their own self weight, midspan shoring was used during construction of the connections, which reduced the dead load being distributed to the steel structural members. After structurally connecting the RRFCs, a layer of pea gravel was placed on the RRFCs to facilitate deck drainage. This was followed by installation of an asphalt milling driving surface approximately 5.5 in. (140 mm) and 9.0 in. (229 mm) deep, respectively, at the edges and middle of the bridge (with respect to the tops of the flanges on the exterior members). Finally, a guardrail system was attached.

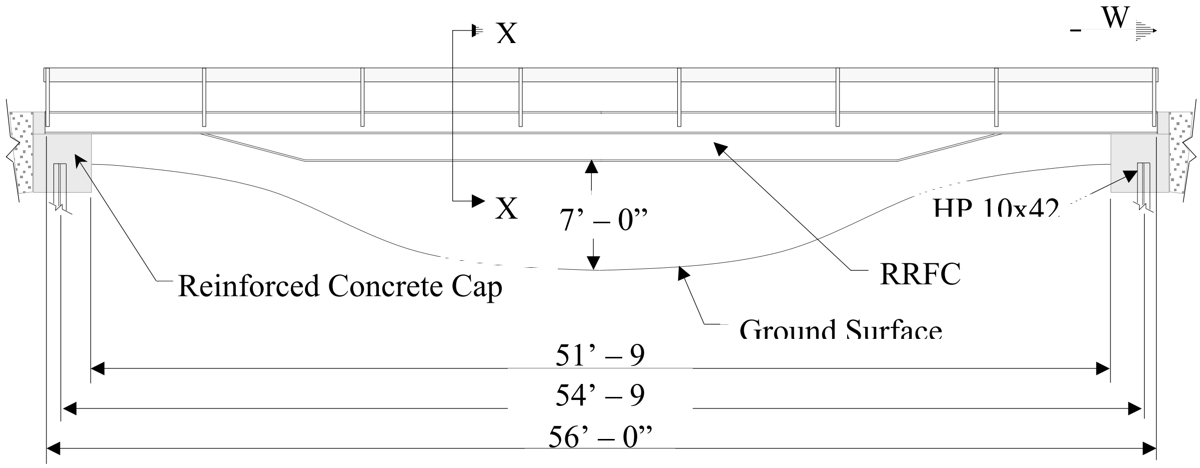
Winnebago County Bridge

The WCB demonstration bridge is a three span structure because preliminary calculations determined that the 89-ft RRFCs would be inadequate for a single span (See Figure 4a). Therefore, the 89-ft (27.1-m) flatcars were supported by steel-capped piers and abutments at the RRFCs' bolsters and ends, resulting in a 66-ft (20.1 m) main span with two 10-ft (3.0 m) end spans. The use of steel as the substructure saved construction time by eliminating the need for concrete formwork and curing time. A sheet pile wall provided roadway support at each abutment. For connection between adjacent RRFCs, the top half of the exterior channels above the deck surface was removed on the exterior girders, and longitudinal plates were welded to the top and bottom of the trimmed, exterior channels to form a structural tube (See Figure 4b and Figure 4c). The tube was reinforced with transverse threaded rods on approximately 24 in. (610 mm) centers, and a single #5 longitudinal reinforcement bar was added for crack control prior to filling the void with concrete. After connecting the flatcars, the south ends of the RRFCs were welded to the abutment to restrain vertical and horizontal translation, and expansion joints (which prevented vertical translation) were added to the north ends of the RRFCs, and on each pier. Recycled timber planks were installed transversely and connected to the flatcars to help provide transverse load distribution. A gravel driving surface was placed on top of the timber planks, and a guardrail system was installed.

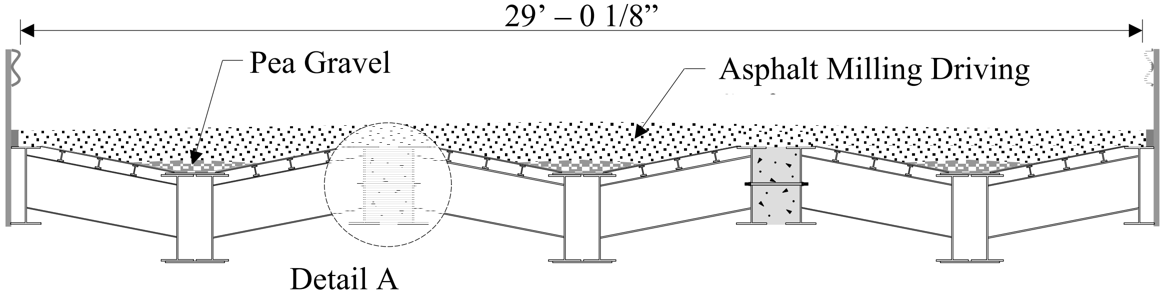
RRFC BRIDGE PERFORMANCE

Laboratory Tensile Coupon Testing

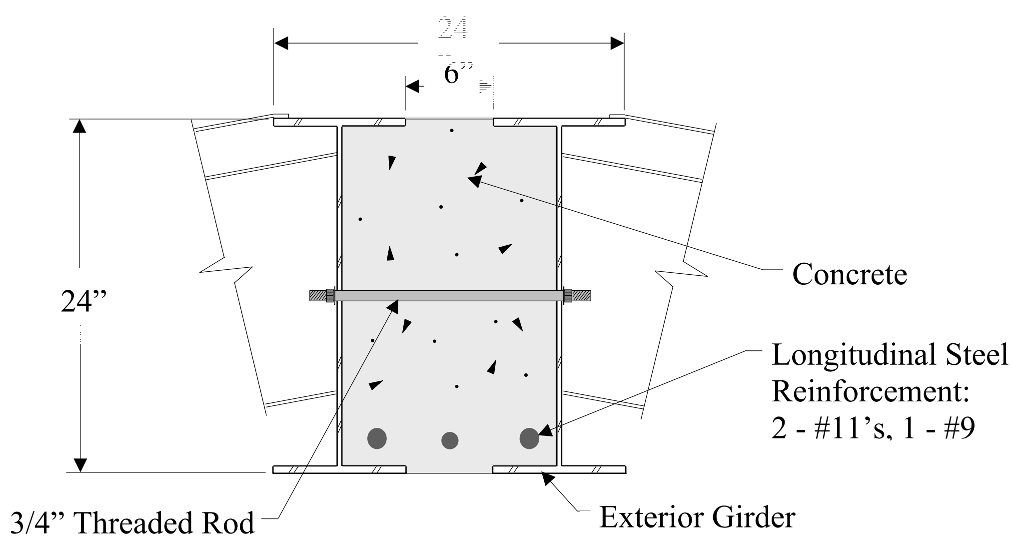
ASTM tensile tests were performed on steel coupons from both the 56-ft (17.1-m) flatcar and the 89-ft (27.1-m) flatcar. The proportional limit and modulus of elasticity (MOE) from the stress-strain diagrams for both types of flatcars were determined to be approximately 40 ksi (275 MPa) and 29,000 ksi (200,000 MPa), respectively. Thus, a conservative yield strength was assumed to be 36 ksi (248 MPa) for both flatcars.



(a) Side View of BCB

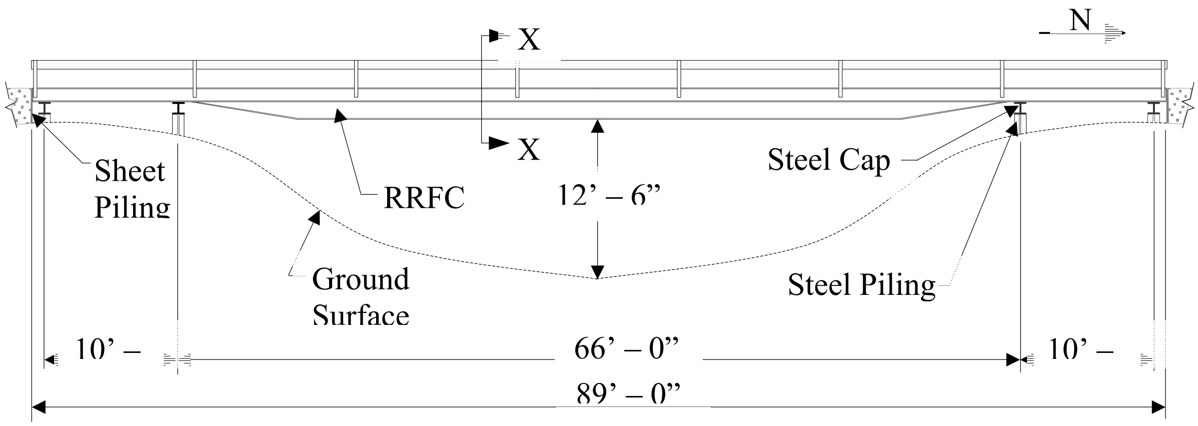


(b) Section X – X

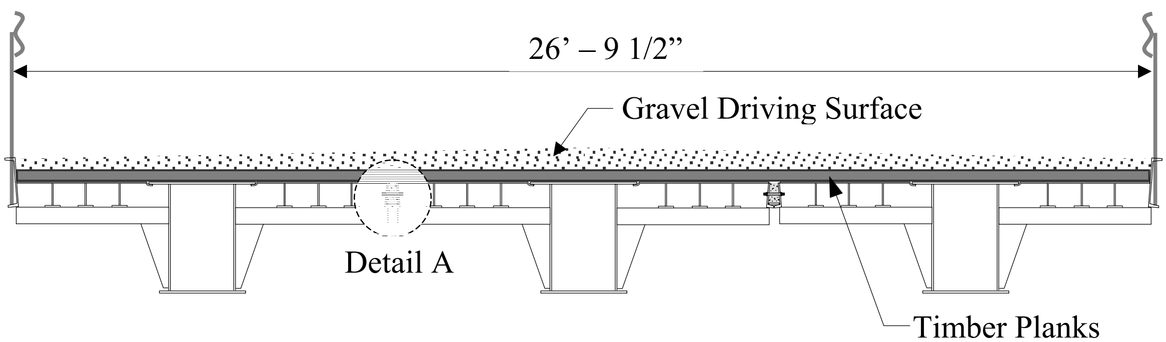


(c) Detail A

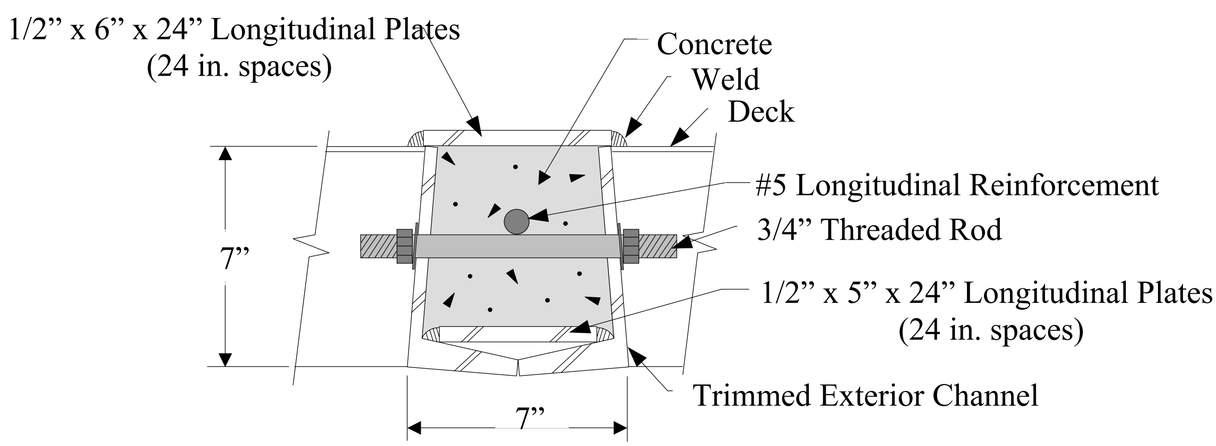
FIGURE 3. Buchanan County RRFC Bridge



(a) Side view of WCB



(b) Section X – X



(c) Detail A

FIGURE 4. Winnebago County RRFC Bridge

Field Loading Test Procedure

To investigate bridge behavior, each RRFC bridge was field tested with rear tandem trucks carrying gross loads of 51,000-52,500 lbs (227-234 kN). On each bridge, Load Test 1 (LT1) was performed after the flatcars were placed on the abutments and/or piers, but before they were connected. Load Test 2 (LT2) was performed immediately after construction of the bridge was completed. Comparison of LT1 and LT2 test results revealed the effectiveness of the longitudinal flatcar connections.

In each test, strains and deflections were continually measured and recorded at several critical longitudinal locations as the truck crossed the bridge. Through a feature in the data acquisition system, it was possible to highlight strain and deflection results for a specific time that corresponded to an individual, longitudinal truck position. In LT1, strains and deflections were recorded during static truck positioning, and results in LT2 were recorded while the tandem truck slowly rolled across the bridge. Because strains and deflections illustrate the same behavior, only deflection results are presented in this paper.

Buchanan County Bridge Results

LT1 consisted of 4 tests on the center RRFC with no connection between the flatcars. Since the tandem wheel base width (outside-outside of tires) was only 18.5 in. (470 mm) narrower than the width of the RRFC, it was only possible to position the test truck approximately 9.25 in. (235 mm) eccentric with respect to the longitudinal centerline of the RRFC. As a result, two tests were conducted with the test truck transversely centered on the RRFC, and two were performed with the test truck transversely eccentric, respectively, at the north and south edges of the RRFC.

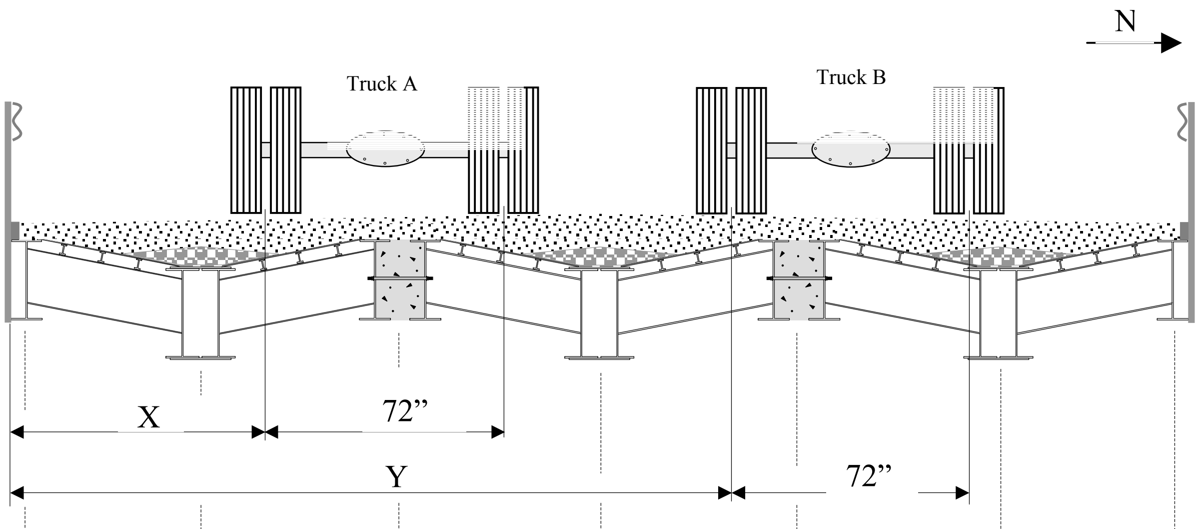
LT2 tests were performed with the longitudinal connections in place between the three RRFCs. To investigate if the asphalt milling driving surface had any effect on transverse load distribution, identical tests were performed with test trucks positioned in several transverse locations before and after the driving surface was installed. The results showed that the driving surface had minimal effect on strains, deflections, and thus, transverse load distribution.

BCB LT2 deflection results are presented in Figure 5; the maximum measured midspan deflection was 0.38 in. (9.7 mm). According to the 1994 LRFD (*3*) and 1996 LFD (*4*) AASHTO Bridge Design Specifications, the maximum allowable deflection should not exceed 1/800 of the span length, which is 0.84 in. (21.3 mm) for a 56-ft (17.1-m) span. Using the coupon MOE, the maximum flexural stresses in the longitudinal girders of the bridge due to test truck loads were approximately 4.6 ksi (31.7 MPa).

Winnebago County Bridge Results

LT1 consisted of 5 tests on the middle RRFC with no connection between the RRFCs. Three tests were performed with the RRFCs unrestrained at the abutments, and two tests were performed after the addition of the expansion joint and pinned restraint at the north and south abutments, respectively. Since the tandem wheel base width was approximately the same width as the flatcar, it was not possible to eccentrically load the 89-ft (27.1-m) flatcars.

LT2 tests were performed with longitudinal flatcar connections between the three RRFCs, and the transverse timber planks and gravel driving surface in place. The four truck positions that were investigated and their corresponding midspan deflection patterns are shown in Figure 6; the maximum measured midspan deflection was 0.63 inches (16 mm). For the 66-ft (20.1-m) main span, according to the 1994 LRFD and 1996 LFD AASHTO Bridge Design Specifications, the maximum allowable deflection should not exceed 0.99 in. (25 mm). Using the coupon MOE, it was determined that the maximum flexural stresses in the longitudinal girders from test truck loads were approximately 7.1 ksi (48.8 MPa).



LT2 Test	Transverse Position (in.)	
	Truck A	Truck B
	X	Y
1	138.75	-
2	253.50	-
3	24.00	-
4	198.00	-
5	24.00	253.00
6	67.75	209.75

Note: Truck B only used in Test 5 and Test 6.

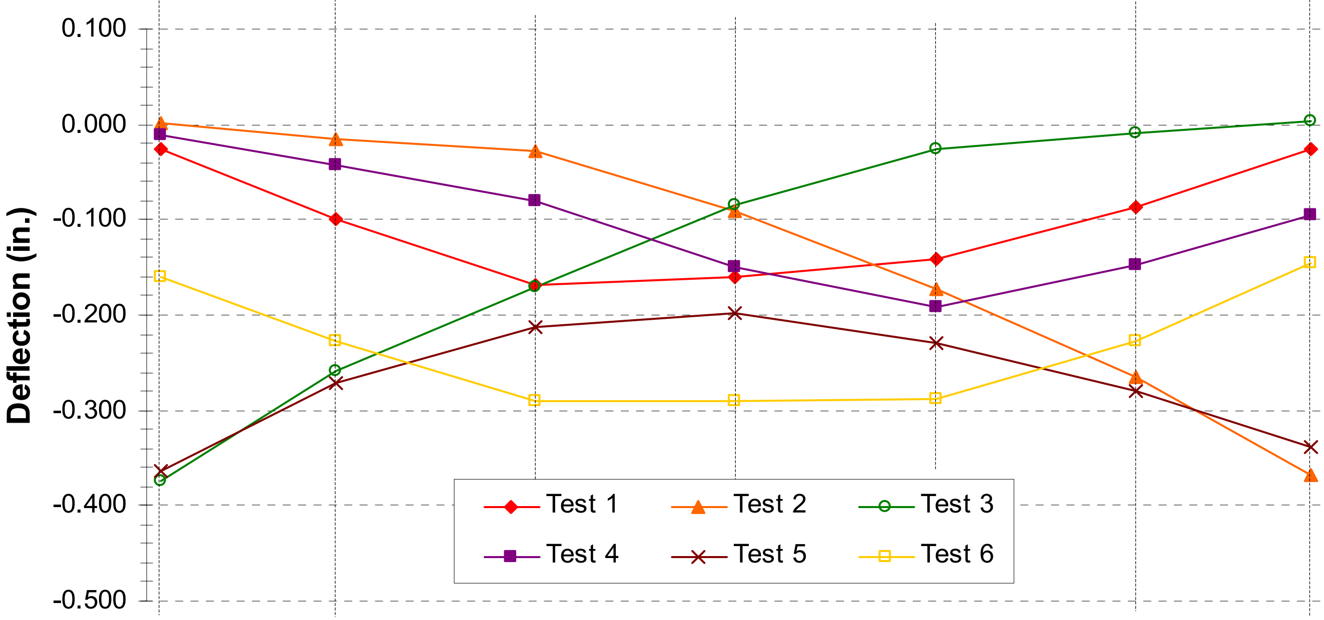
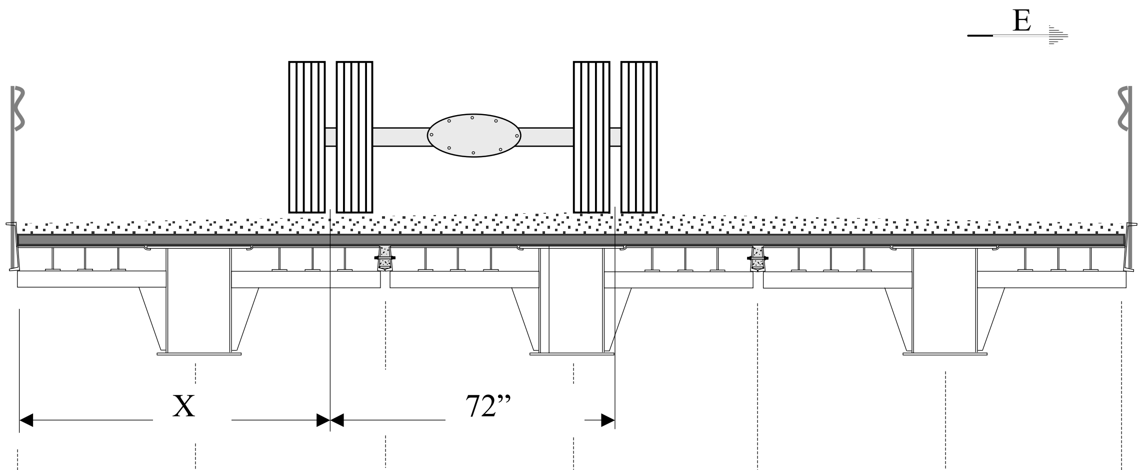


FIGURE 5. LT2 Midspan Deflections in the BCB



LT2 Test	Transverse Truck Position, X (in.)
1	24.00
2	124.75
3	297.50
4	124.75 *

* Speed = 30 mph

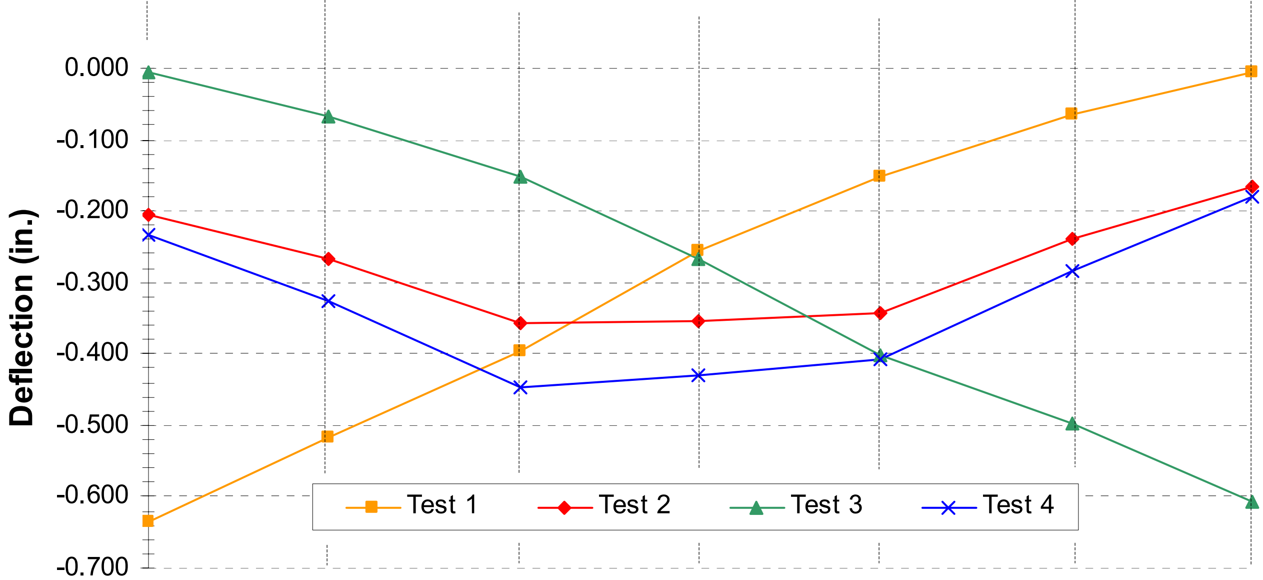


FIGURE 6. LT2 Midspan Deflections in the WCB

Summary of RRFC Bridge Performance

The deflection patterns for both bridges in LT2 showed that both types of longitudinal connections helped effectively distribute loads transversely across the bridge, and thus, utilized the combined strength of all three flatcars to support the live load. When the maximum live load stresses from the field testing are combined with calculated dead load stresses, the maximum resultant stresses in the longitudinal girders of the BCB and WCB were approximately 12.7 ksi and 16.7 ksi, respectively. Thus, the maximum stresses in the longitudinal members of both bridges were well below the conservative yield strength of 36 ksi (248 MPa), and the maximum deflections were significantly less than the AASHTO requirements. Therefore, it has been verified that RRFC bridges can adequately handle Iowa legal loads for tandem trucks.

THEORETICAL ANALYSIS

A grillage model of each bridge was developed using ANSYS, finite element software. When the test truck loads from LT1 and LT2 were applied to each model, the results from the grillage model were in good agreement with the results obtained from the field testing, and thus, showed that stresses and deflections on these types of bridges are predictable. As a result, design recommendations were developed to simplify live load distribution calculations in RRFC bridges.

RRFC BRIDGE CONSTRUCTION COSTS

The economics of the RRFC alternative bridges may be illustrated by comparing RRFC bridge costs with those of a conventional structure. Each 56-ft (17.1-m) RRFC cost $6,500, and each 89-ft (27.1-m) RRFC cost $9,700; both prices included shipping to the bridge site. If the labor and equipment costs are disregarded for each bridge, the BCB and WCB RRFC bridges cost approximately $20 per square foot

and $26 per square foot, respectively. If the actual costs for the county labor and equipment are included, the BCB and WCB RRFC bridge costs would be $39 per square foot and $37 per square foot, respectively. The rationale for excluding the county labor and equipment costs from the RRFC bridge

costs is that they were already budgeted expenses by the county. Thus, those costs would have existed throughout the construction season, regardless of the bridge replacement alternative. Therefore, only the materials and contracted labor associated with the RRFC bridges were extra expenses to the counties. The county alternative to each RRFC bridge was to contract for a concrete slab bridge costing approximately $65 per square foot.

CONCLUSION

Through the design, construction, field testing, and analysis of two RRFC demonstration bridges, it has been determined that RRFC bridges are a viable, economical alternative for low-volume bridges. The success of these demonstration bridges may be directly linked to the careful selection of the RRFC, design of the longitudinal connections between flatcars, and overall design and construction practices.

Therefore, results of this research have introduced a new, efficient, and engineered low-volume bridge replacement that in some situations will help improve secondary road transportation.

ACKNOWLEDGEMENTS

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation.

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An Overview of Scour Types and Scour-Estimation Difficulties Faced at Bridge Abutments

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ABSTRACT

The paper describes the range of scour types that may occur at bridge abutments, and addresses the extensive attendant difficulties confronting accurate estimation of abutment-scour depth. The difficulties, which include similitude aspects of laboratory experiments on scour at bridge abutments, complicate the development of reliable scour-estimation relationships. In a practical sense, the difficulties imply that estimation relationships can only be of approximate accuracy. They stem from the nature of the approach flow–field, the soil and sediment conditions at typical abutments, and thereby from the mix of scour and slope-stability failure processes potentially at play in the vicinity of bridge abutments. The full set of failure processes has yet to be determined and documented, and inevitably entails extensive investigative experiments using laboratory flumes. Arguably, the difficulties have muddled perceptions of scour extents observed at bridge sites, and have contributed to a certain level of skepticism regarding existing relationships used for scour-depth estimation.

Key words: abutments—bridges—foundations—rivers—scour

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INTRODUCTION

A particularly complex set of hydraulic engineering problems are faced when assessing the likelihood of scour of alluvial channels at bridge crossings, especially in the vicinity of bridge abutments located in compound channels. The complexities, and attendant difficulties, arise from considerations of the flow field, the varied sediments and soils, as well as from the mix of failure modes that may occur at and near abutments. Additionally, some of the complexities inevitably are difficult to replicate in a laboratory flume, and pose issues of hydraulic-modeling scale and similitude. It is small wonder that these complexities raise a concern that, relative to approach-flow depth or abutment dimensions, maximum values of local-scour depths observed in laboratory flume studies exceed values observed at actual abutments.

The present paper discusses the complexities and attendant difficulties in broad terms, addressing itself to the mix of scour processes that may occur at bridge abutments, and to the scale and similitude considerations attendant to hydraulic-modeling of scour. The complexities are under close consideration in a project the writer and colleagues currently are conducting at IIHR; the project is NCHRP 24-20, Prediction of Scour at Abutments. This project aims at producing reliable predictive relationships for scour estimation at bridge abutments. The relationships inevitably must be derived in large part from laboratory flume experiments. However, in planning and conducting such experiments, the writer and his colleagues immediately are confronted with the nettlesome issues incurred with reducing complex abutment situations to simplified, relatively tractable, yet practically meaningful flume experiments.

The first difficulty confronting flume experiments is assessment and replication of the complex flow field. A further difficulty is replication of the variable nature of the sediments and soils found at most abutment sites. These difficulties together greatly hamper estimation of abutment scour. Yet a further difficulty is appropriate scaling, via similitude considerations, of results obtained from laboratory flumes.

The sequence of figures given as Figures 1 and 2 illustrate the complexities faced when attempting flume experiments aimed at producing reasonably general and acceptably reliable prediction relations for estimating scour at abutments. Factors characterizing abutment-site morphology and sediment (and soil) conditions influence the flow field in the vicinity of an abutment the abutment site; producing a range of flow conditions. In turn, the flow field influences the type and extent of abutment failure that may occur.

FLOW FIELD

To varying extents, most channels, natural or built, are compound in shape and/or roughness. As depicted in Figures 1 and 2, they comprise a central deeper portion flanked by side portions (floodplains) formed to aid conveyance of larger flows. Though substantial information exists regarding the flow field around an abutment in rectangular channel without a floodplain, little is known about the flow field formed at a floodplain abutment that is in close proximity to the main channel, as sketched in Figures 1 and 2. It is clear, though, that the near field of flow at an abutment is significantly influenced by the far field of flow.

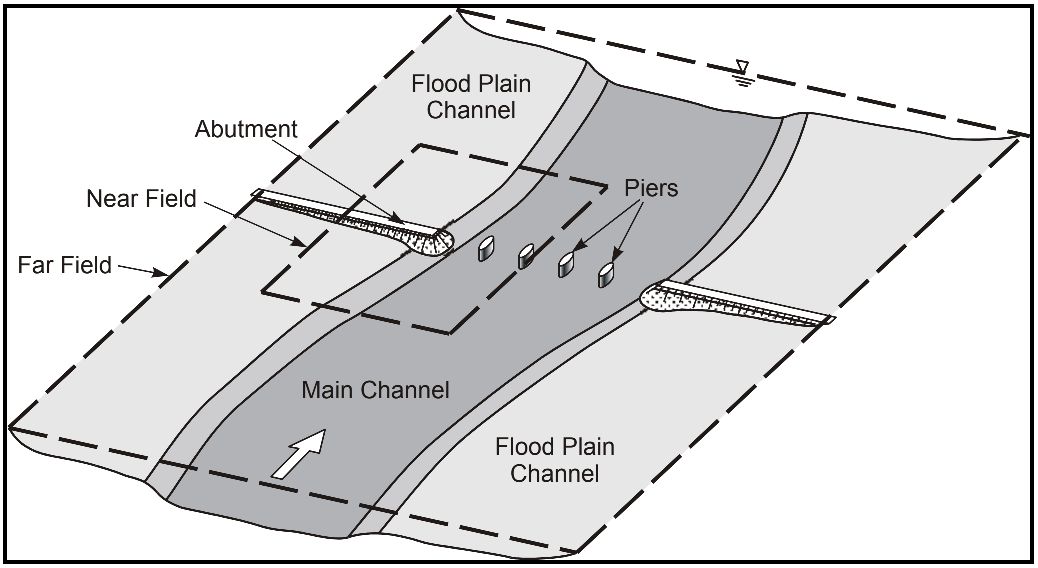

FIGURE 1. Overview of Far-Field and Near-Field of Flow in Abutment Vicinity

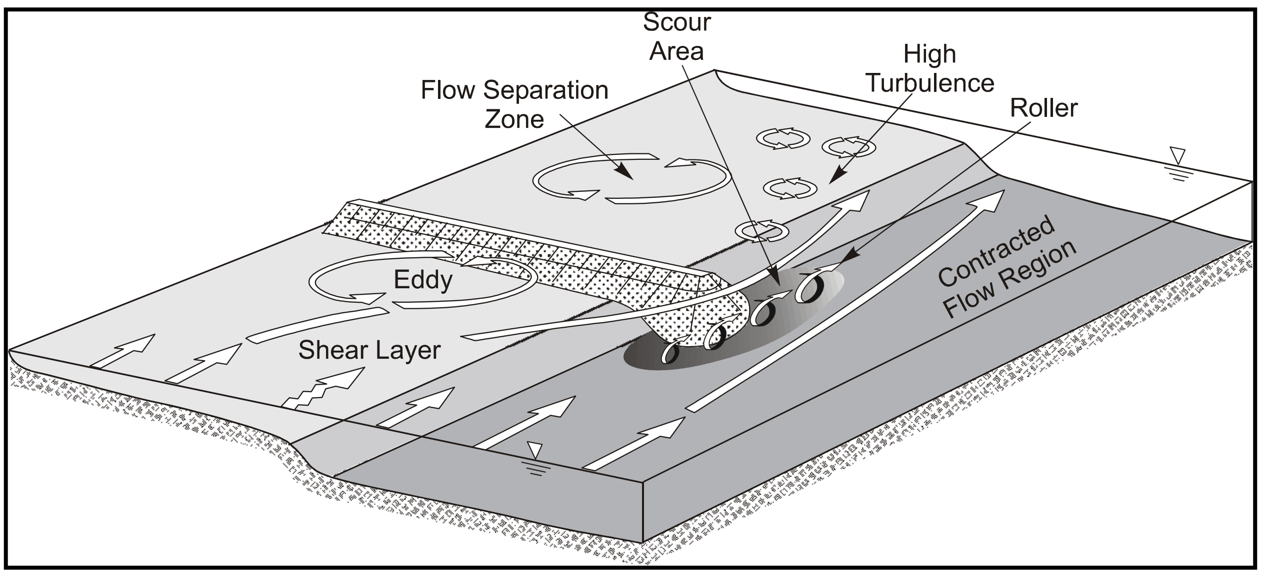

FIGURE 2. Schematic of Near-Field Flow around an Abutment (Spill-Through Shape)

Additionally, it is important to observe that, besides the overall complexity of flow field in compound channels, turbulence (its generation, dispersion, and decay) at a variety of scales is a prominent feature of the flow field, not only in the immediate vicinity of the abutment, but also at in the approach flow to the bridge. This flow feature poses a similitude difficulty for hydraulic modeling.

Fascinating composite flow interactions can occur between the floodplain and the main channel of a compound channels. The interactions involve exchange of flow between portions. They also involve the formation of large-scale turbulence and eddies in the shear layer developed in the flow region between the channel portions. By virtue of their protrusion into a compound-channel flow, abutments significantly increase local complexity of flow field, as sketched in Figure 2, developed from flume experiments at IIHR. The flow field in the vicinity of an abutment is sensitive to circumstances of abutment geometry and setting in a compound channel. Additionally, the flow field evolves as the flow substantially scours the channel around the abutment. To date, there exists little information on the local flow field at an abutment for a situation such as shown in Figure 2.

As shown schematically in Figure 2, the flow field at an abutment typically comprises an acceleration of flow from the upstream approach to the most contracted cross section somewhere at or just downstream of the head of the abutment, followed by a deceleration of flow. A flow-separation region forms immediately downstream of the abutment, and flow expands around the flow separation region until it fully re-establishes across the compound channel. Just upstream of the abutment, a flow-separation point and a small eddy may develop (Figure 2). The size of the upstream eddy depends on the length and alignment of the abutment. The curvature of the flow along the interface between the stagnation region and the flow causes a secondary current that, together with the flow leads to a spiral motion or vortex motion like flow through a channel bend. The vortex in flow around an abutment head is more localized and it has a strong scouring action. The vortex erodes a groove along its path and it also induces a complex system of secondary vortices. At abutments with wing walls (Figure 3), the flow impinging on the wall may create a downflow (similar to at a bridge pier), which excavates a locally deepened scour hole at the wall. The effect of downflow is potentially reduced for spill-through abutments.

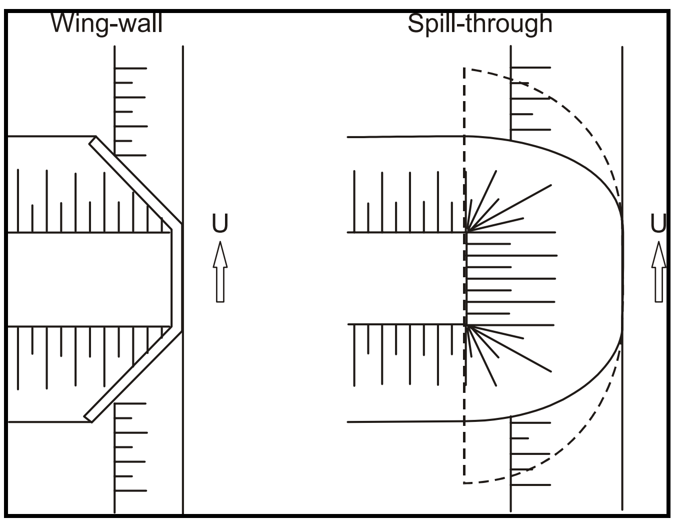

FIGURE 3. Two Basic Shapes of Abutment: Wing-Wall and Spill-Through

FOUNDATION SOILS

Foundations at abutment sites may comprise sediment and soils quite varied in their constitution and erosion behavior. For compound channels, such as sketched in Figures 1 through 3, the soils comprising the floodplain likely differ in erosive behavior from the sediment forming the bed of the main channel. Floodplain soils likely contain greater amounts of fine sediment (silts and clays), and likely are more cohesive in character than main-channel sediment. The same may be said for soils forming the embankment and the abutment. The banks flanking the main channel attest to the greater strength of floodplain soils.

Much of the complexity confronting flume experiments aimed at replicating abutment situations as in Figures 1 and 2 revolves around simulating erosion of cohesive (or somewhat cohesive) soils forming the floodplain, simulating the geotechnical slope stability of the main-channel bank and the embankment, as well as simulating scour of the non-cohesive sediment comprising the bed of the main channel.

SCOUR TYPES

Consequent to the variably complex nature of abutment flow fields and sediments and soils, several types of scour may lead to abutment failure. Field observations show that all have occurred. Figures 4 through 8 illustrate several scour-related processes that may occur at spill-through abutments (possibly, a parallel set of figures could be prepared for wing-wall abutments, though such abutments are more common for small rivers and streams). The scour types can be summarized as follow:

Type I, abutment in single channel (no floodplain)

- (I-a) Abutment threatened by local scour of main-channel bed (Figure 4)
- (I-b). Abutment threatened by local scour and contraction scour of main-channel bed (not illustrated)

Type II, abutment on floodplain (to varying extents)

- (II-a) Abutment threatened directly by scour (local and constriction) of main channel (Figure 5)
- (II-b) Abutment threatened by collapse of main-channel bank consequent to scour (local and constriction) of main-channel bed (Figure 6)
- (II-c) Abutment threatened by scour of floodplain (Figure 7)
- (III-d) Abutment threatened by embankment erosion (Figure 8)

The scour types may result from several flow conditions:

1. General scour of the main channel bed. It occurs in response to an overall propensity of the main-channel flow to degrade should an imbalance of sediment supply along the channel occur.
2. Change in main channel alignment and morphology, which adversely affects abutment location and orientation relative to flow in the main channel (e.g., a meander-loop migration may direct flow adversely towards an abutment).
3. Contraction scour of the main channel (and possibly a part of the floodplain channel) at the abutment site. Flow, constricted at the abutment site, locally scours the site, until a new balance is established between flow and bed. Contraction scour can be severe in situations where a long embankment to a bridge abutment intercepts flow over a floodplain; the intercepted flow is funneled through the abutment site.
4. Local scour attributable to the local flow field at an abutment.
5. Contraction scour of the floodplain at the abutment. Flow on the floodplain adversely impinges against the approach embankment.

A complication for flume studies and for developing reliable predictive relationships, is that these scour processes may occur at the same time, and therefore be difficult to estimate reliably (notably Types I-a and –b, and II-a and –b). Additional factors, such as variable vegetation cover and roughness of the floodplain, complicate the flow conditions.

For scour Types I-a and –b (Figure 4), abutments are threatened by scour of the main-channel bed, and by direct entrainment of material from the abutment face. The deepening scour hole may pose a slope-stability problem for the abutment. Failure, to varying extents, relates to extent of abutment-slope failure and washout. The scour may be attributed (depending on abutment length and channel width) to the combined effects of local and contraction scour processes. So far, most flume experiments have investigated this category. The simplest (relatively speaking) sub-category of this scour type is that for a rectangular abutment, or a wing-wall abutment, sited in a uniformly deep alluvial channel. In that case, abutment failure occurs when scour undermines, or reduces support for, the abutment's foundation. Most flume experiments so far have investigated this condition (e.g., as summarized in reference [*1*]).

One or more of several possible scour types may occur for abutments on, or protruding from, floodplains. Type II-a (Figure 5), illustrates essentially the same scour process as in category I-a, except that the presence of a floodplain may alter the flow field at the abutment.

Scour Type II-b (Figure 6) has received little attention, but may be common for bridges. Here, the abutment is threatened by a geotechnical failure of the main-channel bank. The failure is triggered by scour of the main channel bed at the bank. The scour could be caused by a combination of local as well as contraction scour processes, and by main-channel shifting.

Type II-c (Figure 7) is scour of the floodplain immediately at the abutment. And, Type II-d (Figure 8) is erosion of the embankment approach to the abutment.

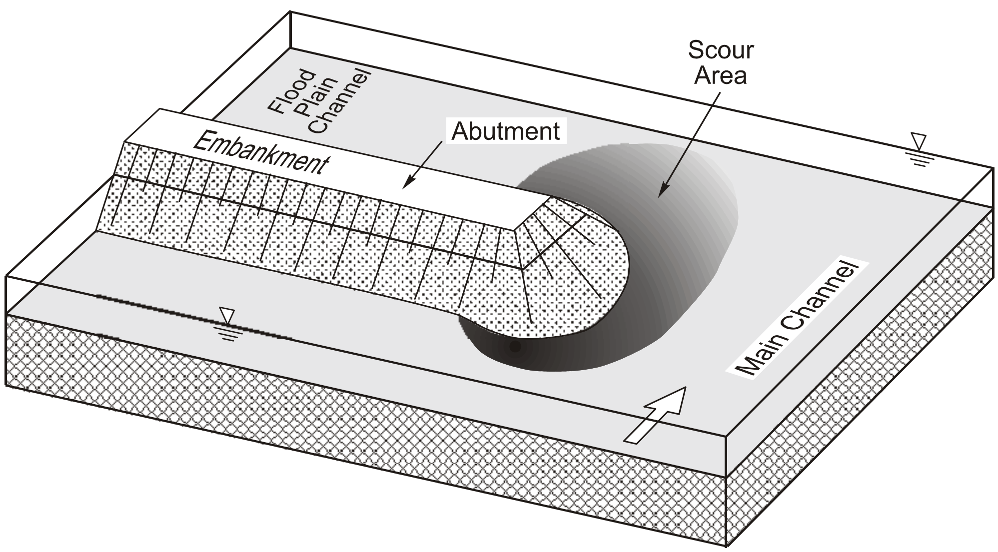

FIGURE 4. Scour Type I-a, Abutment Threatened by Scour of Main-Channel Bed (Note: scour Type I-b may include additional effect of contraction scour.)

The gallery of scour processes illustrated in Figures 4 through 7 poses considerable modeling complexities for laboratory flume investigation of abutment scour. Note that these figures depict abutment circumstances uncluttered by the additional ad-hoc complications attributable to variable vegetation, or presence of an adjacent pier or an entire other bridge, or other features (e.g., debris) influencing the abutment flow field.

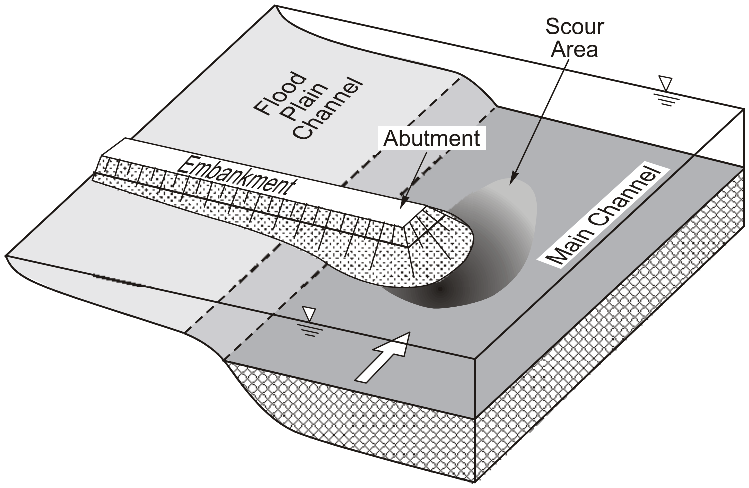

FIGURE 5. Scour Type II-a, Abutment Threatened by Scour of Main-Channel Bed

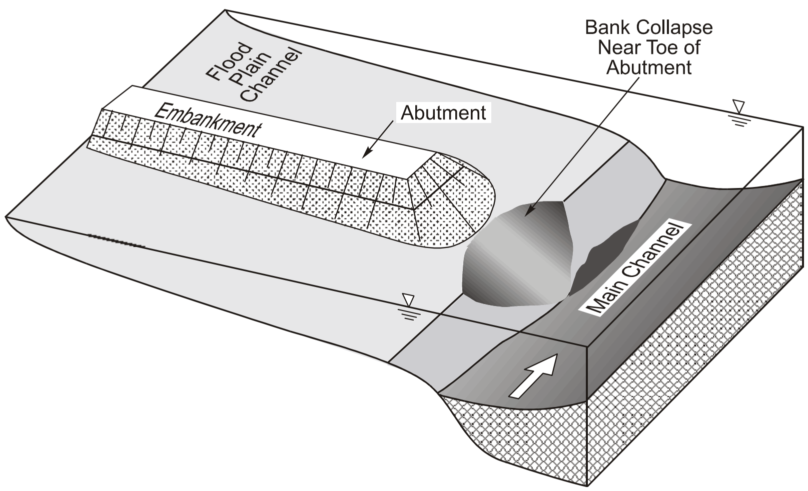

FIGURE 6. Scour Type II-b, Abutment Threatened by Collapse of Main-Channel Bank Consequent to Scour (Local and Constriction) of Main-Channel Bed

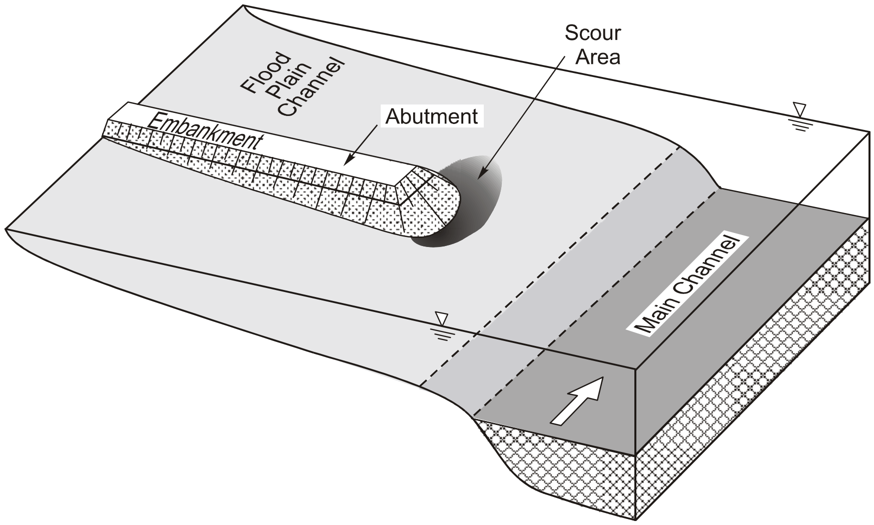

FIGURE 7. Scour Type II-c, Abutment Threatened by Scour of Floodplain

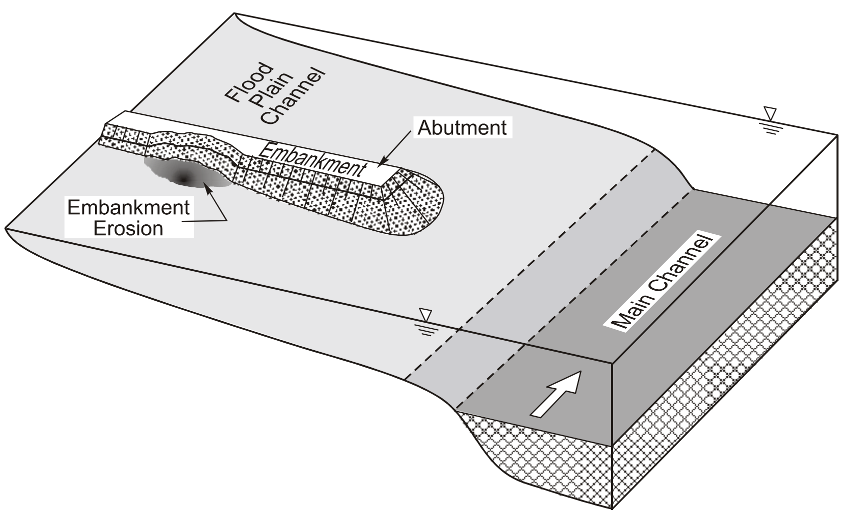

FIGURE 8. Scour Type II-d, Abutment Threatened by Scour of Embankment

SIMILITUDE DIFFICULTIES

Limitations in hydraulic-model similitude hamper the capacity of flume experiments to directly replicate many of the complexities of soil/sediment and flow conditions at abutment sites, and thereby to reproduce the abutment-failure processes shown in Figures 4 through 8. The similitude limitations stem directly from the material properties of water as well as sediment and soils. The limitations must be recognized then worked around. Presently, the leading publications on scour do not take into account the appropriate similitude concerns (e.g., references [*2, 3, 4, 1*]),

Sediments and Soils

The essential difficulties concern simulating scour of cohesive sediment and simulating slope stabilities (of main-channel bank, and of abutment and approach embankment). These difficulties especially face flume experiments intended for investigating abutments prone to Type II failures; though, actually they face all experiments on spill-through abutments, because failure of those abutments seems predominantly to occur as slope instability and collapse consequent to scour of the main channel or floodplain at the base of the abutment.

The practical limitations in simulating cohesive sediment also limit direct hydraulic modeling of slope failure (unless the slope is formed of non-cohesive sediment). Those limitations greatly complicate flume investigation of scour category Types II-c and II-d.

Flow-Field

Two not unrelated considerations complicate flume experiments of abutment scour. One consideration concerns the extent of far-field flow to be encompassed in the experiment setup. The other, a more fundamental issue, concerns flow-field similitude, notably simulation of shear stresses as well as pressures.

Flume experiments must balance considerations of extent of flow-field to be simulated, and the balance of forces acting on the flow. A difficulty is that abutment layout and size, as well as the pertinent extent of approach-channel bathymetry together with the non-uniform and the complexly turbulent nature of the approach flow (e.g., as sketched in Figure 2), and similitude constraints, require that hydraulic models of abutments practically be far-field models that encompass a substantial area of the approach channel, yet also be large enough in size as to facilitate accurate replication of flow forces. This composite requirement confronts investigators with the need to use a large (especially wide) flume for investigating Type II scour categories.

Recent work at IIHR infers that a substantial scale effect occurs in loose-bed modeling whose similitude primarily is based on intensity of bed sediment movement is used as the primary criterion for similitude, as elaborated briefly below. Ettema and Muste (*5*) also show that elevated levels of turbulence in models significantly affect flow distributions in small-scale models of dikes and wingdams. Not only does exaggerated Froude number result in increased stagnation-pressure heads, it amplifies the effect of centrifugal acceleration in regions of curved flow.

CONFRONTING THE DIFFICULTIES

The difficulties discussed above presently face the writers embarked on NCHRP Project 24-20, whose objectives are to improve reliability of abutment-scour prediction, especially for abutments straddling floodplains. The writers are configuring a plan of study that aims to address, and to work around, them. Early issues in preparing the plan, which involves extensive flume experiments, are –

1. What scour processes are pertinent?
2. How to design effective experiments for elucidating the pertinent scour processes?
3. What levels of scour-prediction accuracy are practical and even meaningful?

Preparation of the plan has involved the following series of tasks:

1. Select the most important and common scour types for spill-through and wing-wall abutments;
2. Design experiment setups for replicating the selected scour types;
3. Define the flow-field associated with each scour type;
4. Conduct parametric investigation of scour depths and extents incurred with each scour type.

Floated below for audience/reader comment are the scour types selected and the commensurate flume experiments contemplated. The experiments will be conducted in two flumes, one being 5-m wide, and the other 2.5-m wide.

Scour Types

The scour types identified to be of highest importance are Types II-a and -b (Figures 5 and 6). Also important, but somewhat less so, is Type I (Figure 4). These scour types should be investigated for through-flow and wing-wall abutment forms.

Flume-Experiments

Three basic series of experiments are centrally required. Of them, Type I scour (Figure 4), is relatively straightforward to set up in a flume, and for which quite a few prior studies have been conducted. This series can be conducted at a range of scales, and used to ascertain scale effects.

Flume experiments on Types II-a and -b scour can be investigated using a flume fitted with a fully rigid floodplain with an alluvial (erodible) main channel. The simplifying assumption used here is that the floodplain is much less erodible than the bed of the main channel. The experiments would seek to determine the extent of scour in the main channel adjacent to the abutment, but would not go so far as to replicate bank failure. The objective of the experiments would be to provide a predictive relationship for maximum depth of main channel scour. Predictions of scour depth would be given to a geotechnical engineer, who then would estimate the stability of the main-channel bank. The experiments should entail varying abutment-head location relative to the edge of the main channel. Additionally, they may entail varying abutment orientation relative to the main channel.

A variation of flume setup involving a rigid floodplain is to fit an erodible sediment recess around abutment head. The remainder of the floodplain could still be kept rigid. This setup would be used to investigate possible interaction between scour in the main channel and scour on the floodplain around the abutment.

As the flow field for scour type illustrated in Figures 5 and 6 is not adequately known, flow-field delineation is a necessary precursor task before determining predictive relationships. It is especially necessary for abutments situated on floodplains. Of prime interest in this respect are situations where the abutment is in close proximity to the main channel. In these situations, scour likely results as the combined impact of flow contraction and the flow features generated by the abutment itself.

CLOSING REMARKS

Many bridge abutments are located in compound channels whose morphology (alignment, bathymetry) is fairly complex. Additionally, many bridge abutments are located in situations where the channel is formed of various materials, occupying different locations within a bridge site. Non-cohesive sediments may form the bed of a main channel; silts and clay may predominate in riverbanks and underlying floodplains; and rocks may have been placed as riprap protection for the abutment, as well sometimes along adjoining riverbanks.

The scour-estimation relationships and guidelines presently available do not adequately take into account the complexities of channel morphology and sediment/soil disposition. Given the complexities described in this paper, the writer believes that scour-prediction relationships should aim at a level of practical approximation, whereby bridge designers may estimate reasonable upper-bound extents of scour produced by overall general (and somewhat simplified) conditions of scour; e.g., as illustrated in Figures 4 through 8. Bridge designers should be aware of the influences on scour of additional important factors such as proximity of other structures and of, say, floodplain vegetation. To include many of those factors in a predictive relationship is a Herculean task that likely cannot be completed satisfactorily.

The flume experiments outlined above aim at investigating several general, though simplified, conditions of scour at abutments. The experiments confront some scour complexities (e.g., influence of similitude), and hopefully sidesteps others (e.g., slope stability). The writer is curious to learn if the conference audience concurs with the flume-experiment approach outlined herein.

ACKNOWLEDGMENTS

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Roughness Progression Model on Kansas PCC Pavements

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ABSTRACT

Accurate prediction of pavement performance over longer time horizon represents a critical issue in the pavement surface type selection process by the Kansas Department of Transportation using the life-cycle-cost analysis. Prediction of roughness progression on the Portland Cement Concrete (PCC) pavements is very important since the current model used by Kansas Department of Transportation is based on the pavement serviceability (1993 *AASHTO Design Guide*). In this study, a statistical analysis approach was used to develop an accurate, time-dependent roughness prediction model for the newly constructed PCC pavements in Kansas. Data used in the model development process include construction and materials data as well as other inventory items, such as, traffic and climatic data. Using multiple regression analysis technique, a time-dependent roughness (International Roughness Index, IRI) prediction model was developed. The developed model produced output values that are very close to the actual (measured) IRI values ($R^2 = 0.73$). The 20-year and 30-year IRI values were also predicted. The results show that the PCC pavements with stabilized, non-drainable bases would likely outperform those with stabilized, drainable bases for majority of the projects. The sensitivity analysis conducted in this study quantified, to some degree, the impact of various key input parameters on the time-dependent PCC pavement roughness profile. Several other important conclusions were also drawn in this study.

Key words: life-cycle-cost analysis—portland cement concrete pavement—roughness progression model—sensitivity analysis

INTRODUCTION

A significant portion of funds is spent on pavement reconstruction. The construction program for the jointed plain concrete pavements (JPCPs) in Kansas is in the range of $150 to $200 million (*1*). The pavement surface type selection process of the Kansas Department of Transportation (KDOT) uses life-cycle-cost (LCC) analysis. Performance prediction of JPCP over a longer analysis period (30 years as minimum) is required in order to compare the life cycle costs of competing surface types. Performance prediction models for JPCP are now lacking in Kansas since new construction practices (shorter slabs, dowel bars, etc.) started in the early 1990's. Identifying and selecting appropriate strategies that can potentially perform better than others would result in high benefit return (*2*).

In the past, many studies have been conducted in order to develop performance (i.e., pavement distress and roughness) prediction models for JPCP. Current performance prediction and analysis models involved in predicting PCC pavement distress indices are primarily an improvement over the models used in the prototype performance-related specifications (PRS) for JPCPs. Using version 2.0 of the PaveSpec PRS demonstration software, Yu et al. (*3*) established that the prototype PRS was improved. As a result, the PaveSpec PRS demonstration software was upgraded to Version 3.0. In the case of the development of Performance-Related Specifications for JPCP, the specific data elements required by distress indicator models were summarized. Depending on the required data elements, such models predicted the development of joint spalling, faulting, slab cracking, and pavement smoothness over time (*4*). Similarly, using local Microsoft Access database, Titus-Glover et al. (*5*) developed improved pavement distress and roughness prediction models that incorporate mechanistic principles but that are still practical for use by State highway agencies. Likewise, Yu et al. (*3*) used the ORACLE data base management system to evaluate the performance of 303 in-service concrete pavement sections located throughout North America. Current efforts should be concentrated on the development of enhanced and improved pavement models to predict the time-dependent pavement roughness profiles.

DATABASE DEVELOPMENT

Table 1 lists the 24 PCC projects, on state, US and Interstate routes in Kansas, selected in this study. The projects were constructed between 1993 and 1997. All pavement sections are JPCP with 4.6 m (15 ft.) joint spacing and doweled joints. The JPCP slab thickness varied from 229 mm (9 in.) to 292 mm (11.5 in.). The database developed for each section includes annual roughness values measured during each year after construction and a number of design, construction, and climatic variables. Longitudinal profile measurements were done on the right and left wheel paths with a South Dakota–type Profilometer. From these profile data, the International Roughness Index (IRI) values were calculated. Only the right wheel path IRI values were used in this study.

The data was categorized and assembled into an Excel spreadsheet for statistical analysis. Initially, 56 different potential independent variables were considered. However, after data cleansing and thorough examination, the database was limited to 26 practical input variables. The selected input variables include pavement profile (initial smoothness, represented by initial roughness IRI value), pavement section and layer data, time-series traffic, subgrade type and treatment, climatic conditions, shoulder type, concrete mixture design data and concrete materials test results. Pavement layer data included the PCC slab thickness and type of base, such as, portland cement treated base (PCTB) and bound drainable base (BDB). Time-series traffic data refers to the cumulative 80-kN (18-kip) equivalent single-axle loads (ESALs). Climatic information was included in order to develop a model that can directly account for the section-specific climatic conditions. The output variable was the time-series (i.e., annual) right wheel path IRI value, which is used to quantify the long-term pavement roughness performance.

TABLE 1. Selected PCC Projects in the Study

Project K Number	County	Lane	Route	Construction Date	Begin Milepost	End Milepost
K-2633-01	Lyon	East	I 35	1994	10.9	16.7
K-2633-01	Lyon	West	I 35	1994	10.9	16.7
K-3596-01	Franklin	East	I 35	1995	0	3.2
K-3596-02	Franklin	East	I 35	1995	3.2	9
K-4088-02	Johnson	East	I 35	1996	13	16
K-4088-02	Johnson	West	I 35	1996	13	16
K-2446-01	Shawnee	North	I 70	1993	11.7	15
K-3344-01	Shawnee	South	I 70	1993	9	10
K-2447-01	Wyandotte	North	I 70	1993	15.6	17.1
K-2447-01	Wyandotte	South	I 70	1993	15.6	17.1
K-3637-01	Johnson	West	I 435	1996	0	3.3
K-4058-03	Harvey	Undivided	US 50	1995	28.7	35.6
K-3216-02	Chase	Undivided	US 50	1997	0	9
K-3217-02	Chase	Undivided	US 50	1997	9	19
K-4422-02	Ford	Undivided	US 56	1996	12.2	16
K-3251-01	Jackson	East	US 75	1995	8	17.3
K-3251-01	Jackson	East	US 75	1995	12	17
K-3251-01	Jackson	West	US 75	1996	8	17.3
K-3251-01	Jackson	West	US 75	1996	12	17
K-4341-01	Shawnee	East	US 75	1996	20	22
K-4341-01	Shawnee	West	US 75	1996	20	22
K-3220-01	Marion	Undivided	US 77	1995	11.1	11.8
K-3684-01	Sedgwick	West	K 15	1997	0	5.7
K-4460-01	Sedgwick	North	K 96	1996	3.9	14.7

Table 2 lists the variables used in modeling into the following groups: (a) inventory, (b) construction, and (c) climate. The climatic data was obtained from the Kansas State University Weather Data Library. The historical roughness data was obtained from the KDOT PMIS database.

TABLE 2. Data Elements Selected as Independent Variables for PCC Pavements

Inventory	Construction	Climate
-County code -Route No. -Project No. -Begin milepost -End milepost -Project length -Cumulative AADT (year) -Cumulative truck factor (year) -Cumulative yearly ESAL values* -Initial right wheel path, IRI roughness (in./mile)* -IRI roughness value at age (n) year (in./mile)*	-Age of pavement (year)* -PCC slab thickness (in.)* -Base thickness (in.) -Plasticity index of natural subgrade soil material* -% Material passing No.4 sieve* -% Material passing No.200 sieve* -Subgrade treatment: *No treatment (N/A) (=0)** *6" lime treated subgrade (=1)** *Subgrade modification (=2)** -Unit weight (lbs./ft^3)* -Flexural strength (PSI) -Total width of outside shoulder (ft.) -Shoulder type (paved=1 or unpaved=0) -Shoulder thickness (in.) -Drainable base (=1) or non-drainable base (=0): (Base material/treatment type) *Cement Treated Drainable Base (CTDB)=1** *Bound Drainable Base (BDB)=1** *Edge drain=1** *Portland Cement Treated Base (PCTB)=0** *no edge drain=0** -Pavement cross-slope (1.6%) -Water-cement ratio -Slump (in.) -Air content (%) -Cement factor (lbs./yd^3)* -Weight percentage of coarse aggregate in mix -Weight percentage of fine aggregate in mix	-Cumulative annual total precipitation (in.) -Cumulative total no. of days below 32 ^{o}F/yr* -Cumulative total no. of days above 90 ^{o}F/yr* -Cumulative no. of wet days/year (more than 10 mm precipitation)* -Avg. no. of freeze-thaw cycles per year* -Mean annual temperature (^{o}C) -Max. annual temperature (^{o}C) -Minimum annual temperature (^{o}C)* -Depth of frost penetration (in.)

* independent variable used in models.

REGRESSION EQUATION METHOD

In order to develop roughness prediction equations for the PCC pavements in Kansas, linear regression analysis was used to find the best relationship between the independent variables and the dependent variable. The backward selection procedure in SAS (*6*) computer program was selected for multiple regression analysis. This method starts with a full model (all independent variables entered) and then eliminates one variable at a time until a reasonable good regression model is selected. The selected model contains the most significant independent variables.

The following model for predicting future IRI of the PCC pavements was obtained:

$$\begin{aligned} IRI = {} & 218.38 - 0.61*FSI - 0.07*TSI + 7.88*MIAT \qquad (R^2 = 0.73) \\ & + 9.10*SLTH + 8.45*BTY + 1.64*AP + 0.78*IIRI \\ & - 0.01*WET - 1.673e - 7*ESAL + 11.97*SUBTRT\ , \end{aligned} \tag{1}$$

where

IRI = yearly right wheel path roughness IRI value
FSI = % subgrade materials passing No.4 sieve
TSI = % subgrade materials passing No. 200 sieve
MIAT = minimum annual temperature (^{o}C)
SLTH = PCC slab thickness (inch)
BTY = drainable base (=1) or non-drainable base (=0)
AP = age of pavement (year)
IIRI = initial right wheel path IRI (in./mile)
WET = cumulative no. of wet days per year (more than 10 mm precipitation)
ESAL = cumulative yearly ESAL values
SUBTRT = subgrade treatment:
no treatment (N/A) (=0)
6” lime treated subgrade (=1)
Subgrade modification (=2)

The IRI prediction model yielded a coefficient of determination, R^2 of 0.73 as shown in Figure 1. The 20-year and 30-year IRI values were predicted, and the R^2 value for each project was also calculated. Table 3 shows the predicted IRI values. It appears that the projects with stabilized, non-drainable bases would likely outperform the projects with stabilized, drainable bases for majority of the projects.

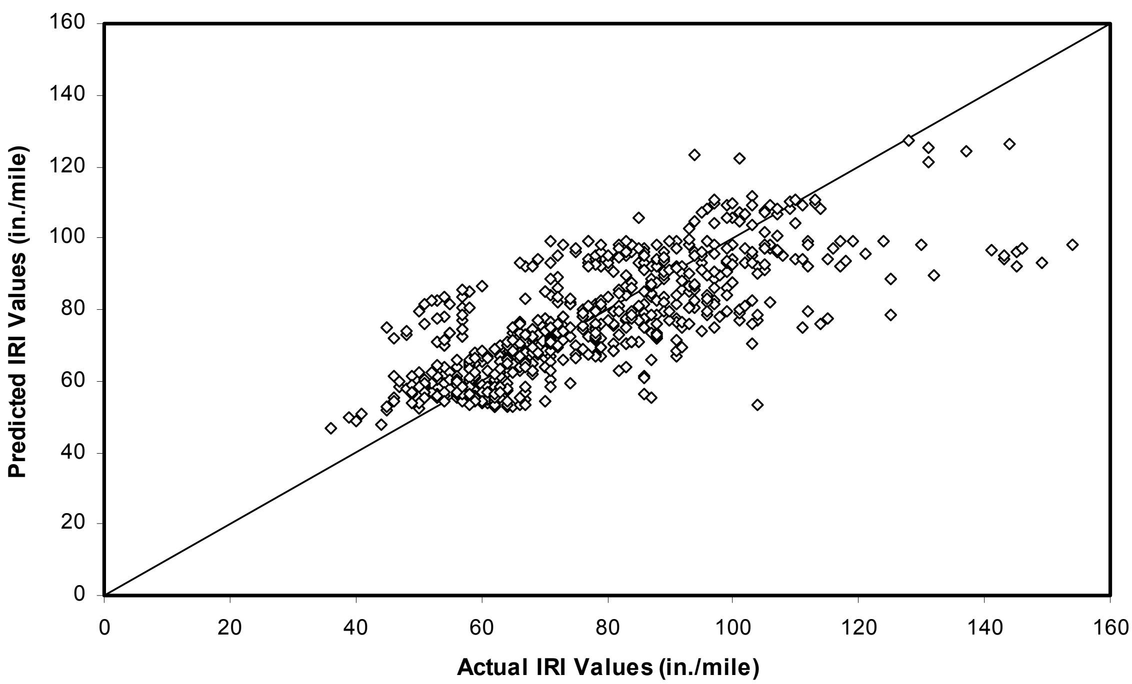

FIGURE 1. Comparison between Actual and Predicted IRI Values for the SAS-based Prediction Equation (R^2=0.73)

TABLE 3. Future 20-Year and 30-Year IRI Using SAS-Based Prediction Equation

Route	Lane	Initial IRI (in./mile)	20-Year IRI (in./mile)	30-Year IRI (in./mile)	R^2
I 35	East	52	96	106	0.07
I 35	East	90	100	110	0.35
I 35	East	97	92	101	1.00
I 35	West	100	123	132	1.00
I 35	East	84	111	121	0.99
I 35	West	99	111	121	0.99
I 70	North	46	76	86	0.95
I 70	South	36	66	76	0.98
I 70	North	110	120	129	0.99
I 70	South	97	120	129	0.99
I 435	West	80	92	101	1.00
US 50	Undivided	57	82	91	0.83
US 50	Undivided	71	93	102	0.51
US 50	Undivided	93	108	119	0.75
US 56	Undivided	97	113	125	0.03
US 75	East	71	87	95	0.89
US 75	West	67	87	95	0.89
US 75	East	71	87	96	0.85
US 75	West	68	83	92	0.89
US 75	East	68	90	100	0.38
US 75	West	85	92	103	0.61
US 77	Undivided	131	141	151	1.00
K15	West	50	74	84	1.00
K 96	North	67	73	83	1.00

* Drainable base: edge drain, cement treated drainable base (CTDB), bound drainable base (BDB).
** Non-drainable base: no edge drain, portland cement treated base (PCTB).

SENSITIVITY ANALYSIS

In order to assess the impact of each independent input variable on the time-dependent IRI profile, a sensitivity analysis was performed (Figure 2). To accomplish this objective, a PCCP section with average input values was chosen. The sensitivity analysis then determined the effects of three levels, minimum, median, and maximum, of each independent variable while keeping all other input variables stationary. As shown in Figure 2, the PCC slab thickness and the initial roughness have greater impact on the roughness profile than percent subgrade materials passing US No. 200 sieve and cumulative yearly ESAL values. PCC pavements with lower slab thickness tend to sustain the smoothness longer. After seven years of service, other things being constant, an increase in the PCC slab thickness by 2.5 in. (11.5 inches vs. 9 inches) will increase the predicted roughness by 23 in./mile. Similar observations have also been made by Siddique et al. (*7*) for some other Kansas PCC pavements and by Perera and Kohn (*8*) for the PCC pavements in the LTPP program. Also, PCC pavements built with lower IRI values tend to sustain smoothness longer, too. Subgrade soils with a high amount of materials passing the US No. 200 sieve tend to remain smoother. This may appear to defy common experience. However, it is to be noted that those soils will generally have higher plasticity. In that situation, KDOT usually would require some form of subgrade treatment/stabilization using lime or lime and fly ash to reduce volume change potential under varying moisture conditions. Thus, treated subgrade would be beneficial for sustaining smooth PCC pavements. Also, the developed model is not highly sensitive to the traffic loading parameter (ESAL).

CONCLUSIONS

Roughness prediction models were developed in this study for Jointed Plain Concrete Pavements (JPCP) in Kansas using historical roughness, traffic, inventory, and climatic data. Roughness predictions were done for 20-year and 30-year horizons with good coefficients of determination value in most cases. The results show that the projects with stabilized, non-drainable bases would likely outperform the projects with stabilized, drainable bases for most cases. Higher PCC slab thickness would result in higher future roughness values. The same would happen for higher initial (as constructed) roughness. The future predicted roughness did not appear to be very sensitive to the traffic.

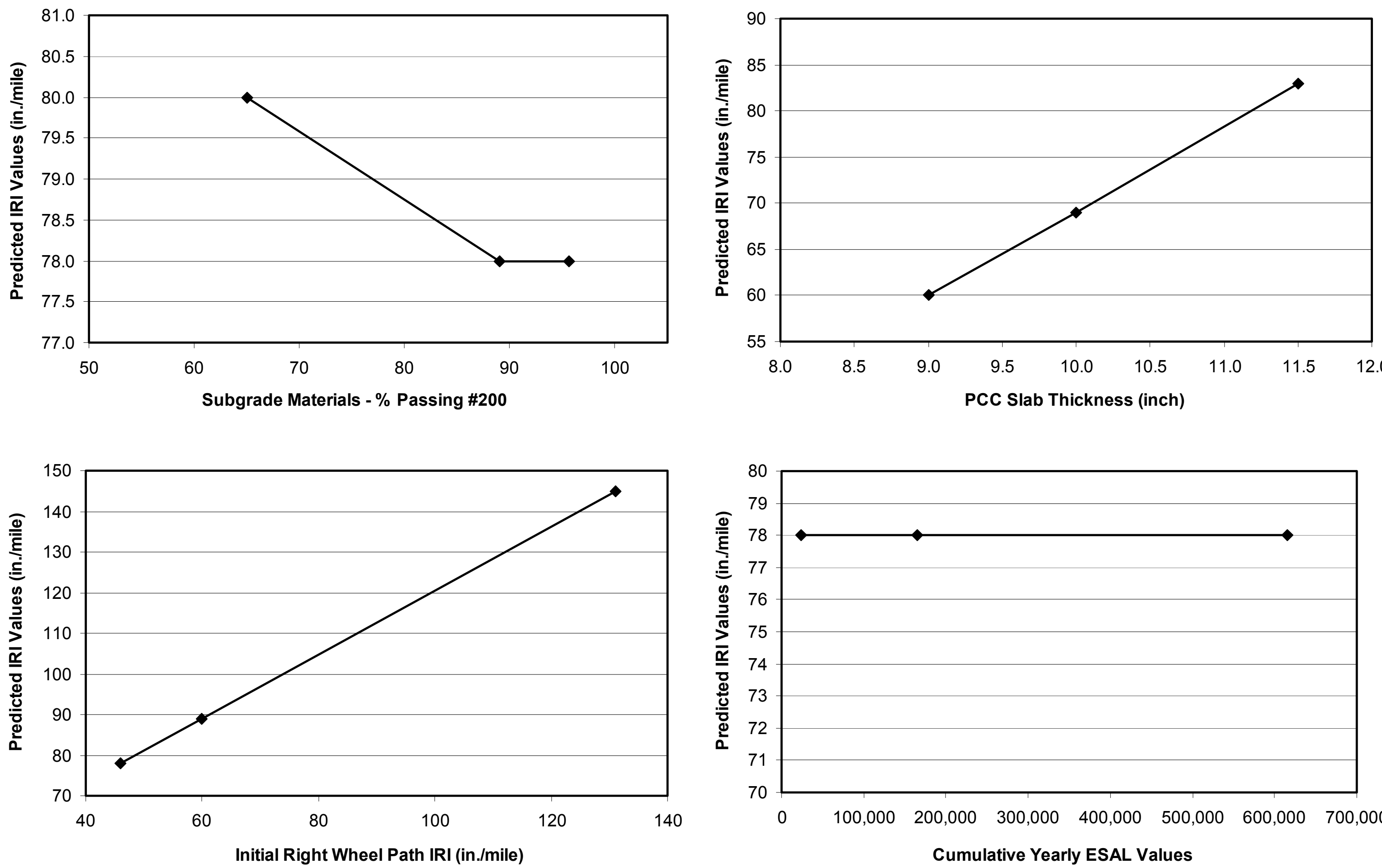

FIGURE 2. Sensitivity Analysis Results

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Long-Term Plan for Concrete Pavement Research and Technology

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ABSTRACT

A new generation of concrete pavements is coming of age, thanks in part to an innovative, long-term CPTP plan for research and technology. Envisioning what we want concrete pavements to look like in the next 20, 50, or 100 years and developing a long-term research plan to make the vision a reality will require the industry to examine itself closely—and perhaps even redefine itself.

These and other questions are guiding a national project to develop a far-reaching road map to tomorrow's concrete pavements. Under the Transportation Equity Act for the 21st Century, Congress authorized the Federal Highway Administration (FHWA) to undertake a significant research program to improve the performance of concrete pavements. The Innovative Pavement Research Foundation, an industry research consortium, and FHWA are sponsoring the development of a Long Term Plan for Concrete Pavement Research and Technology under the joint Concrete Pavement Technology Program (CPTP). The CPTP selected an Iowa State University-led team of program planners, university researchers, engineering consultants, and practitioners from industry and public agencies to create the plan.

Key words: long-term plan—performance—portland cement concrete pavement

Note: Preparation of this paper was still in progress at the time of publication; final results will be presented at the symposium.

FHWA Bridge Research and Technology Deployment Initiatives

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ABSTRACT

The Federal Highway Administration (FHWA) has long-standing technology programs and continues to support both short-term and long-term research needs. In the area of bridge engineering, FHWA's current bridge research and technology research and deployment thrusts include: (1) accelerated repair and construction, (2) bridge and tunnel security, (3) bridge system preservation (management, inspection, maintenance and rehabilitation), and (4) load and resistance factor design (LRFD) implementation. The FHWA is also currently developing a more formal structure for stakeholder involvement in research agenda-setting and monitoring, and in the delivery and deployment of new technologies to the bridge engineering community.

This paper provides an overview of the FHWA's current bridge technology focus areas, discusses selected studies and results, and proposes a mechanism for future involvement of bridge engineering stakeholders in a formalized research and technology deployment process.

Key words: bridge—engineering—research—technology

INTRODUCTION

The Federal Highway Administration (FHWA) has a long history of promoting highway technology and facilitating the construction of the network of roads and bridges within the United States. The focus on new highway construction peaked in the mid-1960s, during the development of the Interstate Highway System. With the Interstate System substantially complete, the focus of the FHWA has shifted from new construction to one which has an increased emphasis on broader and more diverse U.S. highway transportation needs, in order to enhance user mobility and interstate commerce, ensure safety to the traveling public, and promote efficiency and intermodal connectivity within the system. The current focus is also has FHWA providing technology solutions to its State and industry partners, which is intended to provide support in solving short-term problems, and in laying the groundwork on which new technologies will be developed for the U.S. highway industry in the longer term.

Research and technology on the physical elements of highway infrastructure is particularly important. Highways accommodate 90% of all personal travel and more than 80% of freight, and are thus critical for both personal mobility and commerce. Demands on the system increase annually and traffic volume growth has been dramatic both for passenger vehicles and freight. Meanwhile, the highway system infrastructure, including bridges, retaining structures, and tunnels, continue to age.

Bridges within the U.S. highway system are, on average, more than 40 years old. The majority of these structures were designed with a theoretical 50-year design life and yet are assumed to have essentially an infinite life. As these structures continue to age, their needs continually increase. This is particularly important when additional demands are introduced through deterioration of structural components and changing traffic demands. Today, more than 25% of the inventory of nearly 600,000 highway structures contained within the National Bridge Inventory (i.e., more than 150,000 bridges with a length in excess of 20 ft) is classified as structurally or functionally deficient and in need of repair, rehabilitation or reconstruction. Sufficient funds are not available to address the current level of needs; meanwhile, we are faced with a rapidly growing increase in the number of structures requiring work and in the complexity of work required to provide unimpeded access to these highway structures.

In order to meet the challenge of improving our highway bridge network in the face of increased demands, continued deterioration, and constrained funding, things must be done differently. Bridge owners will need to work smarter and more efficiently in their maintenance and reconstruction programs. They will, however, need technological solutions to assist in this effort, and both short- and long-term research and development efforts are required to provide these technology solutions. In the short-term, there are many recently developed technologies and innovations that can be put into practice to enhance system performance. Longer-term, higher-risk, and potentially higher-payoff research is also required to complement these short-term efforts, in order to provide the breakthrough technologies that will be needed for decision support to meet the challenges of the future.

The FHWA historically partners with AASHTO, State transportation agencies, the Transportation Research Board (TRB) and its National Cooperative Highway Research Program (NCHRP), and private industry and academia in research and technology. It will continue to sponsor, sustain, and guide research and technology efforts to improve and sustain the Nation's highway infrastructure.

In this regard, the current priority focus areas have been identified by the FHWA for highway bridges

- accelerated Repair and Construction Technologies: in order to decrease the impact on the traveling public, accelerated repair and construction technologies and approaches are required. Bridges must be repaired or replaced in less time and with less cost, in order to decrease the impact on the traveling public and maximize the use of limited available funds.

- bridge and tunnel security: vulnerabilities within the inventory of structures must be identified and mitigated to ensure adequate levels of highway system performance following the occurrence of natural or manmade extreme events.

- bridge system preservation: as noted above, the inventory of existing structures is aging, and limited funds are available for rehabilitation and replacement. In order to "get ahead of the deterioration curve," smart preservation strategies must be employed in order to prevent structures from becoming deficient, and decision support algorithms must be implemented to assist decision makers in optimizing the expenditure of limited funds.

- implementation of load and resistance factor design: previous bridge design codes and load rating procedures only minimally considered structure safety and reliability in an explicit manner. Current design philosophies feature the use of limit states, multiple load and resistance factors, and a semi-probabilistic determination of a structure's reliability. The Load and Resistance Factor Design (LRFD) approach has been adopted by AASHTO for the design of new bridges, and AASHTO has indicated its intent to standardize on LRFD design nationwide by 2007. Efforts are underway within FHWA to facilitate and assist bridge owning agencies to effectively implement the LRFD philosophy.

Each of these priority areas are discussed in further detail below. The most critical aspect of ensuring success in the deployment of these technologies, however, is to ensure adequate "stakeholder" involvement at every stage in the research identification, development, and deployment process. As a result, FHWA is currently developing a proposed new mechanism for better stakeholder involvement in FHWA research and technology activities, which is also described below.

RESEARCH PRIORITY AREAS

Accelerated Repair and Construction Technologies

The need for accelerated repair and construction is clearly evident when considering urban environments. Traffic growth in cities throughout the country has reached levels so significant that mobility disruptions occur any time that activities are initiated. It becomes difficult to close lanes to repair a structure without causing extremely large traffic disruptions, even in the late evening hours or overnight. Rapid repair techniques are required to strengthen a structure within a limited time-frame in order to minimize traffic disruption and the associated impacts on mobility and commerce. These techniques should also minimize the potential for future interventions. Consider that, today, a new segment of highway typically takes years to construct —sometimes as many as 10 years for a 10 mile stretch of pavement which may have 5 bridges on it; why can't this be decreased to less than 1 year using innovative design, contracting, and construction approaches?

It is for this reason that the current FHWA "mantra" is to "get in, get out, and stay out." A number of studies have been performed in recent years intended to address the needs of accelerated repair and construction. This has culminated in a focused program being supported both by FHWA and AASHTO in accelerated repair and construction. From a bridge technology standpoint, much of the current effort is on the appropriate application of high performance materials, smart foundation construction technologies, and prefabricated bridge components and systems.

High performance materials typically include high performance concrete (HPC), high performance steel (HPS), and fiber reinforced polymer (FRP) composites. High performance does not necessarily mean, nor does it necessarily exclude, properties that relate to high strength. High performance does however include properties that relate to significantly improved constructability, durability, and resistance to environmental and vehicular degradation.

HPC and HPS members and structures can be made lighter and possibly simpler than those comprised of conventional concretes and steels, thereby easing construction and facilitating rapid application. These advanced structural materials can also be engineered to eliminate problems with the traditional materials; e.g., with increased density or lower permeability which reduces chloride intrusion, or with improved weldability and increased fracture toughness. HPC members can be constructed with rapid-curing properties, allowing structures to be opened to traffic more quickly, thereby minimizing the impact on the traveling public.

FRP composites, which are used extensively in aerospace and military structural applications, are making inroads into the civil and bridge engineering infrastructure. In its FRP composite research program, FHWA seeks to advance the technology for new construction, rehabilitation and preservation, and strengthening of the existing inventory. FRP composite bridge decks and structural members are typically much lighter, and can be preassembled to facilitate construction. The use of the FRP materials for repair and strengthening of structural concrete members is increasing and, when properly designed and applied, can be effective and efficient.

For a growing number of States, the use of prefabricated bridge elements and systems is also helping to achieve reduced traffic and environmental impacts, enhanced safety, improved bridge constructability, and accelerated project completion. Prefabricated bridge elements, which range from bent caps to deck panels, and superstructure and substructure systems, are manufactured under controlled conditions and brought to the construction site ready to install. Prefabricated elements are particularly useful in situations where traditional cast-in-place construction would have to be sophisticated and expensive; e.g., at long water crossings or complex interchanges. Prefabrication also facilitates construction in urban settings, where work space is limited. Safety is improved and traffic impacts are lessened because some of the construction is moved from the roadway to a remote site, minimizing the need for lane closures, detours, and use of narrow lanes.

Bridge and Tunnel Security

For bridge engineers, protecting bridges and other structures used to mean guarding against such processes as fatigue, scour, and earthquakes. The events that occurred on September 11, 2001, changed all that – as engineers are now also faced with the challenge of how to protect structures from potential terrorist attack. Bridge and tunnel engineers throughout the United States are being asked to assess the vulnerability of their structures, and to provide solutions to reduce this vulnerability. To equip engineers to answer these new and complex questions, FHWA has launched a number of structural security initiatives.

Among these is a workshop developed in cooperation with the U.S. Army Engineer Research and Development Center (ERDC). For the past 35 years, the ERDC has carried out structural vulnerability research and development work, which has included numerous full-scale explosive tests on bridges and tunnels, the analysis of complex structural data, and developing appropriate computer tools. FHWA and the ERDC have created an ongoing workshop series intended to train FHWA, state, and private sector engineers on topics which range from bridge and tunnel vulnerability to explosive attack, structural response to blast-induced loadings, and vulnerability predictive tools and mitigation methods.

The FHWA has also been working in partnership with AASHTO under the flag of the AASHTO/FHWA Blue Ribbon Panel on Bridge and Tunnel Security. The panel is developing short- and long-term strategies for increasing the security of critical bridges and tunnels, including implementing design and retrofit techniques, and is identifying current and future research needs. Opportunities exist for research to better understand blast loading and the effectiveness of retrofit techniques, the development of enhanced risk analysis procedures, introduction and use of information technologies to mitigate and prevent damage due to terrorist activities, and other areas to reduce the threat and the exposure. The results of this effort will be used to establish a roadmap for prudent, cost-effective measures to make our transportation system more secure.

Bridge System Preservation

With an aging bridge population, appropriate decisions must be made regarding when and what type of maintenance to conduct on a large number of bridges; and whether rehabilitation or replacement is a more appropriate strategy for other bridges. In order to address this need, rigorous decision support systems are required and more detailed and informative bridge inspection information is necessary. The FHWA has been performing research in these areas for a number of years and, through this effort, many new applications have been developed and implemented. Research continues to be conducted in order to enhance the current bridge management system decision support algorithms and to develop and implement inspection technologies that provide better characterization of structural damage.

Improved bridge and tunnel inspection technologies are being developed via the Non-Destructive Evaluation Validation Center (NDEVC), which is located at the FHWA's research facilities, the Turner Fairbanks Highway Research Center. With more accurate, non-contact and non-destructive inspection techniques, damage can be characterized at earlier stages so that life-preserving corrective actions can be taken. Among recent techniques for quantitative, non-subjective non-destructive evaluation of bridges which have been developed through the NDEVC are

- ultrasonic systems which send waves into the material to detect cracks;

- eddy current testing which creates a magnetic field that opposes the magnetic field of the original material – locations of cracks will appear where the created impedance diverge;

- bridge acoustic emission Local Area Monitoring, which can be used to track known defects, whether corrective measures are working, and to monitor structural integrity of welds and connection points;

- laser measurement technologies to track the deformation of the material under stress;

- ground-penetrating radar, which can be used to show variances in material types, is being tested for application in bridge deck inspections to identify areas of delamination; and

- active thermographic crack detection which detects changes in temperature when heat is applied to a material – these changes in temperature are a result of cracks allowing the escape of heat.

A number of other activities are also underway at FHWA to support bridge system preservation strategies. Research to develop technologies which provide relatively simple repairs for deteriorated or cracked concrete or steel members has been an ongoing program for many years. Improvement of and support for bridge management systems and approaches is an important part of the FHWA bridge program.

To support the bridge system preservation initiative, FHWA and AASHTO jointly sponsored an international scanning tour in the spring of 2003. The FHWA/AASHTO Panel on Bridge System Preservation and Maintenance, which was comprised of ten members representing AASHTO, FHWA, State DOTs, the National Association of County Engineers, and academia, traveled to the African and European continents and met with highway agency representatives, and bridge management and inspection technology practitioners and researchers, from South Africa, Switzerland, Germany, France, Denmark, Sweden, Finland, Norway, England, and Wales. Specific topics of interest addressed during the scanning tour included

- organizational, policy and administrative issues including: relationship among agencies (national, local); organization of their bridge activities (design, construction, operation, inspection); inventory ownership and management; inventory characteristics (number, type, materials, span lengths); and inspection type, frequency, and rigor;

- status of their bridge management systems (BMSs) including: economic modeling and forecasting; deterioration modeling; and information technology (databases, architecture, input, and updating);

- inspection issues and practices including: typical practices; innovative methods; use of non-destructive evaluation (NDE) technologies; use of load testing; design for inspection (e.g., accessibility) and "smart" bridges; and

- operations issues and practices including: permit vehicles; load rating and load posting; indicators of performance and their relationship to design, maintenance, repair, and enforcement.

A number of important policy and operational issues that could have a significant impact on U.S. bridge management practices were identified during the scanning tour, and will be further evaluated and discussed with appropriate U.S. bridge owning and operating agencies. Included in these are: setting a rational bridge inspection frequency based on bridge type and consequence risk; defining minimum bridge inspector qualifications and training; development of integrated highway structure management approaches which include bridges, tunnels, free-standing retaining walls, sign and light structures, etc.; and, application and use of appropriate waterproofing systems for bridge deck protection.

The AASHTO/FHWA Panel is currently preparing a draft report documenting the findings of the Scanning Tour, which should be ready for dissemination and technology implementation by AASHTO and the FHWA near the end of 2003.

Implementation of Load and Resistance Factor Design

The bridge engineering profession is currently moving to the Load and Resistance Factor Design (LRFD) philosophy. The AASHTO LRFD Bridge Design Specifications employ state-of-the-art analysis and design methodologies, and make use of load and resistance factors based on the known variability of applied loads and material properties. The load and resistance factors contained in these specifications are calibrated from statistics obtained from the inventory of existing bridges, and are intended to provide a uniform level of safety. Bridges designed with the LRFD specifications should therefore lead to a higher level of serviceability and maintainability.

As LRFD will positively impact the safety, reliability, and serviceability of newly designed bridges, AASHTO has set a transition date of October 2007 by which time all new bridges must be designed with the LRFD Specifications. The FHWA has therefore developed a strategic plan to assist and facilitate State transitioning to LRFD by the 2007 target date. This strategic plan includes

- identifying past, current, and future LRFD implementation plans of States;
- identifying and deploying a showcase of successful LRFD implementation approaches by State which have completed their transitioning;
- developing a model implementation plan and guidelines that can be used by States to identify and prioritize items to be accomplished for successful LRFD implementation; and to make decisions, set priorities, determine actions, and review performance on a regular basis;
- deploying planning assistance by providing hands-on implementation and transition planning that integrates a State into the detailed implementation planning process;
- developing two comprehensive design examples – one each for a steel and concrete bridge;
- deploying technical LRFD training and assistance to States;
- developing detailed, hands-on training courses, to be delivered by the FHWA's National Highway Institute;
- compiling and maintaining a comprehensive list of LRFD resources (books, software, courses, etc.); and
- supporting LRFD research.

A wealth of information supporting LRFD transition activities can be found on the following website: http://lrfd.aashtoware.org

STAKEHOLDER INVOLVEMENT IN FHWA RESEARCH AND TECHNOLOGY

Stakeholder involvement is critical to the success of both the short- and long-term research and technology deployment efforts undertaken by the FHWA. Past efforts to involve stakeholders in bridge research and technology programs has been somewhat ad hoc in nature, although there have been fairly high levels of involvement over time. Current efforts within FHWA are focused on the creation of a more structured and proactive mechanism to ensure adequate stakeholder involvement in research agenda setting, research progress and output monitoring, metrics for measuring success, and in the effective deployment of new technologies.

FHWA's research and technology stakeholder model assumes the following characteristics: that involvement is strategic rather than tactical; long-range rather than immediate; continuous rather than periodic; provides a process that is open, visible, and transparent; and is representative of broad range of stakeholders and needs. We are looking to: involve recognized leaders in bridge research and technology deployment; provide an equal voice for all participants; ensure proactive participation in the process; and for stakeholders to provide additional insights on issues of a technical, operational, political, user constraint, cost basis.

FHWA envisions the role of stakeholders in its research and technology programs as

- providing focus and direction for the program via a peer review process;
- identifying needs and gaps, opportunities, innovations;
- setting priorities;
- evaluating quality and value; and
- identifying champions to assist in new technology deployment.

However, this is an evolving stakeholder involvement model and process, and there are still are large number of unresolved issues and concerns which must be addressed before the process is formalized. Among these are

- the level and source of funding necessary to support research and technology activities, committee involvement, and FHWA staff time;
- the amount of time and level of effort which will be required of stakeholders and FHWA staff;
- the size, representation, and rotation rules for stakeholder committees and panels; and
- whether this stakeholder structure is operated by FHWA, TRB, AASHTO, or another organization to ensure independence and effective interaction.

The FHWA is currently encouraging the bridge engineering stakeholder communities to share their thoughts regarding an appropriate stakeholder involvement model, and we welcome your input.

CONCLUSION

The need for a focused highway bridge and tunnel research and technology program cannot be understated. With significantly escalating demands and reduced resources, “business as usual” is not sustainable. The FHWA has a number of high priority bridge engineering technology development and deployment areas, of which four are briefly described herein: (1) accelerated repair and construction, (2) bridge and tunnel security, (3) bridge system preservation, and (4) implementation of load and resistance factor design.

The driver for these bridge technology programs is the needs of the bridge engineering stakeholder community. It is recognized that all of these programs require a high level of stakeholder involvement, which can provide input on the broad range of research and technology deployment activities. Among the areas for inclusion of stakeholders are: policy and agenda setting, project scoping and merit review, research evaluation, and deployment and implementation.

A Dynamically Configurable Network-Scanning Tool for the Spatial Cluster Analysis of Linear Network Feature Corridors

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ABSTRACT

Many current state-of-the-practice crash analysis methodologies designed for crash clustering analysis are limited by several key data topology factors. These factors are products of limited analysis software capabilities and data production and maintenance methods and do not lend themselves well to producing objectively comparable corridor ranking results.

This research wishes to address the issues associated with statically segmented spatial network analysis. It is expected that a dynamic linear feature of homogeneous length and capable of 'walking' a vector-based network topology at discreet increments would produce analysis results that are more objectively comparable and therefore yield better informed safety improvement decision-making.

To test this hypothesis, a software tool implementing the aforementioned behaviors is being developed. Whereas similar tools have been previously developed, it is anticipated that user interface design and the greater amount of analyst interaction afforded by the software architecture will further advance the state of the art in this arena. In addition, this research eventually intends to explore the potential benefits of coupling this type of tool with pattern-recognition software such as Artificial Neural Networks for spatially contextual, dynamic, and graphically interactive data-mining operations.

As this research is in its early stages, the focus of this submittal will be on the functional design and desired capabilities of the proposed tool. In addition, a real-time analysis using a prototype implementation will be performed and the results discussed. Any limitations and obstacles relating to the hardware, software, and data encountered during the research and development process will also be presented. Finally, the submittal concludes by posing critical questions yet to be addressed by research such as the statistical relevance of analysis results and their sensitivity to key analysis configuration parameters such as test feature-length, step increment, and spatial selection distance.

This submittal will potentially be of interest to the transportation safety, GIS, or software engineering professional.

Key words: corridors—crashes—network

Effect of PG Binder Grade and Source on Performance of Superpave Mixtures under Hamburg Wheel Tester

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ABSTRACT

Rutting and stripping of hot-mix asphalt pavements continue to be major problems even after the Superpave mixture design has been implemented. In this study, the effects of Performance Grade (PG) binder and source on the Superpave mixtures used on a project on US-169 in Kansas have been studied with the Hamburg wheel tester. The objective of the study was to assess whether the PG binder grade (and its source) used on this project had any impact on the premature rutting observed. The effect of compaction of the Superpave mix before wheel testing was also investigated. The test results were statistically analyzed using the Analysis of Variance (ANOVA) technique. The PG 70-28 binder grade appeared to perform better than all other PG binders used in the study. The Superpave mixtures showed significant difference in performance in terms of the numbers of repetitions to reach maximum 20 mm rut depth and to show stripping when compared by the binder source. Mixture compacted with lower air voids performed much better than the mixtures compacted with only two percent higher air voids. This implies that better compaction is a prerequisite for better performance of the Superpave pavements.

Key words: binder— hot-mix asphalt —pavement—wheel testing

INTRODUCTION

Two of the most common problems associated with hot-mix asphalt pavements are rutting and stripping. Rutting is the channelized depression on the wheel paths and results from accumulation of small amounts of unrecoverable strain due to repeated wheel loads. Stripping is the separation of the binder and aggregates in the presence of moisture**.** Both problems tend to occur during the early stages in the life of a pavement and trigger early undesirable maintenance actions. Thus asphalt mixtures should be designed so that they are rut resistant and are not susceptible to stripping. Many highway agencies have been using Loaded Wheel Testers (LWT's) for accelerated evaluation of the rutting and stripping potential of designed mixes (*1-6*). The absence of a mechanical test for the Superpave volumetric mixture has also made this type of LWT very attractive for evaluating potentially undesirable mixtures. The Hamburg wheel tester is one such device that can be used to predict the rutting and stripping potential of asphalt mixes.

In 1998, a newly placed Superpave pavement on US-169 in Neosho County, Kansas showed rutting immediately after construction. The pavement section consists of 200 mm of a 19 mm Superpave mixture with a PG 52-28 binder and 100 mm of a 19 mm Superpave mixture with a PG 58-28 binder in the base layer, and a 25 mm surface layer of a 9.5 mm Superpave mixture with a PG 64-28 binder. Due to unavailability of the binders from a single supplier, two different sources of binder were also used. It is presumed that higher PG binder grades would have prevented this premature rutting. In this study, performance of four different PG binder grades and two different sources in the 19 mm Superpave mixture, used on US-169, was evaluated in the Hamburg wheel tester.

HAMBURG WHEEL TESTER AND LINEAR KNEADING COMPACTOR

The Hamburg wheel-tracking device used in this study was manufactured by PMW, Inc. based out of Salina, Kansas, and is capable of testing a pair of samples simultaneously. Figure 1 shows the Hamburg wheel tester at Kansas State University. The sample tested is usually 260 mm wide, 320 mm long, and 40 mm deep. This slab sample has an approximate mass of 7.6 kg and is compacted to 7 ± 1 % air voids. The samples are submerged under water at 45°C. The wheel of the tester is made of steel and is 4.7 cm wide. The wheel applies a load of 705N and makes 52 passes per minute. Each sample is loaded for 20,000 passes or until 20 mm deformation occurs. The maximum velocity of the wheel reached is 340 mm/sec, which is at the center of the sample.

FIGURE 1. Hamburg Wheel Tester

Around 6 to 6 ½ hours are required for a test for maximum of 20,000 passes. Rut depth or deformation is measured at 11 different points along the length of each sample with a Linear Variable Differential Transformer (LVDT).

The various results that are obtained from the Hamburg Wheel Tester are creep slope, stripping slope and the stripping inflection point as illustrated in Figure 2 (*7*). The creep slope relates to rutting from plastic flow and is the inverse of the rate of deformation in the linear region of the deformation curve, after post compaction effects have been ended and before the onset of stripping. The stripping slope is the inverse of the rate of deformation in the linear region of the deformation curve, after stripping begins and until the end of the test. It is the number of passes required to create one mm impression from stripping, and is related to the severity of moisture damage. The stripping inflection point is the number of passes at the intersection of the creep slope and the stripping slope and is related to the resistance of the HMA to moisture damage. An acceptable mix is specified by the City of Hamburg to have less than 4 mm rut depth after 20,000 passes at a 50°C test temperature (*7*). However, this criterion was found to be very harsh in subsequent studies of the Colorado Department of Transportation (*7*,*8*).

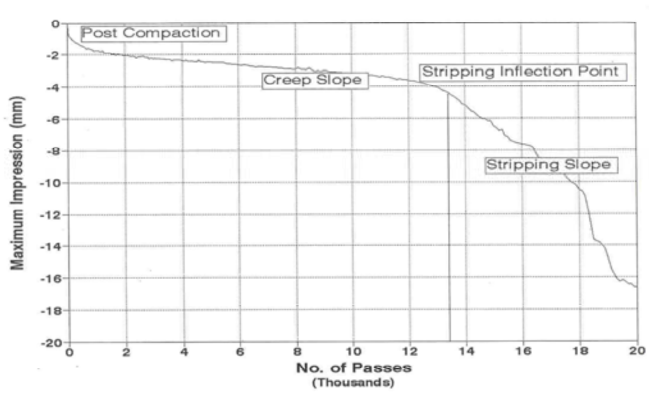


FIGURE 2. Interpretation of Results from the Hamburg Wheel Tester (*7*)

A linear kneading compactor was used to produce samples for the Hamburg wheel tester. Two slab samples of 320 × 260 mm and 40 mm or 80 mm height can be produced at a time. The samples are compacted to a known height; hence the target air void of the compacted sample can be achieved easily. The mold is filled with a pre-determined weight of the mixture from the knowledge of the theoretical maximum specific gravity of the mix. The sample can then be compacted within ± 1% of the targeted air voids, by a series of 12 mm wide steel plates, which are placed on the loose, mix in the mold. A linear compression wave is produced in the mix by the bottom edges of the plates as the roller pushes down on each plate. This kneading action allows the mixture to be compacted without fracturing the aggregates and is probably very similar to a steel wheel roller (*8*). The compaction time is less than 10 minutes.

MIXTURES TESTED

PG Binder and Sources

Four different binders were used in this study. The binders were obtained from two different refineries – Source 1 and Source 2. For source 1, PG 52-28 and PG 64-22 were used. PG 58-28 and PG 70-28 binders from source 2 were also used in the study.

Aggregates

The coarse and fine aggregates, used in the Superpave mixes in this study, were obtained from a number of sources. Two very similar 19 mm mix designs (corresponding to different binder sources) were developed using aggregates from three sources. Figure 3 shows the gradation of the aggregate blends used. In both cases, the gradation passed below the restricted zone. Blend 1 corresponds to mix number 3 and Blend 2 corresponds to mix number 2. Mix #2 consists of 31% Nelson crushed limestone, 29% Nelson crushed limestone screening, 20% Nelson manufactured sand, 10% Bingham chat (a slag aggregate from the zinc smelting industry in southeast Kansas), and 10% Ritchie sand. Mix #3 used 31% Nelson crushed limestone, 29% Nelson crushed limestone screening, 25% Nelson manufactured sand, 10% Bingham chat, and 5% Ritchie sand. The significant difference between these two mixtures is the proportion of river sand (from Ritchie Corp.). Five percent river sand in mix #2 has been replaced by 5% manufactured sand in mix #3. This increased the fine aggregate angularity of blend #2 by three per cent.

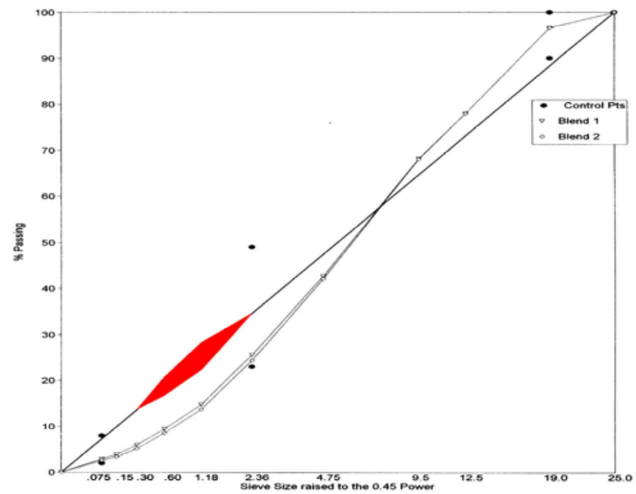


FIGURE 3. Aggregate Gradation of the Mixes Studied

Superpave Mixtures

A 19 mm nominal maximum size mix (#2) for the mainline base (bottom 200 mm) and shoulder was used for the binders from source 1, and a 19 mm nominal maximum size mix (#3) for the mainline base layer (top 100 mm) was used for the binders from source 2. The $N_{initial}$, N_{design}, and N_{final} for this project were 7, 86 and 134, respectively**.** Table 1 shows the required and achieved volumetric and aggregate properties. The asphalt contents of mix #2 and mix #3 were 5.5% and 5.4%, respectively.

TABLE 1. Properties of the Superpave Mixtures (N_{design} = 86)

Mixture/Aggregate Blend Property	Required/ Criteria	Mix #2	Mix #3
Asphalt Content (%)	-	5.50	5.40
Air Voids (%) at Ndes	4.0	4.0	4.00
VMA (%)	13 min.	14.0	14.0
VFA (%)	65-78	71	70.5
Dust-Binder Ratio	0.6 - 1.2	0.62	0.67
TSR	80% min.	98	95
%Gmm at Nini	89% max.	84	85
%Gmm at Nmax	98% max.	97	97.7
Sand Equivalent (%)	40 min.	93	95
Fine Aggregate Angularity (%)	40 min.	45	47
Coarse Aggregate Angularity (%)	50* or 75 min.	95.5	99
Flat & Elongated Particles (%)	10% max.	0.5	0.0

* for Mix #2

TESTING

The aggregates and the binders were mixed in the laboratory and tested following an experimental plan prepared by the Kansas Asphalt Pavement Association to extract specific information with a minimum amount of testing. The mixing and compaction temperatures were determined corresponding to the PG binder viscosities of 0.17 Pa.s and 0.28 Pa.s, respectively. The mixtures were aged for two hours at the compaction temperature before compaction in the kneading compactor. Maximum specific gravity tests were performed on each mix and the results were used for calculating the quantity of asphalt mix required to reach the desired air void content ($7 \pm 1\%$) in the compacted sample. Testing of the samples was usually done in the Hamburg wheel tester 24 hours after molding. All tests were done with the samples submerged under water at 45^{o} C. The test temperature was selected following manufacturer's recommendation. Testing in this study continued until the rut depth at any of the 11 points on the specimen reached a maximum value of 20 mm or the maximum number of wheel passes (20,000) was reached. Two replicate samples were tested side by side. All samples, except the mixture with PG 70-28 binder, showed signs of severe stripping and shoving during tests. Fines were observed to come out of the mixes during testing.

The Hamburg wheel tester automatically records deformation (in mm) at 11 different points along the specimen for each wheel pass. After the test, the wheel pass number versus deformation curve, similar to the one shown in Figure 2, was plotted using the Excel spreadsheet. Using the curve fitting technique available in Excel, the creep slope, stripping slope and stripping inflection point were determined. The results are summarized in Table 2. It appears that higher binder grade enhances mixture performance.

TABLE 2. Summary of Hamburg Test Results

Mix Type	Binder Source	Number of Passes		Average Number of Passes	Average Creep Slope	Average Stripping Inflection Point	Average Stripping Slope
		Specimen #1 (Left)	Specimen #2 (Right)				
PG 52-28	1	4260	4840	4550	465	3131	289
PG 64-22	1	5741	7881	6811	571	5081	266
PG 58-28	2	4701	2700	3700	217	3220	138
PG 70-28	2	9180	10781	9980	952	6971	281

STATISTICAL ANALYSIS

Experimental Design and Variables Studied

Comparison among different factors that might affect the performance of the Superpave mixes was made using the Analysis of Variance (ANOVA) technique and the SAS software (9). The statistical experiment analyzed is a randomized balanced experiment with blocking on the wheel/specimen. ANOVA was performed using the Least Square Means (LSMeans) approach (10) to test the effect of different factors on the dependent (response) variable.

In this study four response variables were studied: (a) Number of repetitions to reach a 20 mm maximum rut depth, (b) Creep slope, (c) Stripping Slope, and (d) Stripping inflection point. The effect of PG grade is studied in the ANOVA with the model in Equation (1):

$$(\text{Response Variable})_{ij} = \text{Binder}_i + \text{Wheel}_j + \varepsilon_{ij} \qquad (1)$$

Where, $(\text{Response Variable})_{ij}$ = The various response variables studied;
Binder_i = ith binder effect;
Wheel_j = jth wheel effect (of the Hamburg wheel tester); and
ε_{ij} = Error term.

RESULTS AND DISCUSSIONS

Effect of PG Binder Grade

The results of the ANOVA are shown in Table 3 incorporating all variables shown in Equation 1. All conclusions were drawn at a 95% confidence interval. For the number of wheel passes to reach a 20 mm maximum rut depth any where in the sample, significant differences were found between the binder grades PG 52-28, PG 58-28 and PG 70-28. PG 64-22 and PG 70-28 were found to be statistically similar in terms of the number of wheel passes to reach maximum 20 mm rut depth. Significant difference was also found between the binders PG 58-28 and PG 70-28 when the creep slope was analyzed. This probably was mainly due to the poorly performing Sample #2 for the PG 58-28 mixture. However, the statistical conclusions are still valid since during analysis “blocking” was done on the “wheel/sample” to take into account this variability. The results indicate that the rate of rutting of the mixture #3 with PG 58-28 binder is higher than PG 70-28 binder. Since PG 58-28 in mixture #3 was used on US-169, a PG 70-28 binder might have prevented early rutting on this project. For the stripping slope, results are similar to that for the creep slope. Significant differences were also obtained between PG 52-28 and PG 58-28. The stripping inflection point was found different for PG 70-28 when compared to the other three binder grades. This indicates that as far as moisture susceptibility is concerned, PG 70-28 would perform the best.

TABLE 3. Effect of the PG Binder Grade on the No. of Wheel Passes, Creep Slope, Stripping Slope and Stripping Inflection Point

Number of Wheel Passes to reach 20mm rut depth					Creep Slope				
PG Grade	52-28	58-28	64-22	70-28	PG Grade	52-28	58-28	64-22	70-28
52-28	-	Similar	Similar	Different	52-28	-	Similar	Similar	Similar
58-28	Similar	-	Similar	Different	58-28	Similar	-	Similar	Different
64-22	Similar	Similar	-	Similar	64-22	Similar	Similar	-	Similar
70-28	Different	Different	Similar	-	70-28	Similar	Different	Similar	-
Stripping Slope					**Stripping Inflection Point**				
PG Grade	52-28	58-28	64-22	70-28	PG Grade	52-28	58-28	64-22	70-28
52-28	-	Different	Similar	Similar	52-28	-	Similar	Similar	Different
58-28	Different	-	Different	Different	58-28	Similar	-	Similar	Different
64-22	Similar	Different	-	Similar	64-22	Similar	Similar	-	Similar
70-28	Similar	Different	Similar	-	70-28	Different	Different	Similar	-

Effect of Binder Source

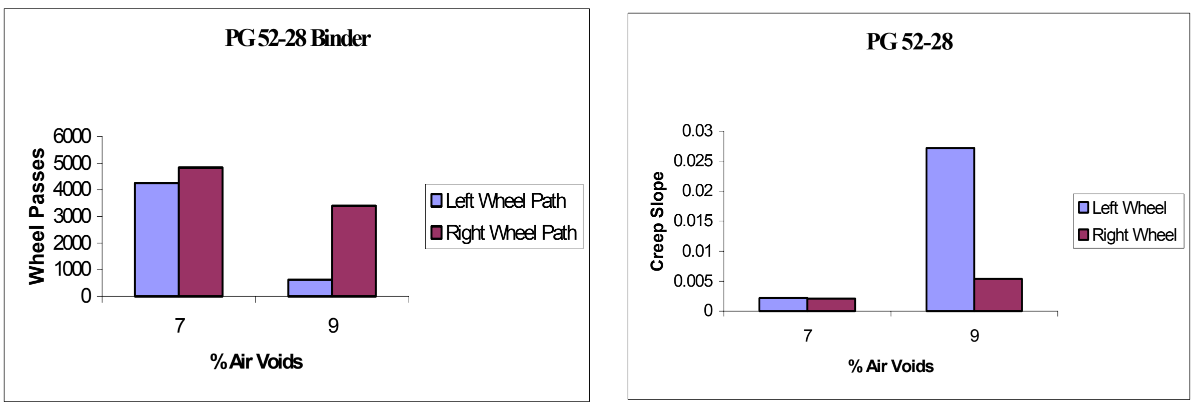
ANOVA analysis was done to compare the two different sources of binder using "CONTRAST" for the variable "source" over the PG grades used. Note that although the experiment factorial is not a "complete" one, such comparison is still valid since there is very little difference in aggregate gradations between mix #2 and mix #3. The results of this analysis are presented in Table 4. All comparison was done at a 95% confidence interval. Significant differences were found between the sources of binder for the two mix designs with respect to the number of repetitions to reach 20 mm maximum rut depth and the stripping inflection point. For other variables i.e., creep slope and stripping slope, no significant differences were found between the binder sources. This supports the conclusion drawn earlier – use of PG 70-28 binder from any source might have prevented the early rutting on US-169.

TABLE 4. Effect of the Source of Binder on the No. of Wheel Passes, Creep Slope, Stripping Slope and Stripping Inflection Point

No. of Wheel Passes			Creep Slope		
Source	Source 1	Source 2	Source	Source 1	Source 2
Source 1	-	Different	Source 1	-	Similar
Source 2	Different	-	Source 2	Similar	-
Stripping Slope			**Stripping Inflection Point**		
Source	Source 1	Source 2	Source	Source 1	Source 2
Source 1		Similar	Source 1		Different
Source 2	Similar		Source 2	Different	

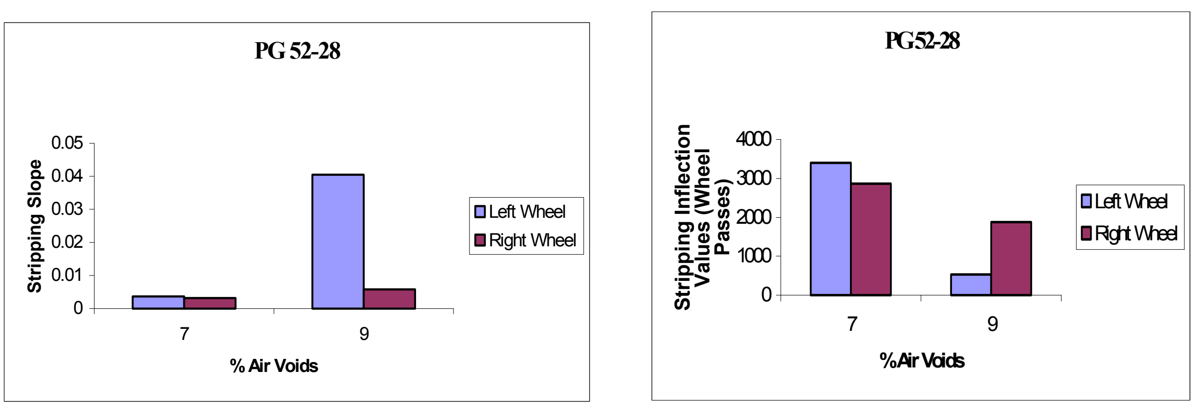
Effect of Compaction

There is some opinion that early rutting on US-169 was due to compaction variability although the contractor achieved bonus payments for the in-situ density. The density variability was also studied. An earlier study in Colorado (*8*) found significant differences in mixture performance in the Hamburg wheel testing of the asphalt mixes compacted in the laboratory and those from the field. Effect of compaction in this research was done on the mix #2 with PG 52-28 binder from source 1 at two different air voids, 7% and 9%. Figures 4 and 5 show the comparison of the number of wheel passes to reach a 20 mm rut depth, creep slope, stripping slope and stripping inflection point for the two different compaction levels studied. It is obvious that the samples with 9% air voids failed earlier compared to those with 7% air voids with respect to the number of repetitions to reach maximum 20 mm rut depth. Large variations were also observed between the samples under the left and right wheels of the Hamburg tester. Higher creep and higher stripping slopes for the 9% air void samples indicate that inadequate compaction will lead to both accelerated rutting and stripping failure. The higher stripping inflection point values for the 7% air void samples also suggest that these samples are more resistant to moisture damage compared to the 9% air void samples.



(a) Number of Wheel Passes to 20 mm rut depth (b) Creep Slope

FIGURE 4. Results from PG 52-28 with Different Air Voids



(a) Stripping Slope (b) Stripping Inflection Values

FIGURE 5. Results from PG 52-28 with Different Air Voids

CONCLUSIONS

Based on the results of this study the following conclusions are made:

1. The PG 70-28 binder grade appeared to perform the best when compared with all other PG binder grades used in this study.

2. The Superpave mixtures showed significant differences in performance in terms of the number of repetitions to reach maximum 20 mm rut depth and the stripping inflection point when compared by binder source.

3. Superpave mixture compacted to 7% air voids performed much better than the mixture compacted to 9% air voids.

ACKNOWLEDGMENTS

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Correlation between the Laboratory and Field Permeability Values for the Superpave Pavements

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ABSTRACT

Permeability affects the performance of Superpave pavements. Percolation of water, through the interconnected voids of an asphalt pavement, causes stripping of the asphalt-bound layer as well as deterioration of the foundation layers of the roads. In this study, laboratory and field permeability tests, based on the principles of falling head, were conducted on different Superpave mixes with 19 mm and 12.5 mm nominal maximum aggregate sizes and coarse and fine gradations, to study the correlation between the laboratory and the field permeability values. The objective was to assess whether the field permeability values could be estimated during the mixture design process so that mix design can be adjusted depending upon the degree of permeability desired.

The results show that there was a significant difference between the laboratory- measured and the field permeability values. The field permeability values were very high compared to the laboratory-permeability values. The reason behind this discrepancy was further investigated and explained in this paper. Nevertheless, the field permeability values were found valuable in assessing compaction quality of the Superpave pavements.

Key words: pavement mixture—permeability—superpave pavements

INTRODUCTION

Proper compaction of the hot-mix asphalt (HMA) mixtures is vital for a stable and durable pavement. Low in-place air voids cause problems such as, rutting and shoving, while high in-place air voids reduce the pavement durability through moisture damage and excessive oxidation of the asphalt binder. For dense graded mixtures, numerous studies have shown that initial in-place air void contents should not be below three percent or above approximately eight percent *(1)*. As the in-place air voids increases, the permeability also increases. Past studies have also shown that mixtures with different Nominal Maximum Aggregate Size (NMAS) have different permeability characteristics *(2)*. Coarse-graded Superpave mixtures have a different internal air void structure than the dense-graded mixes used prior to Superpave *(3)*. As the NMAS increases, the size of individual air voids increases, and that leads to an increased potential for interconnected air voids. These interconnected air voids cause permeability in the Superpave pavements. Interconnected voids are the paths through which water can flow and hence mixtures with higher NMAS would be expected to be more permeable at a given air void content compared to the mixtures with a lower NMAS. The gradation characteristics of the aggregate structure also affect the permeability of the Superpave mixtures. Thickness of the Superpave mixture layer/lift is another factor that affects the permeability *(2)*. In normally constructed asphalt pavements all void spaces are not necessarily interconnected. Voids that are not interconnected do not allow water to flow thorough them. As the thickness of the pavement increases, the chance for voids being interconnected with a sufficient length to allow water to flow decreases. Because of this thinner pavements may have more potential for permeability.

During the mix design process it is not possible to know the actual permeability of a Superpave mix in the field without actually placing, compacting, and then measuring the field permeability value. In this study, in-place permeability testing was conducted on different Superpave pavements in Kansas to study the correlation between the laboratory and the field permeability values to see whether the field permeability values could be predicted during the mixture design process. By predicting field permeability values, mix design can be adjusted depending upon the degree of permeability desired. Correlations between the field and the lab permeability values and percent air voids were also investigated.

STUDY APPROACH

This study was conducted in two parts: (a) field-testing of Superpave mixtures, and (b) laboratory testing of gyratory compacted mixes obtained from the field. A commercially available field permeability-measuring device, available from Gilson, Inc., was used in this study. Figure 1 shows the schematic of this field permeability-measuring device. The device is based on the falling head principle of permeability and the Darcy's equation:

$$K = \frac{al}{At}\ln\left(\frac{h_1}{h_2}\right) \tag{1}$$

where
K = Coefficient of permeability in cm/sec
a = inside cross sectional area of inlet standpipe in cm^2
l = thickness of the HMA specimen, cm
A = cross-sectional area of the HMA specimen, cm^2
t = time taken for water to flow from h_1 to h_2, seconds
h_1 = initial head of water, cm
h_2 = final head of water, cm

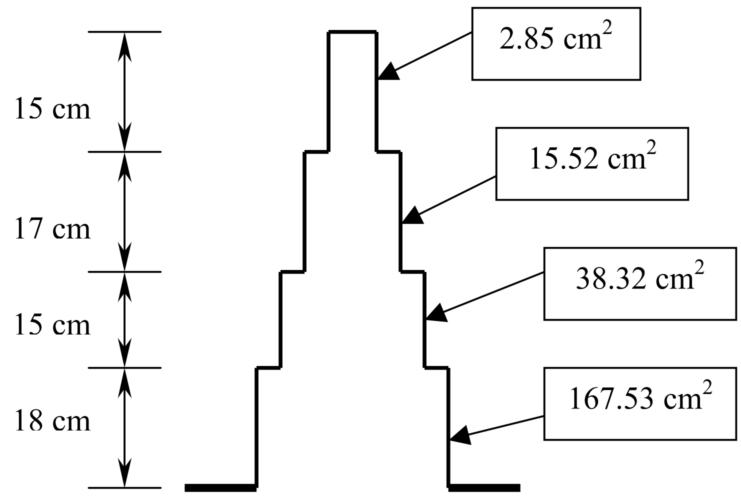


FIGURE 1. Schematic of the Field Permeameter

The permeameter consists of four conjoined segments or “tiers” of clear acrylic plastic, with varying cross section, so that a wide range of permeability can be tested. The area where the permeability test is conducted is cleaned thoroughly to remove the surface dust. Then the permeameter is fitted with a rubber gasket sealant, which ensures a watertight seal between the base of the permeameter and the pavement surface. About 2,500 grams weight is added to each corner of the permeameter to compensate for the head of pressure exerted by the water column. With out this counter weight, the water pressure can break the seal between the permeameter and the surface of the pavement. Water is then filled into the permeameter with a filling tube at a steady rate up to the top and then a tier is selected for monitoring the permeability, which is neither too fast nor too slow for accurate measurement of permeability. The large diameter of the second tier was used since the flow of water would be slow enough for efficient recording of data for most Superpave pavements tested in this study. The time taken for the water level to fall by 100 mm was noted.

For each project, tests were conducted at three locations on the newly compacted Superpave pavement, spaced at about 300 mm apart. At each test location, three replicate permeability tests were conducted before opening the section to traffic. All permeability measurements were made at a distance of about 600 mm from the pavement edge. The permeability device uses a rubber sealant to help seal the permeameter to the pavement surface. After the first test at a given test location, the device was lifted off the pavement and re-sealed immediately to conduct the second replicate test. Each replicate test was conducted at a spacing of approximately 250 mm. Field permeability testing was done in a direction longitudinal to the pavement, since the pavement density tends to be more uniform longitudinally than transversely. Also, plant produced mix was sampled from behind the paver for each project to carry out permeability testing on laboratory-compacted Superpave specimens.

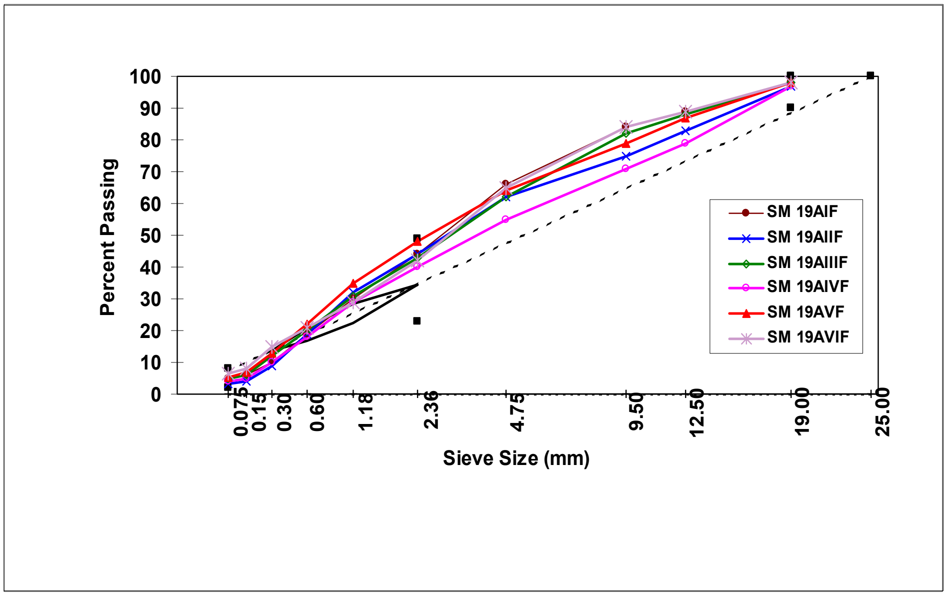
Table 1 summarizes the characteristics of the Superpave mixes obtained from different Superpave projects. All mixtures have either 19 mm or 12 mm nominal maximum aggregate size (NMAS). Figure 2 shows the aggregate gradation charts for the 19 mm and 12.5 mm Superpave mixes used.

All mixtures had gradations passing below the maximum density line in the finer sand sizes. Thus the mixtures are designated as “fine” in Kansas. The mixtures are from different lifts of the pavements. From the aggregate gradations, shown in Figure 2, it can be seen that all gradations pass through the restricted zone. The Kansas Department of Transportation (KDOT) has recently discontinued using restricted zone in the Superpave mixture gradation. The mixtures, however, met all requirements set by KDOT for the 19 mm and 12.5 mm NMAS Superpave mixtures.

TABLE 1. Properties of the Superpave Mixes used in the Field Study

Mixture/ Aggregate Blend Property	Description	Design ESALs (millions)	N_{design}	PG Binder Grade	Asphalt Content (%)	Air Voids (%) at Ndes	VMA (%)	VFA (%)	Dust Binder Ratio	%Gmm at Nini	%Gmm at Nmax
SM 19A(I)	Ritchie K-77	1.4	75	PG 58-28	5.2	4.26	13.96	69.56	1.2	89.1	96.6
SR 19A(II)	Venture I-70 Russell Shld	15	100	PG 64-23	3.6	4.7	14.79	68.21	0.73	90.66	96.08
SR 19A(III)	Venture I-70 Russell ML 1st Lift	15	100	PG 58-28	3.8	4.37	14.81	70.48	1.05	89.66	97.24
SR 19A(IV)	Venture I-70 Russell ML 2nd Lift	15	100	PG 70-28	4.2	4.15	14.12	70.53	0.96	89.71	97.26
SR 19A(V)	Venture I-70 Ellis ML 1st Lift	17.2	100	PG 64-22	3.9	4.4	14.68	76.43	1.2	90.65	97.55
SR 19A(VI)	Venture I-70 Ellis ML 2nd Lift	17.2	100	PG 70-28	4.3	3.6	13.5	73.36	1.4	88.8	97.6
SM 12.5A(I)	Hamm US-73	1.2	75	PG 64-22	6.5	4.74	14.58	67.49	1.29	87.5	96.4
SM 12.5A(II)	Ritchie K-4	0.7	75	PG 64-22	5.4	3.32	13.89	75.15	1.1	89.7	97.7
SM 12.5A(III)	Heckert US-56	1.2	75	PG 64-22	5.25	4.14	13.02	70.78	0.73	88.9	96.7

(a) SM 19 mm Mixes



(b) SM 12.5 mm Mixes

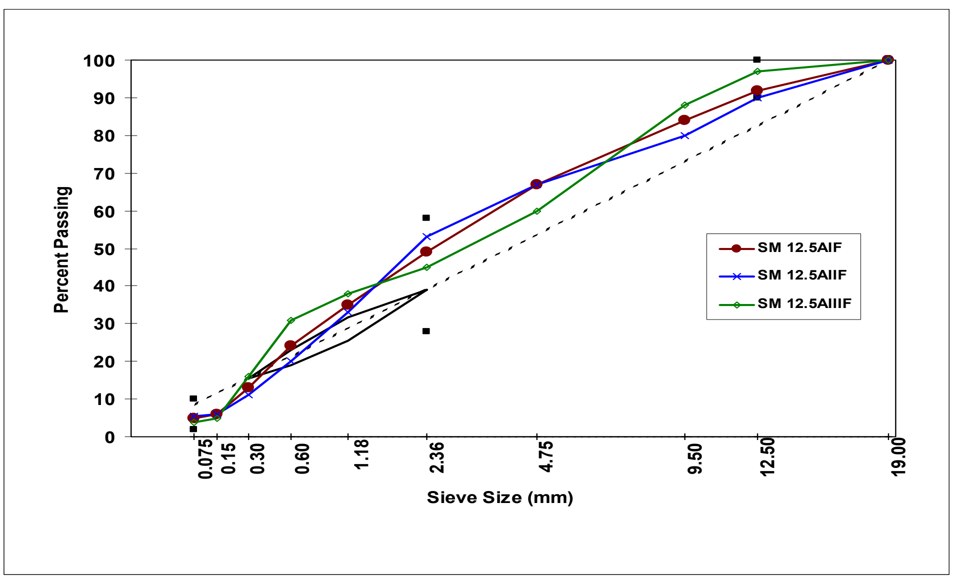


FIGURE 2. Combined Gradation Charts for NMAS 19 mm and 12.5 mm Superpave Mixes

POTENTIAL PROBLEMS IN MEASURING PERMEABILITY

A majority of the recent research work in permeability testing has been conducted on the core specimens that have been cut from the pavements or on the specimens compacted in the Superpave gyratory compactor. This is an important since Darcy's law is applicable for one-dimensional flow as would be encountered in a laboratory test. Measuring the in-situ permeability of an in-place pavement is theoretically more difficult, because water can flow in two dimensions.

Other potential problems include the degree of saturation, boundary conditions of the flow and the type of flow. The flow of water in the laboratory permeability test is in reality is one-dimensional.

Another potential boundary condition problem is the flow of water across (through) the pavement layers. Without some type of destructive test, such as, coring there is no way of knowing whether water flows across the layers *(3)*. Darcy's law was derived based on the experimentation on clean sands and the flow of water through the sand was determined to be laminar. Within a pavement section, it cannot be determined whether the flow of water is laminar or turbulent. Since we cannot use Darcy's law to calculate permeability if the flow is turbulent, hence the flow of water through the pavement *is assumed to be laminar*. While conducting the field permeability tests, it was observed that the drop in water level during the first test usually took lesser time compared to the subsequent replicate tests. One possible explanation is that during the first test, the water fills up the voids, including some that were not interconnected, and during the second and third tests, the water cannot go through these non interconnected voids, and only flows through the interconnected voids *(4)*.

RESULTS AND DISCUSSION

All laboratory test samples were compacted in a Superpave gyratory compactor to a target air void content of 7%. Table 2 summarizes the field and laboratory permeability values. Figure 3 illustrates the comparison of these measured permeability values. A large variation between the permeability values measured in the field and those in the laboratory was observed. These results show that there was a significant difference between the field and laboratory permeability values. The field permeability values are always much higher than the laboratory permeability values. This higher field permeability can be explained in terms of water flow in the field. Unlike laboratory tests, the flow in the in-situ pavement is not confined to one-dimensional flow. Water entering the pavement can flow in any direction (vertical and/or horizontal). Therefore, it would be expected that the field permeability values should be higher than the laboratory permeability values since both are estimated based on the falling head permeability test principles. The difference in the permeability value obtained would be most likely dependant upon the NMAS, aggregate gradation, and the interconnectivity of air voids. Past experience has shown that a large amount of flow took place in the coarser Superpave mixes with thick lifts in the horizontal direction, whereas finer mixes with thinner lifts tend to have more of a vertical flow *(4)*. During most field tests in this study, water was observed to come up through the mat a few centimeters away from the base of the permeameter. This could be due to the horizontal flow of water in the underlying layers of the pavement.

Figure 4 shows the relationship between the laboratory permeability values and the percent air voids for different mixtures tested in this study. The air voids were measured on the laboratory-compacted samples. No meaningful correlation among the field permeability, the laboratory permeability and the percentage air voids of the laboratory-compacted specimens was observed as shown in Figures 3 and 4.

Mat Tearing and Breakdown Rolling

The weight of the breakdown roller is one of the factors that influence the permeability of Superpave pavements during construction. Mat tearing was observed on one of the projects in this study. The mixture was a recycled Superpave mixture with 19 mm NMAS on I-70 in Ellis county, Kansas. The first lift of the pavement exhibited mat tearing at several locations. The permeability values at these locations were also very high. The average field permeability value on the sections which did not show mat tearing was found to be 6.18×10^{-3} cm/sec. The field permeability values on the torn mat was found to be 25.36×10^{-3} cm/sec, nearly four times the average field permeability value for the locations with no mat tearing. The thin cracks appeared on the surface of the HMA pavement, due to the heavy roller compacting a thin lift (65 mm) of HMA.

TABLE 2. Summary of Field and Laboratory Permeability Results

	Field Permeability Results				Lab Permeability Results			
	Project	Location	Field Permeability (cm/sec)	Average Permeability (cm/sec)	Sample	% Air Voids	Lab Permeability (cm/sec)	Average Permeability (cm/sec)
1	Venture Russel County 1st Lift	1	1.8612E-03		1	9.02	6.8814E-05	
	SR 19A	2	1.2279E-03	1.6454E-03	2	7.90	4.6064E-05	5.7950E-05
		3	1.8471E-03		3	6.68	5.8970E-05	
2	Venture Russel County 2nd Lift	1	3.6090E-03		1	6.74	2.8985E-05	
	SR 19A	2	5.5294E-03	5.0507E-03	2	6.63	5.6808E-05	6.24124E-05
		3	6.0136E-03		3	6.65	1.0144E-04	
3	Venture Russel County Shoulder	1	3.4803E-03		1	6.51	4.2530E-05	
	SR 19A	2	3.0712E-03	3.1900E-03	2	6.54	3.0287E-05	3.55285E-05
		3	3.0185E-03		3	6.59	3.3768E-05	
4	Venture Ellis County 1st Lift	1	5.8534E-03		1	7.04	2.5835E-06	
	SR 19A	2	7.5438E-03	6.1797E-03	2	6.70	2.1712E-04	1.0846E-04
		3	5.1419E-03		3	5.56	1.0568E-04	
5	Venture Ellis County 2nd Lift	1	1.4095E-02		1	6.64	5.2743E-05	
	SR 19A	2	1.3210E-02	1.3008E-02	2	6.70	1.7263E-05	1.0846E-04
		3	1.1719E-02		3	6.68	2.2201E-05	
6	KDOT Marion County	1	8.6681E-03		1	6.97	6.8525E-07	
	SM 19A	2	8.6832E-03	8.3794E-03	2	6.85	2.1315E-06	3.37308E-06
		3	7.7867E-03		3	7.40	7.3025E-06	
7	Ritchie Paving K-4	1	3.4972E-03		1	6.91	1.06047E-05	
	SM 12.5 A	2	3.7896E-03	3.7523E-03	2	6.91	3.53998E-05	2.5104E-05
		3	3.9701E-03		3	6.66	2.93075E-05	
8	Hamm Const. US 73	1	5.1538E-03		1	6.88	3.64946E-06	
	SM 12.5 A	2	7.4717E-03	7.0719E-03	2	6.83	3.06385E-06	2.98136E-06
		3	8.5902E-03		3	6.83	2.23077E-06	
9	Heckert US 56	1	3.8079E-03		1	6.54	1.04708E-05	
	SM 12.5 A	2	3.6109E-03	3.6451E-03	2	5.03	5.26915E-07	5.00212E-06
		3	3.5164E-03		3	6.1	4.00864E-06	

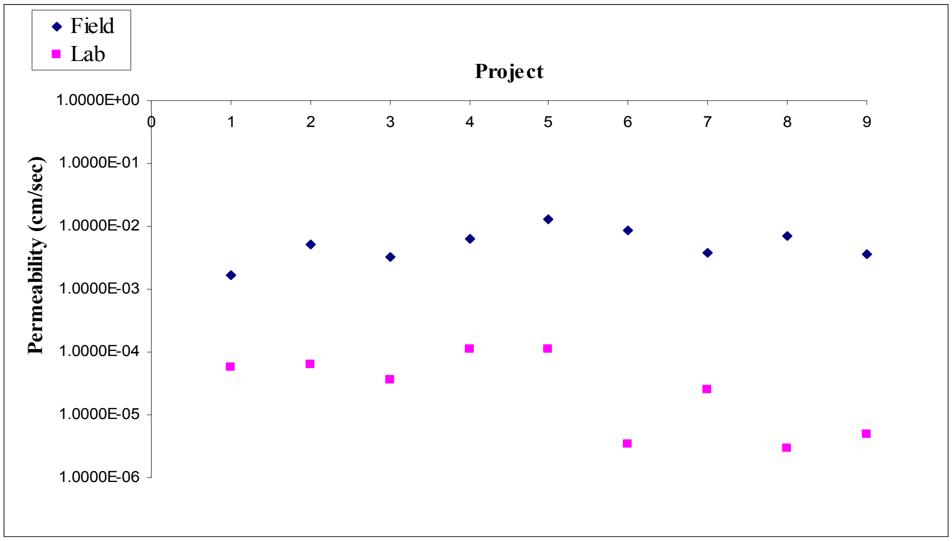


FIGURE 3. Comparison of Field and Laboratory Permeability Values

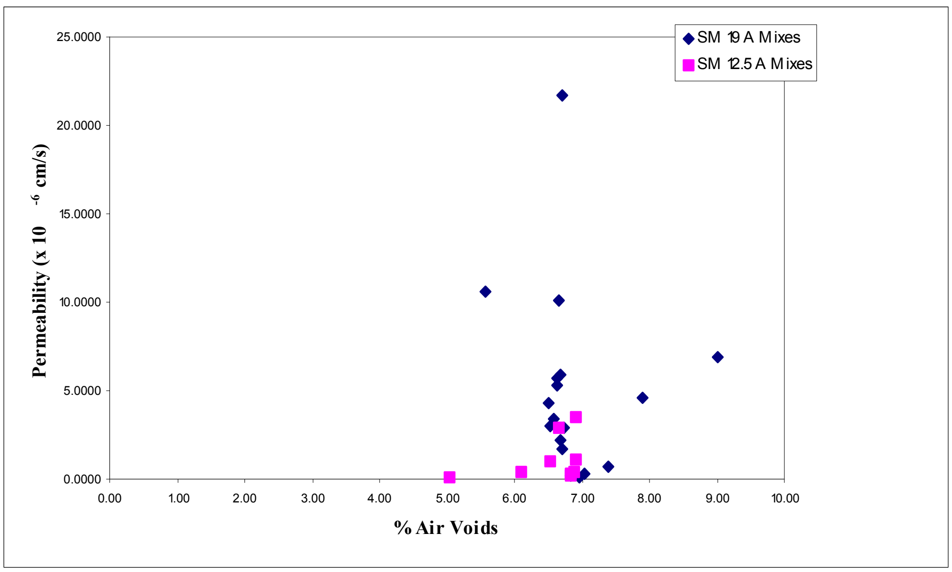


FIGURE 4. Relationship between Permeability and Air voids for the NMAS 19 mm and 12.5 mm Mixes

CONCLUSIONS

1. There is a significant difference between the permeability values obtained from the laboratory tests and the field tests using the same principles of measurement. The field permeability values are consistently higher than the laboratory permeability values.

2. There is no statistical correlation among the field permeability, the laboratory permeability and the percentage air voids of the laboratory-compacted specimens.

3. The mat tearing by heavy breakdown roller significantly increases the field permeability values.

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Bridge Frost: Observations and Forecasts by Numerical Methods

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ABSTRACT

Evaluation of forecast methods for predicting frost on bridges and roadways is difficult due to lack of observations of frost occurrences. Even observations made by Department of Transportation personnel responsible for bridge winter maintenance are compromised because bridges pre-treated for frost suppression (or having residual chemical from previous treatments) respond differently from untreated bridges when meteorological conditions favor frost. We have made early-morning bridge observations for frost during the past two winter seasons on untreated bridges to establish a database of bridge frost occurrences and non-occurrences. When frost is detected, observations are continued until the frost disappears, thereby providing additional information on duration and timing of onset and demise. A numerical simulation model for vertical heat transfer in a bridge has been developed that uses input from a weather forecast model to predict bridge-surface temperatures and to predict onset time, depth, and duration of frost on the bridge. Comparisons of forecasts and observations will be used to assess skill in forecasting bridge frost.

Key words: bridge—bridgeT—forecast—frost—temperature

INTRODUCTION

Frost on roadways and bridges can present hazardous conditions to motorists, particularly when it occurs in patches or on bridges when adjacent roadways are clear of frost. To minimize materials cost, vehicle corrosion, and negative environmental impacts, frost-suppression chemicals should be applied only when, where, and in appropriate amounts needed to maintain roadways in a safe condition for motorists. Accurate forecasts of frost onset times, frost intensity, and frost disappearance (e.g., melting or sublimation) are needed to help roadway maintenance personnel decide when, where, and how much frost-suppression chemical to use. Accurate bridge frost forecasts rely on accurate forecasts of bridge surface temperature, two-meter air temperature, humidity, and wind speed. These factors are routinely calculated by weather forecast models, such as the PSU/NCAR mesoscale model (MM5). However the "surface" temperature in the model represent the dominant land or water surface within the grid area and does not adequately represent bridge surface temperature. A finite-difference algorithm (BridgeT) has been developed that simulates heat movement vertically in a bridge in response to evolving conditions (including surface longwave and shortwave radiation) produced by MM5. A frost accumulation algorithm uses the bridge surface temperature from BridgeT and concurrent meteorological variables from MM5 to produce forecasts of incremental volume per unit area (i.e., depth) of frost deposited, melted, or sublimed. Reliable observations of bridge surface temperature and frost occurrences on uncontaminated bridge surfaces are needed verify the model.

OBJECTIVE

The focus of the bridge frost observation project is to collect observations of surface temperature and frost formation on untreated bridges. Such observations are needed to evaluate the capability of frost accumulation algorithms used in BridgeT and other frost forecasting systems. Observations of frost occurrence made by Department of Transportation personnel are often compromised because bridges pre-treated for frost suppression (or having residual chemical from previous treatments) respond differently from untreated bridges when meteorological conditions favor frost. Bridge surface temperature observations provide an additional means (i.e., in addition to frost verification) of evaluating the frost forecast procedure. They also provide quantitative information on the spatial variability of environmental conditions essential for frost formation. A nearby RWIS site offers additional opportunity for exploring spatial variability. We give an overview of the use of data collected for validation of the frost accumulation algorithm and BridgeT.

OBSERVATION PROCEDURE

Bridge Selection

Frost occurrence was observed for the 2001–2002 winter on the State Avenue Bridge over Highway 30 near Ames, Iowa. For the 2002–2003 frost season, County Line Road Bridge and South Dakota Avenue Bridge were added to the observations protocol, thereby allowing investigation of spatial variations in frost development. Multiple bridges also allowed for the frost observations even if one of the bridges had been treated with de-icing chemicals.

State Avenue Bridge and County Line Road Bridge were selected for close-up observation for this study because they are not routinely treated with frost suppressing materials. They have no on or off ramps for possible turn-around points for IaDOT trucks, thus minimizing the potential for inadvertent chemical spill from the frost-treatment vehicles. These bridges are typically not as heavily traveled as some other nearby bridges during early morning hours, so bridge observations on foot can be made more safely. The South

Dakota Avenue Bridge was selected as a drive-over observation site because it is easily checked along the way while traveling between State Avenue and County Line Road, and does not get frequent frost suppression treatment because it is a new bridge. Traffic on the South Dakota Avenue Bridge is too high to allow observations on foot, but frost observations can be made from the vehicle. State Avenue, South Dakota Avenue, and County Line Road bridges are all north-south concrete bridges that allow passage over US Highway 30 at one-mile intervals from east to west, respectively. Terrain in the vicinity of all bridges is quite flat, with embankments created for the bridge itself being comparable to or greater than natural terrain relief in the immediate area. Flat terrain between bridges minimizes topographically induced influences (drainage flow, shading, wind-tunnel effects, etc.), thereby allowing a cleaner study of natural variability of frost formation.

Frost Observations

Observations began on November 28, 2001 and continued through March 21, 2002 and began again on 15 November 2002 and continued through March 21, 2003. Every day was considered a candidate for frost, unless there was a very low probability of frost (e.g., nighttime temperatures were greater than 40 ºF). Some days were missed due to holidays and snowstorms. The observer visited the bridges beginning at 5:00 AM CST and observed frost conditions both from the car and close-up on foot. While on the bridge on foot, the observers carefully examined the surface for frost and measured the temperature of the bridge surface with an infrared thermometer. The time, date, observations of bridge conditions, general weather conditions, frost characteristics, and surface temperature for each bridge were recorded. If frost was detected, the observer would return periodically until the frost dissipated for follow-up observations and measurements.

Temperature Observations

Measurements of bridge surface temperature were made with hand-held infrared thermometers. In the 2001–2002 frost season and the first two months of the 2002–2003 season, a Raytek thermometer was used that required calibration before each use and required that the surface temperature be taken within a few inches of the bridge surface. Beginning in mid-January 2003 through the end of the season we used an Exergen thermometer that did not require continuous calibration but did require that the thermometer aperture be in contact with the surface. Temperature measurement precision (as judged from lack of sensor drift and uniformity across the bridge surface) and accuracy (as judged by comparison with nearby RWIS observations) were higher for the Exergen instrument.

RESULTS

Frost Occurrence

Winter of 2001 – 2002

Observations were made on the State Avenue Bridge 47 days, and frost was observed 14 times, as presented in Table 1.

TABLE 1. Days when Bridge Frost Was Observed on State Avenue

Days When Bridge Frost Was Observed				
12/2/2001	12/30/2001	1/12/2002	2/3/2002	2/13/2002
12/6/2001	1/3/2002	1/24/2002	2/5/2002	2/17/2002
12/29/2001	1/7/2002	2/2/2002	2/7/2002	

Winter of 2002-2003

Observations were made on 93 days, and frost was observed 10 times, as presented in Table 2. Frequently the observer noted moisture on the bridge that could not be positively identified as frost. Dark or discolored patches or streaks covering various portions of the bridge often were observed , and occasionally the surface had patches of ice. These events were not considered frost events. Frost observation similarity among the bridges is given by the “Agreement” column of Table 2. Agreement is found on slightly more than half of the mornings with frost. It also may be noteworthy that the South Dakota Avenue bridge (located midway between the other two) never disagreed with both other bridges. Therefore lack of agreement could be associated with an east-west frost gradient across all three bridges.

TABLE 2. Morning Frost Occurrences on Observed Bridges

Date	Frost Occurrence			Agreement
	State Avenue	South Dakota	County Line	
11/12/02	yes	--	--	--
11/23/02	no	yes	yes	no
12/06/02	yes	yes	yes	yes
12/11/02	no	no	yes	no
12/19/02	yes	yes	no	no
12/26/02	yes	yes	yes	yes
12/27/02	yes	yes	yes	yes
01/07/03	yes	yes	yes	yes
02/18/03	yes	yes	yes	yes
02/26/03	no	no	yes	no

Temperature

For accuracy and consistency, all temperature comparisons herein reported were derived from observations taken with the newer Exergen thermometer.

County Line Road Bridge tended to be cooler than State Avenue by about 0.65 ºF at the 5:00 AM observation time, which is consistent with the tendency for County Line Road Bridge to have higher frost frequency. The largest temperature difference between the two bridges was 3.4 ºF. The disparity in temperatures and frost occurrence among the bridges suggests that spatial effects or bridge composition variations can be significant factors influencing the potential of frost formation between bridges.

Automated RWIS bridge surface temperature observations are taken from the Interstate 35 overpass over 13th Street on the east side of Ames, Iowa. The distances from the RWIS site to the State Avenue and County Line Bridge are approximately five and seven miles, respectively. The differences and standard deviations of temperature between the RWIS site and the two bridges over Highway 30 at 5:00 AM shown in Table 3 indicate that RWIS surface temperatures are usually a few degrees lower than those taken at the observed bridges. Sensors 1 and 3 are located on the northbound overpass on the passing and driving lanes. Sensor 2 is located on the northbound passing lane. Instrument error and small-scale differences are possible causes of variations among RWIS temperature sensors.

TABLE 3. Observed Temperature Differences (Bridge Minus RWIS) and Standard Deviations at Initial Observation Time

County Line - RWIS				State Avenue - RWIS		
Sensor	Temp. Diff.	stDev		Sensor	Temp. Diff.	stDev
1	1.89	4.47		1	2.30	4.37
2	4.76	4.26		2	5.00	3.98
3	3.22	4.17		3	3.51	3.89

APPLICATIONS

Frost Calculations

The frost accumulation algorithm (*2*) uses weather information to estimate frost deposition by calculating an incremental increase or decrease in frost "depth" and then adding the new accumulation to the previous depth. Frost depth calculated by this algorithm has not been verified by depth observations but allows for intercomparison of relative frost deposition amounts among events and provides an estimate of the onset and termination of frost. It can also help in identifying a threshold of frost significance – the amount of frost necessary to be observable or hazardous. We use the frost accumulation algorithm with meteorological input from RWIS and MM5 for comparison with frost observations. Use of RWIS observations for times concurrent with frost observations as input to the frost algorithm provides an estimate of the upper limit of predictability for forecasting frost. Conventional measures of forecast skill for binary events include false alarm rate (FAR), probability of detection (POD), miss rate (MISS), rate at which the model will correctly reject the possibility of frost (CR), critical success index (CSI), and threat score (TS). "CRN" is the normalized correct rejection rate to 100 days. Table 4 lists values for these quantities based on RWIS "forecasts" during the 2001–2002 and 2002–2003 frost seasons. After the 2001–2002 season we observed that RWIS observations could be better correlated with frost occurrences if the RWIS surface temperature was reduced by 1^{o}C. However, this correction did not improve the 2002–2003 skill values.

For the 2001–2002 frost season we used results from 24-hour forecasts by MM5 as input to the frost depth calculation. The surface (skin or radiating) temperature produced within MM5 assumes surface characteristics to be those of the dominant land or water surface within the grid area. Therefore, the surface temperature of MM5 does not adequately represent a bridge surface temperature. For the 2001–2002 test application we did not correct for this known weakness. Despite this shortcoming, the model performed reasonably well in comparison with the RWIS-derived upper limit.

TABLE 4. Frost Forecast Performance with Raw RWIS, Altered RWIS, and Un-altered MM5 Input

Raw RWIS								
Input	FAR	POD	MISS	CR	CRN	CSI	TS	FOS
RWIS (01-02)	0.57	0.57	0.43	0.78	1.00	0.40	0.40	78
RWIS (02-03)	0.20	0.80	0.20	0.97	0.91	0.67	0.67	107
RWIS (total)	0.30	0.67	0.30	0.92	1.02	0.50	0.50	90
RWIS Corrected With One Degree Reduction in Surface Temperature								
Input	FAR	POD	MISS	CR	CRN	CSI	TS	FOS
RWIS (01-02)	0.44	0.71	0.28	0.70	0.89	0.38	0.45	78
RWIS (02-03)	0.70	0.80	0.20	0.89	0.83	0.36	0.47	107
RWIS (total)	0.45	0.75	0.25	0.83	0.92	0.38	0.46	90
MM5								
Input	FAR	POD	MISS	CR		CSI	TS	
MM5 (01-02)	0.52	0.90	0.09	0.31		0.31	0.45	

BridgeT

A one-dimensional finite-difference algorithm (BridgeT) has been developed, following the procedure used by Crevier and Delage (*1*), that simulates heat movement vertically in a bridge in response to evolving conditions (including surface longwave and shortwave radiation and meteorological conditions at top and bottom of the bridge deck) produced by a weather forecast model such as MM5. The bridge temperature distribution is initialized with RWIS wind speed, bridge temperature and two-meter air temperature. BridgeT was tested with MM5 version 3-4 run at Iowa State University, which produces 48-hour forecasts twice daily, with 20km resolution. BridgeT provides the surface temperature required by the frost depth algorithm, which additionally uses air temperature, wind speed, and specific humidity provided by MM5 to predict minute-by-minute changes in frost depth.

BridgeT is currently configured to forecast surface temperature for a concrete interstate overpass similar to the type found at the Ames RWIS site. The surface temperatures forecasted for this type of bridge may not be reasonable for a dissimilar type bridge due to thermo-physical differences in the bridge material. Other common types of bridges, such as concrete bridges with asphalt overlays, may be accommodated in future versions.

Preliminary Results

BridgeT has not yet been run with extensive MM5 input or tested with frost observations. Preliminary tests using MM5 input for late March 2003 has shown that accurate surface temperatures are calculated when reasonably accurate forecasts are supplied to BridgeT. However, BridgeT produces poor surface temperatures if MM5 provides it with poor forecasts.

Figure 1 provides an example of good agreement between BridgeT surface temperature and the RWIS surface temperature. Agreement (after a few hours of model spin-up) during the first 40 hours is rather remarkable. Air temperature shows less agreement during the day. Better agreement for the bridge surface than the air temperature suggests that for this clear-sky event, the surface temperature is much more strongly governed by the radiation balance than by conductive heat gain or loss from air. In this example, the average difference between RWIS and BridgeT surface temperatures was 0.45 C, with a standard deviation of only 1.2 C. The RWIS and BridgeT values had a correlation of over 0.995. BridgeT surface temperature errors mirror the MM5 air temperature errors (i.e., BridgeT overestimates surface temperature when MM5 overestimates air temperature and vice versa), so surface temperature errors can be closely attributed to errors in the driving forecast. BridgeT surface temperatures seem to react appropriately to abrupt radiation changes that occur during sunrise, and decrease correctly during the gradual nighttime radiative cooling period, suggesting BridgeT is correctly configured to the thermal and physical traits of the Ames RWIS bridge. BridgeT's ability to capture the correct nighttime cooling rate is a valuable trait for accurately forecasting the presence and onset time of bridge frost.

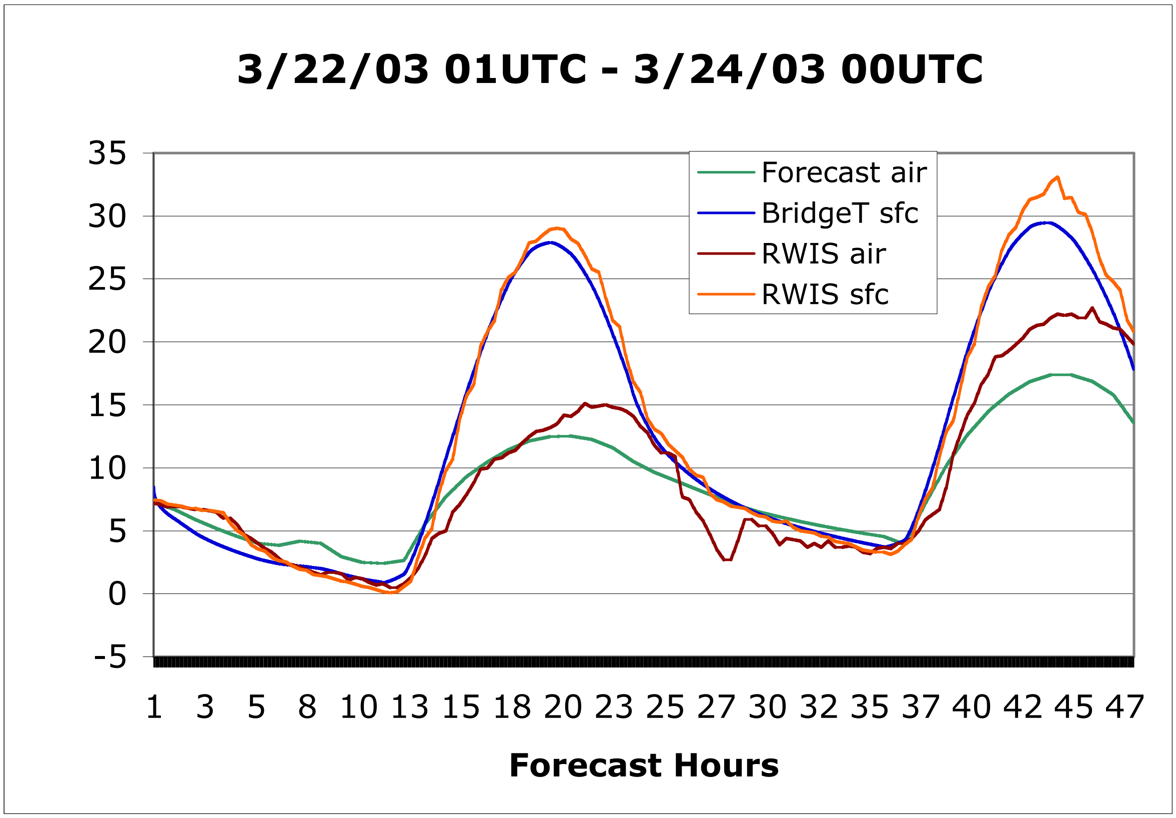

FIGURE 1. BridgeT Performance with Accurate MM5 Input

Figure 2 reveals the difficulty BridgeT has in simulating surface temperature when it is supplied with inaccurate cloud information. During the first forecast day, MM5 had predicted clear skies when it was in fact cloudy, causing BridgeT to overestimate the surface temperature. MM5 air temperature also was too high, thereby contributing to insufficient nighttime cooling of the bridge surface. The forecasted and observed minimum surface temperature was only a few degrees apart, but BridgeT had forecasted the minimum to occur later in the morning. Erroneous MM5 forecasts caused BridgeT to again overestimate daytime surface heating, and overestimate much of the nighttime cooling during the second forecast day. The BridgeT surface temperatures illustrated in Figure 2 was on average 2.5 ºC warmer than RWIS surface temperatures, with a standard deviation of 3.4 ºC. The RWIS and BridgeT surface temperatures had a correlation of only 0.88. The performance of BridgeT obviously is highly sensitive to the quality of MM5 input.

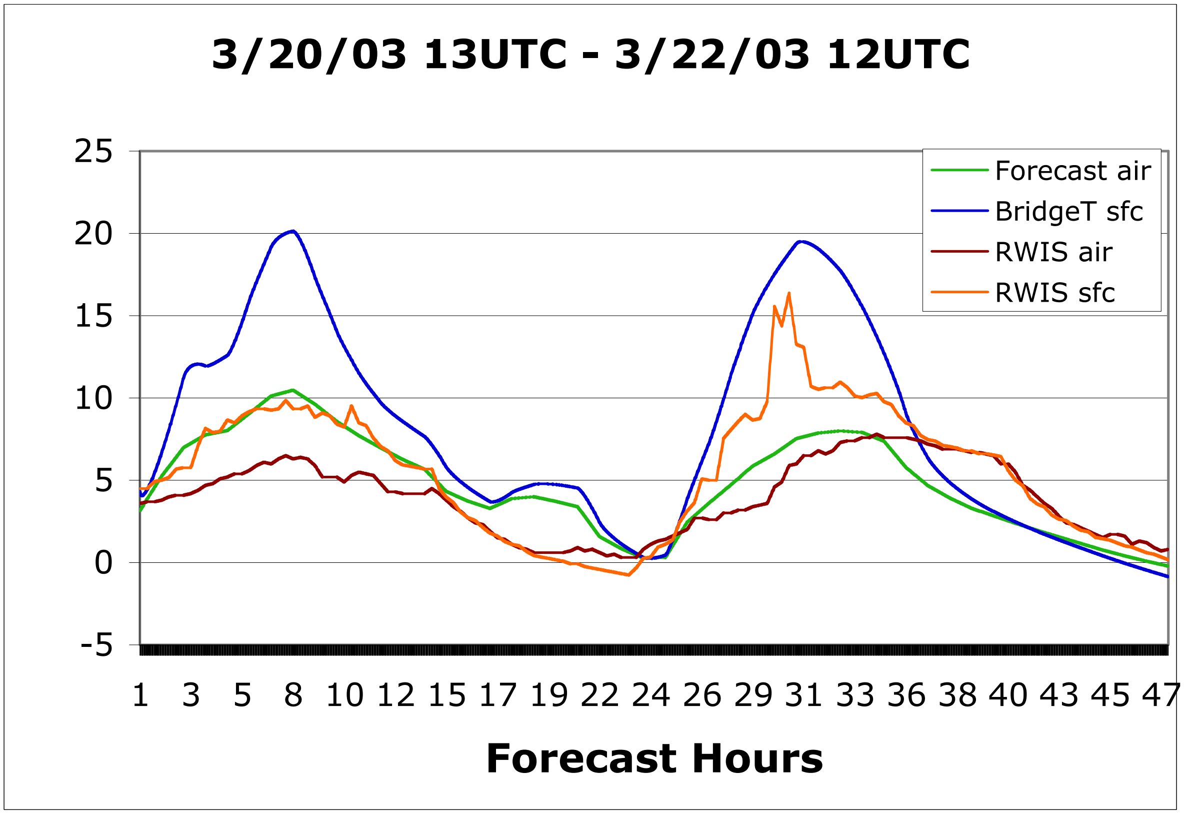

FIGURE 2. BridgeT Performance with Inaccurate MM5 Input

CONCLUSION

Human observations of bridge temperatures and frost occurrence on three bridges within a distance of 2 miles of each other offers a preliminary data set assessing spatial variability of bridge frost and for testing bridge frost forecasts. These observations have been used to validate "forecasts" of frost by a frost accumulation algorithm supplied with concurrent meteorological information from a nearby RWIS site. This procedure will establish an upper limit on predictability that might be expected for a frost forecast procedure. A numerical model for heat transfer in a concrete bridge has been created that takes values from a weather forecast model and calculates the bridge surface temperature. Preliminary comparisons of its results with measured surface temperatures from an RWIS station have demonstrated that the model is capable of supplying surface temperatures within 1 ^{o}C of measured values over a 40-hour forecast period if it is supplied with accurate weather information. The model is particularly sensitive to cloud forecasts because the bridge surface temperature is determined primarily from the radiation budget.

ACKNOWLEDGMENTS

Thank you to Daryl Herzmann, who archived and retrieved important data used in this study, and Dave Flory, for his help with MM5. This project was funded by the Iowa Department of Transportation.

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Use of LiDAR-Based Elevation Data for Highway Drainage Analysis: A Qualitative Assessment

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ABSTRACT

Small-scale elevation models may not provide the accuracy and detail necessary to accurately delineate small watersheds. Moreover, they may not accurately reflect the impact of roads and their ditches on these small watersheds, particularly in flat areas. This research study investigates the differences of utilizing high resolution LiDAR and standard USGS-based elevation data for watershed and drainage pattern delineation along the Iowa 1 corridor between Iowa City and Mount Vernon. Given the limited breadth of the analysis corridor (approximately 18 miles long with LiDAR data available immediately proximate to the road centerline, 0.25 to 1.5 miles), areas of particular emphasis are the location of drainage area boundaries and flow patterns parallel to and intersecting the road cross section.

Key words: highway engineering—hydrology remote sensing—LiDAR

SECTION 1: INTRODUCTION

Hydrology is the science that deals with the occurrence, circulation, and distribution of waters of the earth (*1*). The primary emphasis of hydrology for highway engineering is collection, transport, and disposal of waters originating on, near, or adjacent to the roadway right of way or flowing in highway stream crossings. Adequate hydraulic design is paramount to successful highway engineering. Approximately one-fourth of all highway construction dollars is spent for culverts, bridges, and other drainage structures. (*2*). Insufficient design may also be very costly from the standpoint of mobility and infrastructure deterioration. Therefore, improving the highway engineer's ability to cost effectively accommodate drainage and identify possible deficiencies in existing design may provide significant savings while limiting potential disruption of service due to flood-related road closers.

SECTION 2: PROJECT OVERVIEW

2.1 Scope of Work

Several key components of the hydraulic design for highways are the size, topography, land use, channels/streams, and rainfall of the drainage area. This research study qualitatively assesses whether the use of higher resolution terrain information from Light Detection and Ranging (LiDAR) to better define three of these components; size, topography, and channel location, impacts hydraulic design and deficiency (surety) assessment.

USGS-based elevation data is the most commonly used data source for watershed and drainage pattern delineation. However, USGS data may be too "coarse" to adequately describe surface profiles of watershed areas or drainage patterns around new construction that have been disrupted. Hydraulic design requires delineation of much smaller drainage areas (watersheds) than other hydrologic applications, such as environmental, ecological, and water resource management. For example, a commonly used method in Iowa to determine peak discharge for culvert design (Iowa Runoff Chart) is applicable for rural areas less than 1000 acres in size. By contrast, the smallest surface hydrologic unit (HUC) currently being delineated by the USGS is 10,000 to 40,000 acres in size (12-digit HUC). As a result, highway engineers may require more detailed topographic data to assess impacts due to new construction. LIDAR provides such a dataset.

This study investigates the differences between high resolution LiDAR and standard USGS-based elevation data. In order to evaluate whether terrain data from LIDAR resulted in significant changes in drainage patterns, particularly flow, as compared to USGS terrain data, a pilot study was conducted. The study area is the Iowa 1 corridor between Iowa City and Mount Vernon in Johnson and Linn Counties (Figure 1). Iowa 1 is two-lane roadway throughout the 18 miles of the corridor. The corridor is characterized by a variety of terrain, including rolling farmland and developed (or urban) area. Additionally a river is present which causes significant changes in elevation in portions of the study area. Elevations of the study area range from approximately 650 to 900 feet. Of particular interest is drainage area size and placement of the drainage area boundaries and streams parallel to and crossing the highway. Stream paths were derived from the USGS and LIDAR data using hydraulic modeling and then the accuracy of these locations was also compared to aerial images from the Iowa Department of Transportation (DOT) and Iowa Department of Natural Resources (DNR).

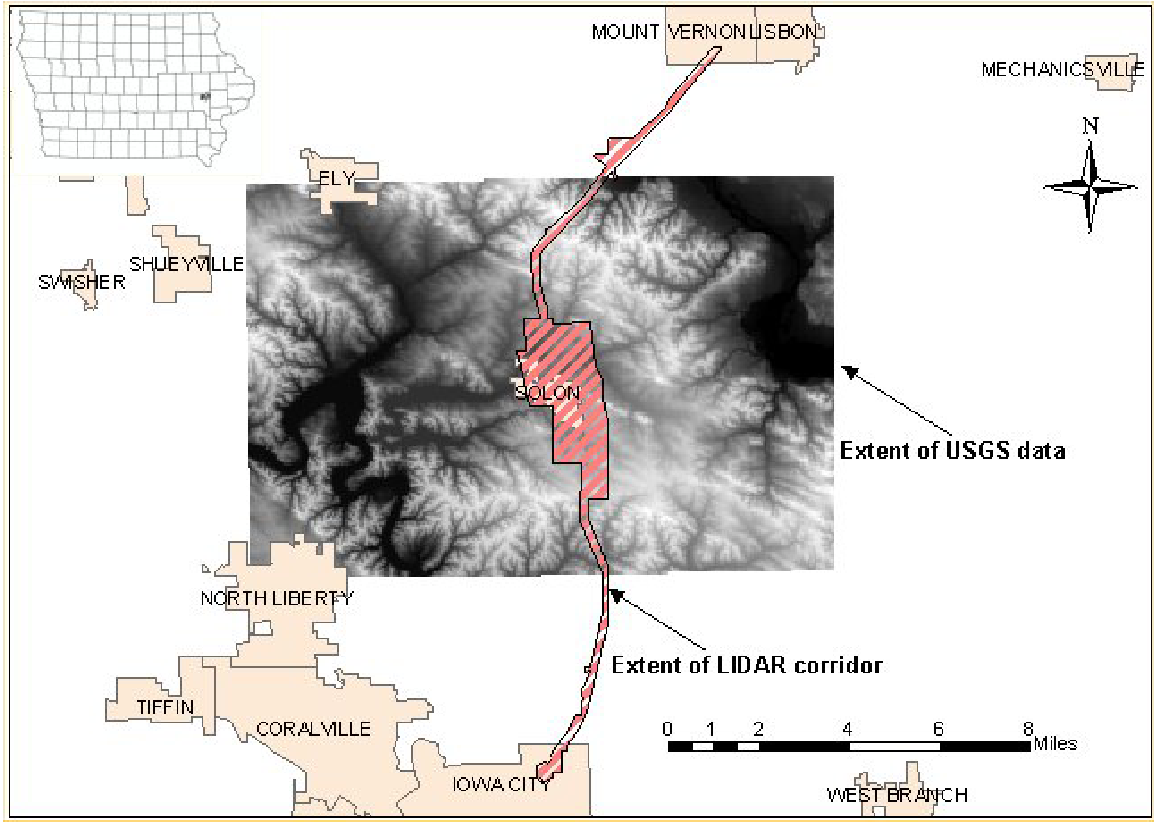

FIGURE 1. Corridor Map

2.2 Potential Benefits

The primary benefit of this study is to determine whether the use of high-resolution terrain data (LiDAR) improves drainage area delineation and the corresponding flow estimates, and how this may influence design of hydraulic features such as culverts. If the increased terrain detail can improve hydraulic design, structures may be more accurately and cost effectively designed and possible deficiencies in existing design may be identified. Possible benefits of deficiency identification include limiting future system failure and the mobility issues accompanying it and the deterioration of pavement and structures resulting from improper drainage.

SECTION 3: BACKGROUND

3.1 USGS DEM

Digital elevation models (DEM) are digital files in raster format consisting of terrain elevations for ground positions at regularly spaced intervals. The U.S. Geological Survey (USGS) produces several digital elevation products which vary by sampling interval, geographic reference system, areas of coverage, and accuracy. Nearly all of the United States has been digitized into grids of elevation values or DEMs over the past few decades by the USGS. The USGS has recently begun creating 7.5' DEMs at a 10 x 10 m resolution with a vertical resolution of 1 foot. USGS DEMs have been used extensively in hydrologic modeling, including drainage basin delineation, storm event modeling, hydrograph creation, and the routing of floods down rivers and through reservoirs. DEMs have also been used in the design of culverts, dams, and detention basins. Specific examples include:

- Calculating subbasin parameters, e.g. slope, slope length, and defining the stream network for the Great Salt Plains Basin (*3*).
- Creating a flash flood prediction model for rural and urban basins in New Mexico, which included delineation of the basin and calculating the slope and aspect within the basin (*4*).
- Designing discharge for flow conveyance structures on Texas highways (*5*).
- Improving the understanding of drainage areas and hydrological flow paths in urban areas adjacent to San Francisco Bay (*6*).

USGS DEMs, however, do have limitations. One recent study compared 30-m USGS DEMs with field data and found that they correctly predicted slope gradient at only 21% and 30% of the field sampling locations in two study sites (*7*). Several other studies have found similar results (*8*) (*9*) (*10*). Numerous authors have argued that DEMs with spatial resolutions of two to ten meters are required to represent important hydrologic processes and patterns in many agricultural landscapes (*11*).

3.2 LiDAR

Since the early 1970s, Light Detection and Ranging (LiDAR) has been used for terrain definition. The LiDAR instrument transmits a beam of light to a target. Some of this light is reflected/scattered back to the instrument. The time for the light to travel out to the target and back to the LiDAR is used to determine the range to the target. LiDAR works best with low vegetation but even in heavy vegetation some light pulses penetrate and are returned so that distance to the ground can be measured. Algorithms are then used to "filter" out the vegetation and buildings leaving what is referred to as a "bare earth" model, which contains precise ground elevations that can be determined after. The resolution and accuracy of aerial-based LiDAR vary among vendors, but a reported horizontal resolution of two meters is common. Reported horizontal accuracies of 1m root mean square error (RMSE) and vertical accuracies of 15cm RMSE, or greater, are also common.

LiDAR terrain data have been used for a number of different applications, including generating contours, creating 3D terrain views, determining fault locations, modeling steep slopes, critical areas and streams and delineating drainage basins (*12*). LiDAR data have recently been used in two extensive hydrologic projects in Texas and North Carolina. Specifically, LiDAR data are being collected to assist in the creation of a drainage system model for Corpus Christi, Texas and in the development of flood insurance rate maps in North Carolina. LiDAR data were also used to capture very small drainage features, such as narrow ditches and potential areas where ponding of water might occur. These LiDAR data were used to interpret drainage patterns producing a detailed drainage network, which was highly representative of all actual water features (*13*).

SECTION 4: METHODOLOGY

4.1 Software tools

Two software tools were used in this study: ArcView Version 3.3 and HEC-GeoHMS Version 1.0. ArcView is a geographic information system (GIS) created by ESRI. The Geospatial Hydrologic Modeling Extension (HEC-GeoHMS) is a software package developed by the Army Corp of Engineer's Hydrologic Engineering Center that utilizes ArcView and its Spatial Analyst Extension to develop hydrologic modeling inputs. HEC-GeoHMS analyzes digital terrain data and transforms the resulting drainage paths and watershed boundaries into a hydrologic data structure representing the watershed response to precipitation (*14*).

4.2 Data Assimilation

As was discussed previously, hydraulic design entails several key components. The primary components investigated in this study are drainage area size, topography, and channel location. The primary data element used to derive and/or assess these components is the digital terrain data. Two sources of terrain data were obtained, LiDAR and USGS DEM. Two 7.5-Minute USGS DEMs (10- x 10-meter data) were also obtained for the corridor. These DEMs possessed reported accuracies of 23 feet (7 meters) vertically and 33 feet horizontally (10 meters) (*15*). The USGS data covered the 18-mile length of the Iowa 1 and was 69,000 feet (21,000 meters) wide and extended at least 27,000 feet (8200 meters) on both sides of the roadway (Figure 1).

LiDAR data for the study area was collected for another research project by EagleScan Corporation. A bare earth model from the LiDAR data was available to the study team. The reported accuracy of the LiDAR data was one meter RMSE horizontal and 15 centimeters RMSE vertical. Horizontal resolution was two meters. Although USGS data were available for 27,000 feet around Iowa 1, LiDAR data were collected for a different purpose and were available for the length of the study corridor but the data only extended 0.25 to 1.5 miles on both sides of Iowa 1 depending on the location. Data for the largest area were available near Solon, at the site of the proposed bypass (Figure 1).

Planimetric data and two sets of aerial images were also obtained for the study corridor. Planimetric CAD files, including culvert locations, and aerial photographs for the corridor were obtained from the Iowa DOT. Color infrared (CIR) aerial photographs for Johnson and Linn Counties were obtained from the Iowa DNR.

4.3 Watershed and Stream Creation

HEC-GeoHMS employs a multi-step process to define streams and watershed boundaries from terrain data. The user may employ either a step-by-step or batch processing approach to derive the stream and watershed coverages. The user has more control in the step-by-step approach, allowing interactive review and verification of the incremental results. The batch process develops all incremental and final data sets allowing only limited user input. This study utilized both approaches.

Before watersheds and streams can be delineated, elevation grids were created from the point-based LiDAR elevation data. An elevation grid consists of a grid of cells, square or rectangular, in raster format having land surface elevation stored in each cell. Four distinct elevation grids were created for this study. These grids will be discussed in the next section. Upon creation of the grids, HEC-GeoHMS employs the following eight steps to create watershed and stream coverages from the input terrain data.

1. Depressions are removed from the source DEM to allow water to flow across the landscape.

2. The direction of flow for each cell is determined by the direction of the steepest descent. Possible directions of flow are the eight cardinal directions.

3. Flow accumulation is calculated for each cell by determining the number of upstream cells that drain into it.

4. Streams are defined based on a user defined threshold value (area or number of cells). The flow accumulation for a particular cell must exceed the threshold to be included in the stream network.

5. Streams are segmented between successive junctions, a junction and an outlet, or a junction and a drainage divide.

6. Watersheds, or subbasins, are delineated for each stream segment.

7. Stream and watershed grids (raster) are converted to vector representations.

8. Aggregated watersheds are created by merging upstream subbasins at every stream confluence. (*16*)

4.4 Terrain Data Sets

Four distinct terrain data sets (elevation grids) were created from the LiDAR and USGS point-based elevation data. Given the limited breadth (area) of the LiDAR data set relative to the USGS DEMs, a strict comparison of the two terrain data sets could not be performed. For example, LiDAR data did not always cover a complete watershed or contributing area for a downstream stream. However, elevation grids were created in a manner that would best facilitate comparison of the available data. This section discusses the elevation grids and factors integral in their creation.

4.4.1 LiDAR Bare Earth. Using the LiDAR bare earth data sets, an elevation grid of 10m-cell size was created for the corridor. While a finer grid could be created from the LiDAR data set, given the density of data points (1 every 25 m2), the 10m-grid was selected for processing efficiency and consistency with the USGS data set. The processing time required to create a 5m grid for the entire corridor was such that it was deemed unrealistic that this would be repeated in practice, with some exceptions without higher performance computers. (An example of a 5m grid for a portion of the corridor is presented later in this document.) The 10m-grid size was also a reasonable size for the USGS data. While some of the terrain detail provided by the LiDAR may be lost, a more consist comparison of the USGS data could be performed. Areas of emphasis were watershed and stream delineation in the immediate vicinity of the highway.

Using HEC-GeoHMS, watershed, stream configuration, and flow accumulation grids were created for this elevation grid. An area threshold value of one percent, or approximately six acres, was used for stream definition (Figure 2).

4.4.2 USGS DEMs. An elevation grid of 10m-cell size was created from the mosaiced USGS DEMs covering the Ely and Solon area. The area represented by these DEMs was much greater than that of the LiDAR data, encompassing both large and small watersheds. Using HEC-GeoHMS two different sets of stream configurations, watersheds, and flow accumulation grids were derived. The first set was created using the batch-processing mode and a default value of one percent, or 200 acres, was used as the stream threshold. These data sets were created to assess the sensitivity of watershed size to the input threshold value. As expected, the watersheds were much larger and the stream coverage was fairly sparse, limited to major streams or channels, because runoff over a greater area was required to initiate a stream. (Figure 3)

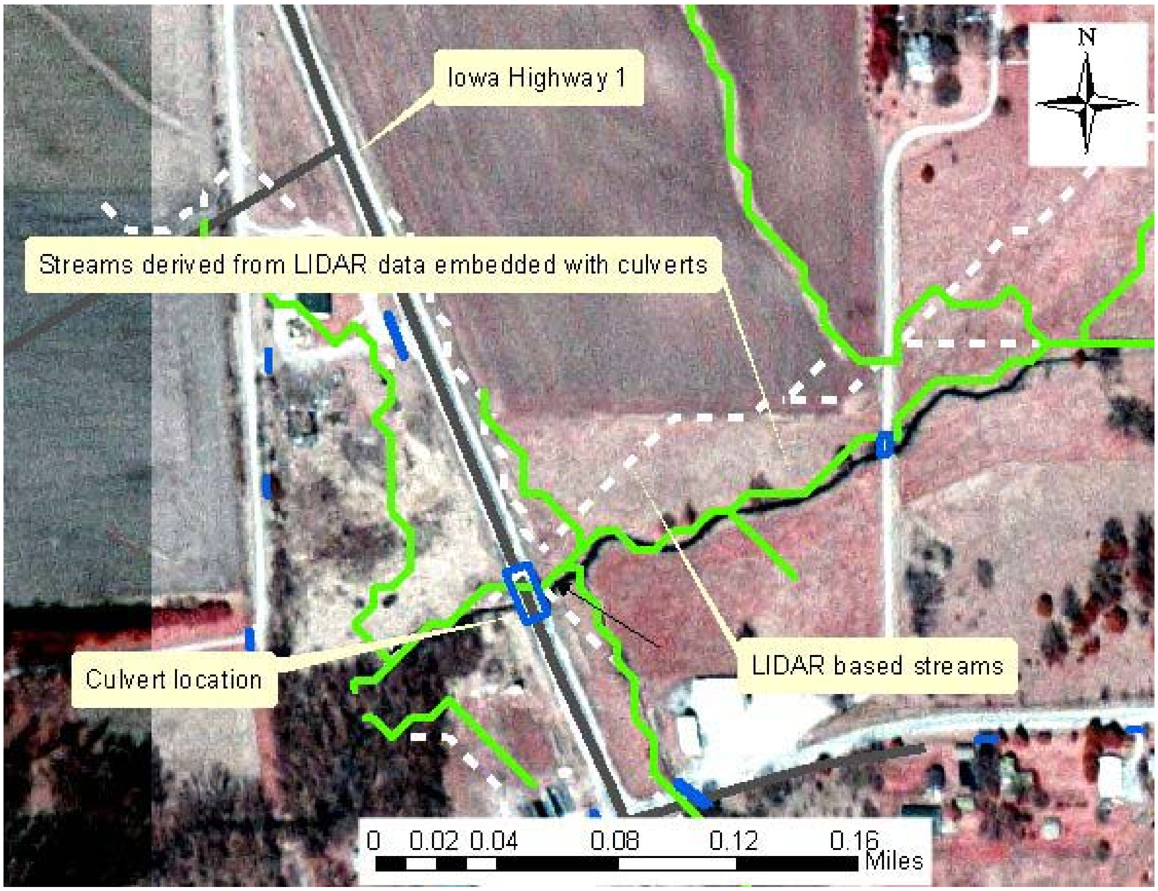

FIGURE 2. Lidar-Based V. Lidar Embedded with Culverts-Based Stream Coverage, Six Acre Threshold for Stream Initiation

The second set of stream configurations, watersheds, and flow accumulation grids was created to compare to the LiDAR results. An area threshold value of approximately six acres (0.018 percent) for stream definition was used to be consistent with the watersheds generated from the LiDAR terrain data.

4.4.3 LiDAR Bare Earth Supplemented with Culverts. In an attempt to influence stream flow through known hydrologic structures, a 10m-grid file of existing bridge and culvert locations, identified from Iowa DOT planimetric CAD files, was created. The elevation of the grid cells at these locations was set to 600 feet, approximately the same elevation as the surrounding terrain, but lower than the surrounding pixels so as to force the streams to flow into the culverts. This grid was then merged with the elevation grid made from the individual LiDAR grids to create a LiDAR grid with culverts embedded. HEC-GeoHMS was used to derive watersheds, a stream configuration, and a flow accumulation grid. These were created using the batch-processing mode in which the default value of one percent (approximately six acres) was used as the stream initiation threshold (Figure 2).

4.4.4 LiDAR Bare Earth Supplemented with Culverts and USGS DEMs. A final 10m-elevation grid was created to assess the impact of utilizing the more detailed terrain data (from LiDAR) in the vicinity of the roadway in conjunction with the more extensive USGS data, which encompasses entire watersheds. The elevation grid created from the USGS elevation grid was merged with the LiDAR bare earth and culvert elevation grid. The combined LiDAR and culvert data were utilized at areas of coincidence or overlap with the USGS elevation grid, yielding more detailed terrain data in the vicinity of the highway. The resulting elevation grid consisted of data from the USGS, LiDAR, and culvert elevation grids. HEC-GeoHMS was used to derive watersheds, a stream configuration, and a flow accumulation grid with an area threshold value of approximately six acres (0.018 percent of the largest drainage area).

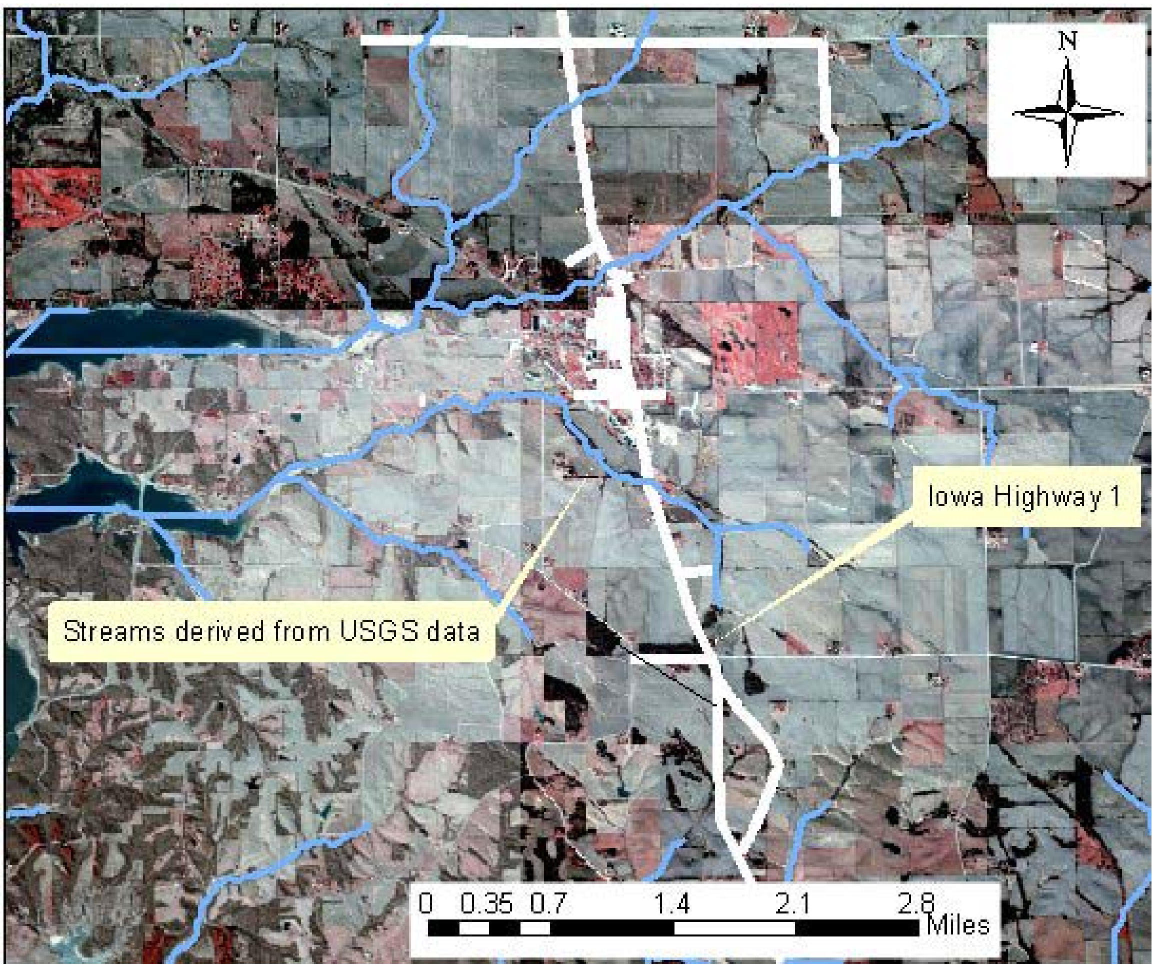

FIGURE 3. USGS-based Stream Coverage, 200 Acre Threshold for Stream Initiation

SECTION 5: RESULTS

5.1 Stream Locations

Since established drainage patterns are disrupted by highway construction, it is important to know the locations of existing streams, particularly for the design of new channels and structures to accommodate their flows. Using the Iowa DOT corridor and Iowa DNR CIR aerial images, the relative accuracy and reasonableness of existing stream placement for each elevation grid created from HEC-GeoHMS was assessed. In addition, stream location with respect to known culvert locations and the highway roadside was reviewed.

The streams (drainage channels) produced from the LiDAR-based elevation grid appeared proximate (at varying levels of accuracy) to streams identifiable from the aerial images and known drainage structure locations. The stream coverage was also fairly dense, as a result of the relatively small drainage areas defined, but lacked curvilinear detail. Both intermittent channels as well as continually flowing streams appeared to be represented. Locations of possible drainage and base inundation parallel to the roadway were also visible. These locations could represent locations of potential base failure and, in turn, increased pavement deterioration.

Stream placement was not without spatial inaccuracies. Accuracies tended to vary throughout the corridor. Streams were generally parallel to visible streams but offset from a few meters to over 50 meters. A possible explanation for these occurrences is sensitivity to subtle terrain changes and errors. Specifically, the LiDAR bare earth data set was found to occasionally contain non-bare earth features, such as buildings, trees, and other vegetation. The presence of these features yielded incorrect terrain representations.

With the addition of the culvert locations to the LiDAR elevation grid, the alignments of natural streams appeared more accurate and detailed (meandering and curvilinear), again indicating sensitivity to subtle terrain changes. Inclusion of culverts appeared to supplement/enhance roadway cross-section information at locations where LiDAR may not be able to collect all terrain surfaces, e.g. ditch foreslope, bottom, and back slope. At approximately half of the culvert locations, the stream alignment was improved to the point that the stream now flowed through the culvert. Stream alignment also improved upstream from the culvert location, better mirroring the streams visible in the aerial photographs.

As mentioned previously, a 5m-elevation grid was created for a portion of the corridor. The stream coverages created from this grid and the 10m-grid from the LiDAR data are presented in Figures 4 and 5. As is apparent in these figures, the two stream coverages closely mirror each other. Alignment differences of approximately 50 meters were present at several locations, in Figure 5, but the 10-m grid stream coverage was actually closer to the existing stream alignment. Therefore, the more finely defined elevation grid (5m) did not appear to yield a superior stream coverage and was more greatly impacted by terrain inaccuracies or false bare earth elevations.

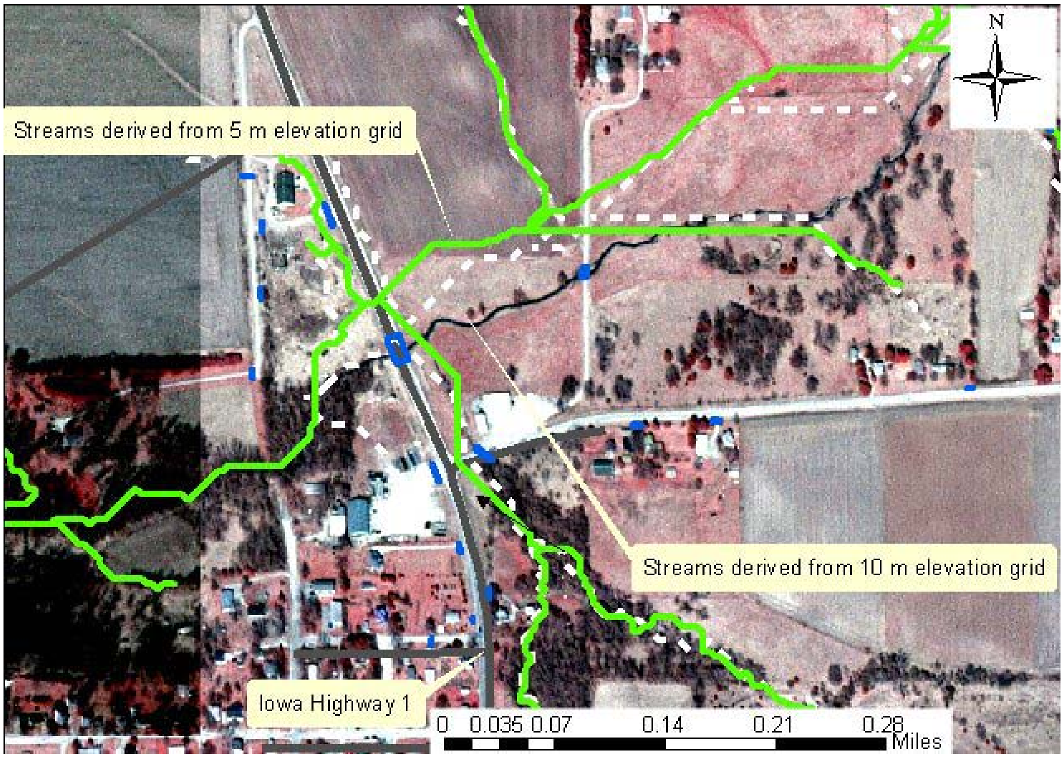

FIGURE 4. LiDAR-based Stream Coverage, 5m v. 10m Elevation Grid (Overview)

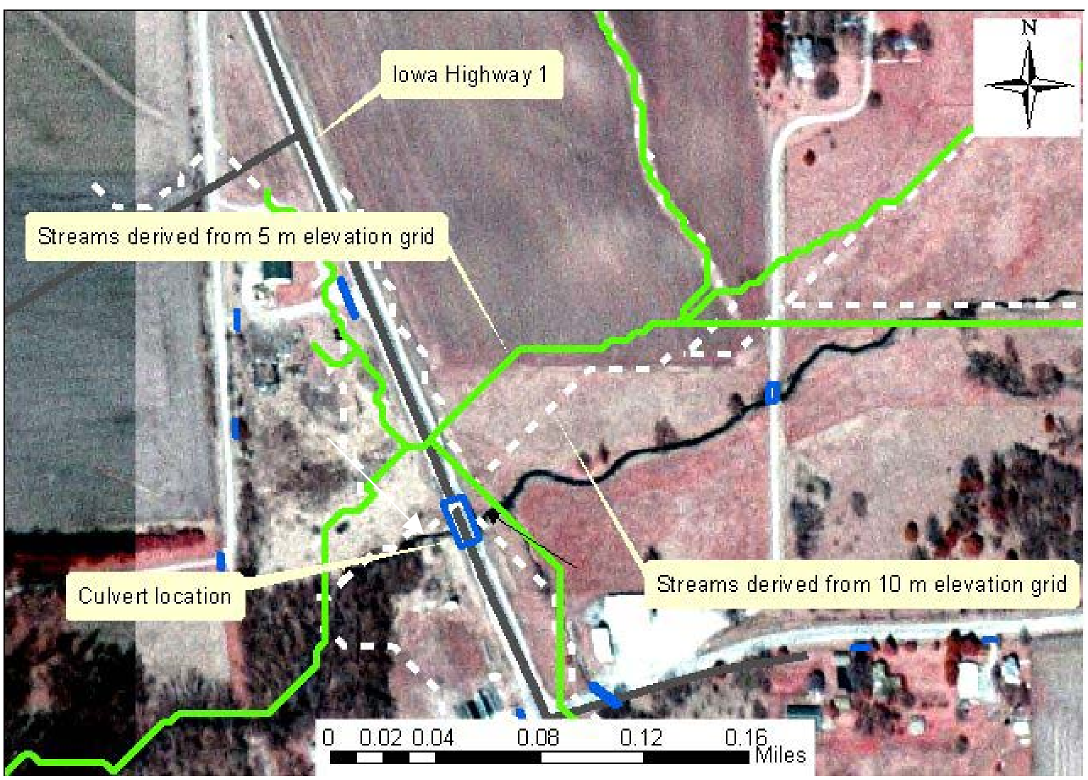

FIGURE 5. LiDAR-based Stream Coverage, 5m v. 10m Elevation Grid (Zoomed in)

The USGS elevation grid yielded similar stream coverages as the LiDAR elevation grid in the vicinity of the highway (Figures 6 and 7). Streams (drainage channels) were proximate to streams identifiable from the aerial images and the known culvert locations. The stream coverage was also dense but lacked curvilinear detail. Accuracies tended to vary throughout the corridor, from a few meters to over 50 meters. In contrast to the LiDAR data (which may be too sensitive to terrain detail), this may result from errors in elevation or lack of terrain detail. Other than differences in stream alignment, the primary difference between the LiDAR and USGS-based stream coverages is definition of minor, feeder streams. The length and alignment of these streams differed as well as the presence (or absence) of these streams between the two coverages. As a whole, the USGS-based elevation grid yielded comparable results to the LiDAR data set without drainage structures.

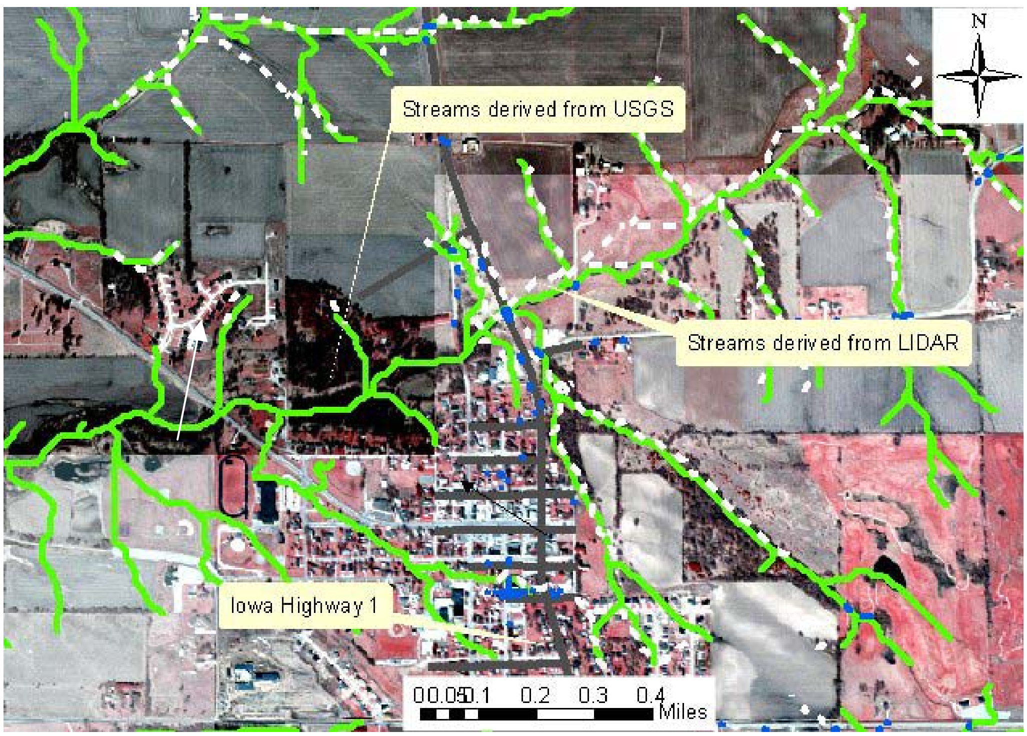

FIGURE 6. USGS-based v. LiDAR-based Stream Coverages (Overview)

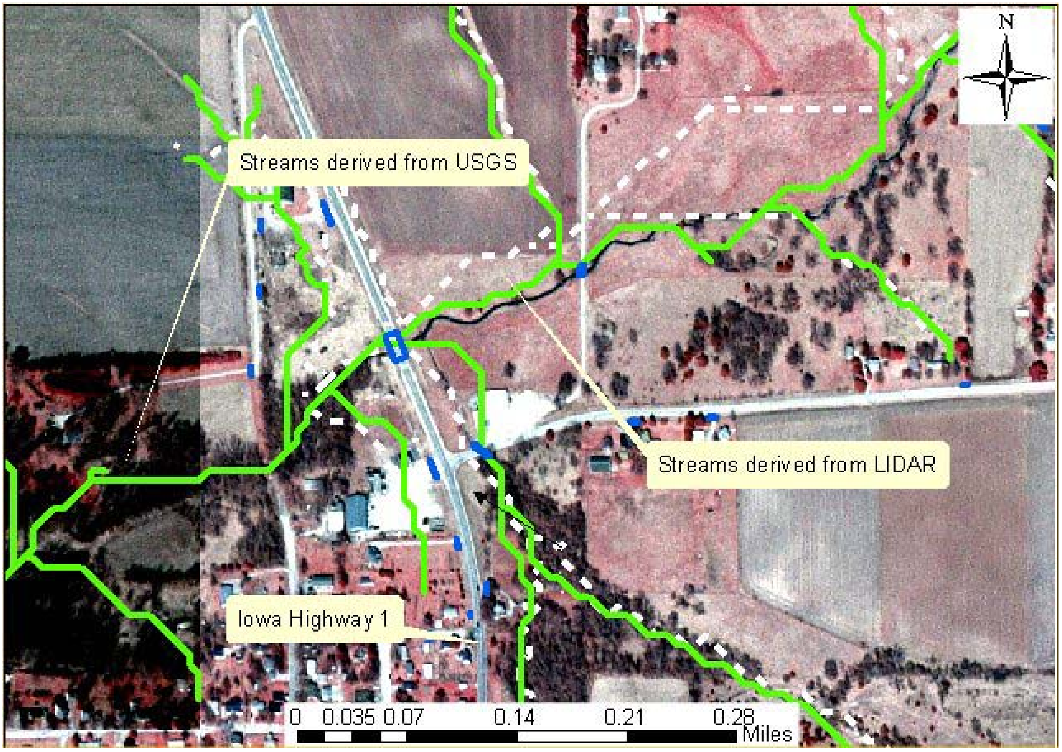

FIGURE 7. USGS-based v. LiDAR-based Stream Coverages (Zoomed in)

Lastly, many of the observations of the LiDAR grid supplemented with culvert locations are also applicable for the combined USGS, LiDAR, and culvert elevation grid (Figure 8). The stream coverage in the areas extending beyond the LiDAR data (USGS data only) appeared to possess the same relative accuracy and detail as the areas where LiDAR was present. Again, the streams (drainage channels) appeared proximate to streams identifiable from the aerial images and the known culvert locations. The benefit of this coverage is two fold. First, complete watershed or contributing areas, extending beyond the LiDAR coverage area, can be derived for downstream streams. Second, inclusion of the drainage structures, in both this elevation grid and the LiDAR grid alone, appeared to increase the accuracy of stream alignment at and upstream from the culvert. This was observed at approximately half of the locations, while minor/no improvement was observed at one-third of the locations, and a poorer alignment resulted at nearly 10% of the locations.

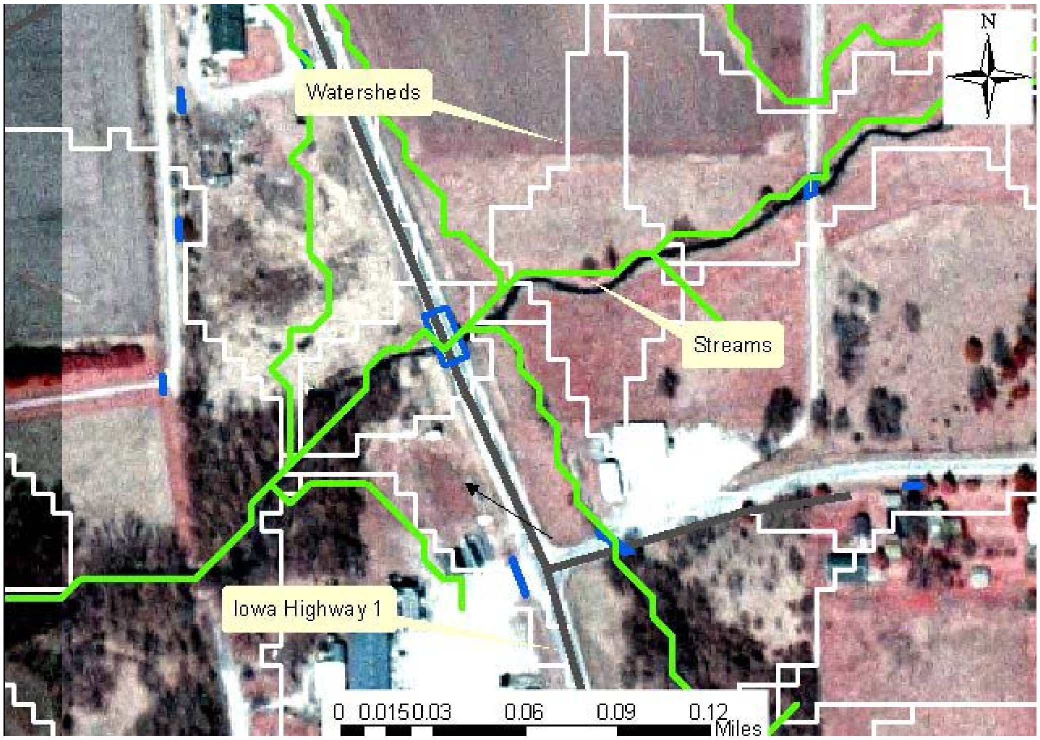

FIGURE 8. Combined LiDAR, USGS, and Culvert Stream Coverage

5.2 Watershed Boundaries

While knowledge of existing streams is important in highway design, the size of the drainage areas contributing to the flow in these streams is critical in the design of hydraulic structures. Of particular interest is the sensitivity of watershed delineation to improved terrain detail. In other words, can the size and nature of watersheds produced from different terrain models impact design inputs, such as flow accumulation?

Given the limited extent of the LiDAR data, only the small watersheds defined in areas where both LiDAR and USGS data existed could be compared. As presented earlier, a relatively small area threshold (six acres) was used to define the streams. This, in turn, also yielded relatively small watersheds. Traditionally, highway engineers do not delineate watersheds using this area-based approach. Topographic maps are used to identify an outlet and all highpoints upstream from the outlet. The highpoints are then connected to define the watershed. Roadways, which are typically not visible on a topographic map, are also utilized to delineate the watershed.

HEC also used roadways to create watershed boundaries with the LiDAR data. This is possible because the horizontal resolution of the LiDAR data often facilitates detection of the roadway within the terrain (Figure 10). This, however, does not hold true for all watersheds along the roadway and seldom, if ever, holds true for the USGS-based data (Figure 10). The horizontal resolution of the USGS DEM, at 10m, is too great to detect a two-lane roadway (Figure 14).

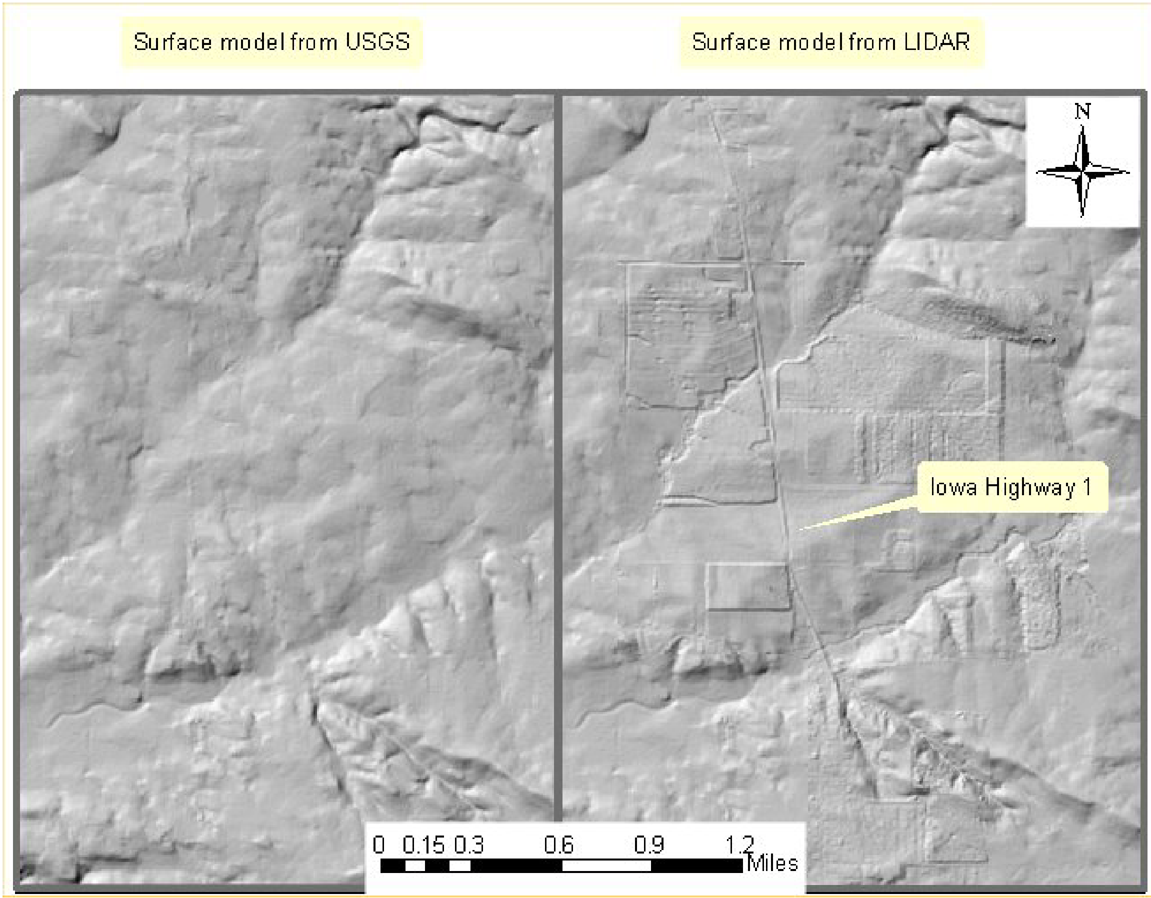

FIGURE 9. Comparison of surface model derived from USGS and LIDAR (10m-grid)

Watersheds were affected by roadway alignments where LiDAR data were present, but near the edges of the LiDAR data set and in area described only by the USGS elevation data, the roadway did not affect the watershed configuration.

In general, the watersheds delineated from the LiDAR-based elevation data appeared to very sensitive to changes in terrain, particularly in areas of modified terrain. This resulted in smaller, more irregularly shaped drainage areas. The USGS-based watersheds were typically larger and less complex. Yet, in many instances the LiDAR and USGS watersheds were similar in extent and/or border definition. The addition of drainage structures to the elevation grids yielded watersheds of limited differences.

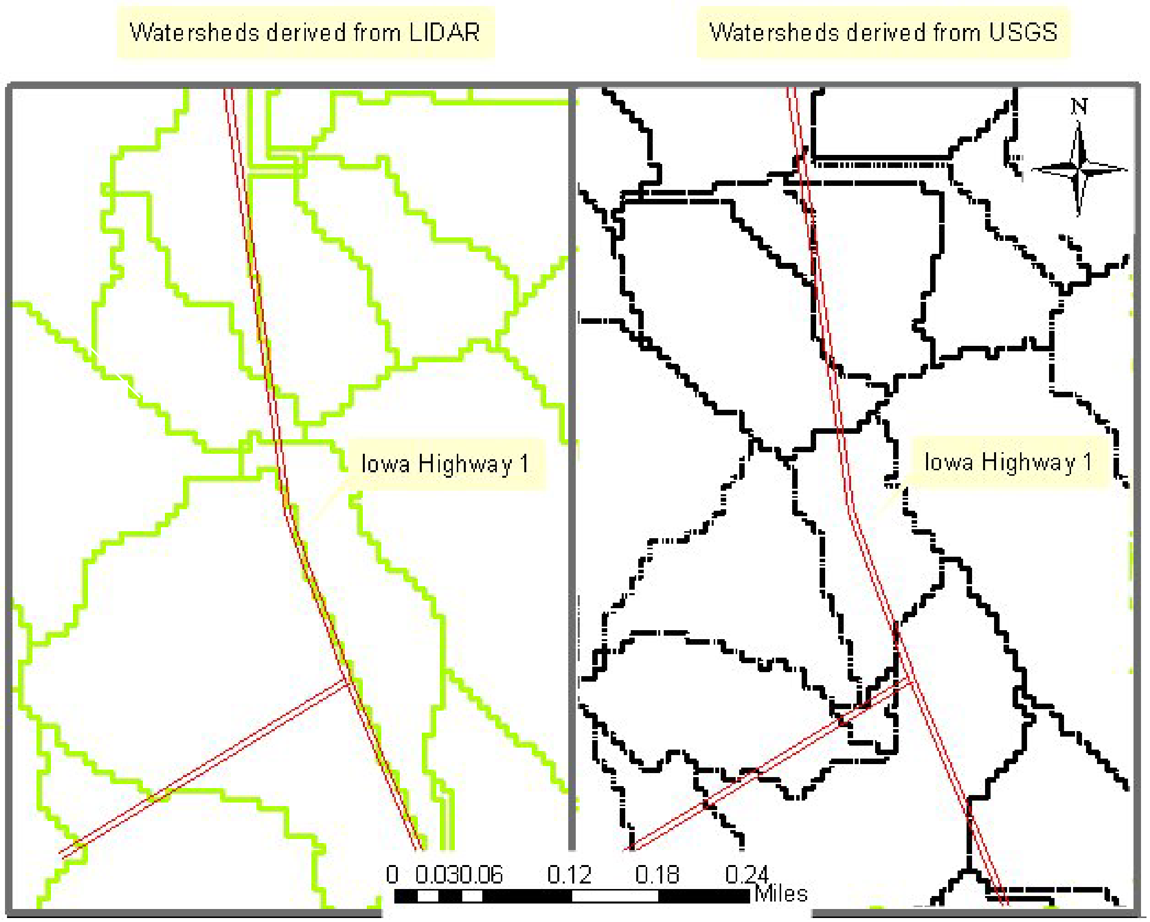

FIGURE 10. LiDAR-base v. USGS-base Watershed Boundaries, Six Acre Minimum Watershed Area

5.3 Flow Accumulation

Because no outlets were defined during watershed delineation, watershed size is not an appropriate measure to assess the possible impact of improved terrain detail on hydraulic design. Flow accumulation, which is the number of upstream cells that drain into a cell, is a more appropriate measure. By identifying the area contributing to flow at each drainage structure, design flow at each drainage structure can be calculated. Flow accumulation and the resulting design flow for the two different terrain models may then be used to assess the possible impact of terrain detail (resolution) on hydraulic design and existing structural surety.

Unfortunately, a true comparison of flow accumulation resulting from LiDAR and USGS-based elevation data could not be performed. Since most drainage areas extended beyond the LiDAR coverage area, only a comparison of LiDAR data (embedded into USGS data) and USGS data alone could be performed. With a few exceptions (less than ten), the primary contributor to most flow accumulation values was the USGS-based data. Therefore, any possible differences in flow accumulation at a structure would be limited to the portion of the drainage area with LiDAR data present.

The flow accumulation for all of the hydraulic structures (bridges and culverts) was identified for the elevation grid and the combined LiDAR, USGS, and culvert elevation grid. The difference in flow accumulation at each location was then calculated. The USGS-based flow accumulation (area) for approximately 90% of these structures was less than 40 acres. The average difference in area between the combine LiDAR data and USGS data was less than four acres. Using the Iowa Runoff Chart to determine peak discharge, and assuming the same flood frequency (50 year), land use (mixed cover), and slope (hilly), the average difference in peak flow was 16.4 ft^3/sec and the range of differences was from 0.4 to 100 ft^3/sec. By comparison, if rolling terrain is assumed instead of hilly terrain for a 40-acre drainage area, the difference in peak flow is approximately 25 ft^3/sec. Therefore, for the locations observed with the limited LiDAR data, the factors utilized to calculate peak discharge have as much, or more, impact as the flow accumulation area provided by different terrain models.

SECTION 6: CONCLUSIONS/RECOMMENDATIONS

Traditional highway hydrology does not appear to be significantly impacted, or benefited, by the increased terrain detail that LiDAR provided for the study area. In fact, hydrologic outputs, such as streams and watersheds, may be too sensitive to the increased horizontal resolution and/or errors in the data set. However, a true comparison of LiDAR and USGS-based data sets of equal size and encompassing entire drainage areas could not be performed in this study. Differences may also result in areas with much steeper slopes or significant changes in terrain.

LiDAR may provide possibly valuable detail in areas of modified terrain, such as roads. Better representations of channel and terrain detail in the vicinity of the roadway may be useful in modeling problem drainage areas and evaluating structural surety during and after significant storm events. Furthermore, LiDAR may be used to verify the intended/expected drainage patterns at newly constructed highways.

LiDAR will likely provide the greatest benefit for highway projects in flood plains and areas with relatively flat terrain where slight changes in terrain may have a significant impact on drainage patterns.

ACKNOWLEDGEMENTS

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Experimental Evaluation of Precast Channel Bridges

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ABSTRACT

The precast channel bridge (PCB) is a short span bridge that was commonly used on Iowa's secondary roads approximately forty years ago. Each PCB consists of eight to ten simply supported precast panels ranging in length from 5.8m to 11.0m. The panels resemble a steel channel in cross-section; the web is orientated horizontally and forms the roadway deck and the legs act as shallow beams. Bundled reinforcing bars in each leg act as the primary flexural reinforcement.

Many of the approximately 600 PCBs in Iowa show signs of significant deterioration. Typical deterioration consists of spalled concrete cover and corrosion of the bundled primary reinforcement. The objective of this research was to access the structural sufficiency of the deteriorated PCBs through field and laboratory testing.

Four deteriorated PCBs were instrumented with strain gages to measure strains in both the concrete and reinforcing steel and transducers to measure vertical deflections. Response from loaded trucks was recorded and analyzed. Test results revealed that all measured strains and corresponding stresses were well within acceptable limits. Likewise, measured deflections were much less than the recommended AASHTO value.

Laboratory testing consisted of loading twelve deteriorated panels to failure in a four point bending arrangement. Although all panels exhibited significant deflection prior to failure, the experimental capacity of eleven panels exceeded their theoretical capacity. The experimental capacity of the twelfth panel, an extremely distressed panel, was only slightly below its theoretical capacity.

Key words: deterioration—flexural capacity—precast channel bridge—precast concrete bridge

INTRODUCTION

Recent data compiled by the National Bridge Inventory revealed 29% of Iowa's approximate 24,600 bridges were either structurally deficient or functionally obsolete. This large number of deficient bridges and the high cost of needed repairs create significant problems for Iowa and many other states. The research objective of this project [1] was to determine the load capacity of a particular type of deteriorating bridge – the precast channel bridge (PCB) – that is commonly found on Iowa's secondary roads. The number of these precast concrete structures requiring load postings and/or replacement can be significantly reduced if the deteriorated structures are found to have adequate load capacity or can be reliably evaluated.

Approximately 600 PCBs currently exist in Iowa. These bridges were constructed primarily in the 1950's and 1960's. A typical PCB span is 5.8m to 11.0m long and consists of eight to ten simply supported precast panels. Abutments and piers typically consist of cast-in-place reinforced concrete caps supported by timber piles. A curb is cast along the top edge of the two exterior panels and steel or concrete rail post bolt to the sides of these panels. A typical bridge cross-section is presented in Figure 1.

Two similar standard panel designs are found in Iowa. The primary difference between the two designs is the configuration of the joint between adjacent panels. Type I panels, shown in Figure 2, are joined by transverse bolts and a continuous grouted shear key. The joint between adjacent Type II panels consists of a concrete-filled galvanized pipe and transverse bolts. Details of the Type II panels are presented in Figure 3.

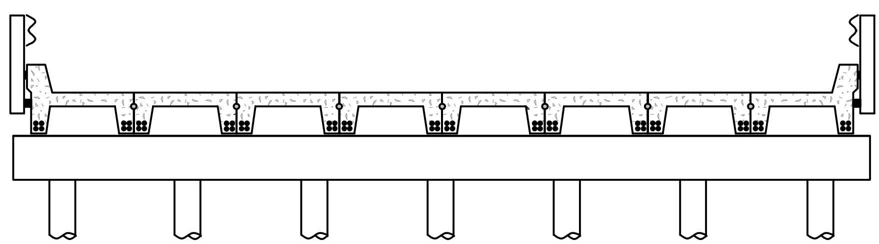

FIGURE 1. Typical PCB Cross-Section near Abutment (Roadway Crown not Shown)

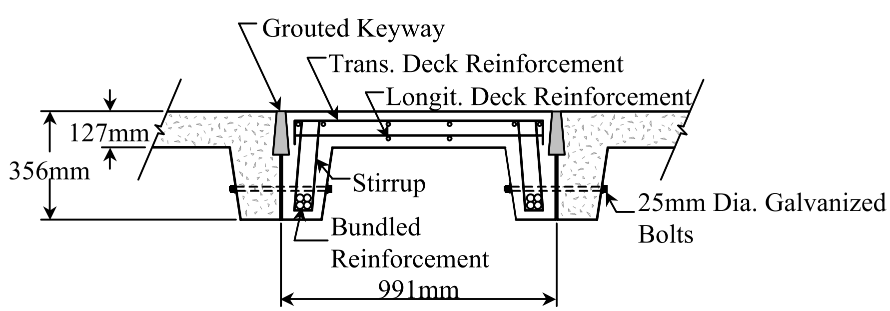

FIGURE 2. Type I PCB Panel Cross-Section

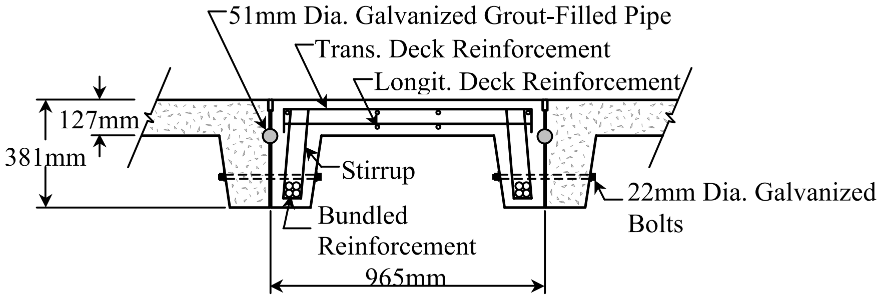

FIGURE 3. Type II PCB Panel Cross-Section

Many of Iowa's PCBs are heavily deteriorated. Typical deterioration consists of spalled concrete and significant corrosion of the primary reinforcing steel. In many cases, as shown in Figure 4, the bottom bars in a stem are exposed over almost the entire span. In other cases, longitudinal cracks and rust stains are often found on the bottom and side of the panel stems. The effects of this deterioration, such as loss of cross-sectional reinforcement area and bond, have lead to concerns over the ability of a deteriorated PCB to safely support legal loads.

FIGURE 4. Typical PCB Deterioration

FIELD TESTING

Four deteriorated PCBs were selected for field testing from the results of a questionnaire sent to all 99 Iowa counties. Selection criteria were the extent of the deterioration, whether the bridge was scheduled for replacement, and location. The geometry and panel type of each bridge is presented in Table 1.

TABLE 1. Geometry and Panel Type for Field Tested PCBs

Bridge	1	2	3	4
No. of Spans	1	1	2	1
Out-to-Out Panel Length (m)	9.4	11.0	9.4	7.6
No. of Panels/Span	10	8	9	9
Panel Type	II	II	I	I

TESTING METHOD

Each bridge was instrumented in a similar fashion. Since each span was simply supported, the instrumentation was located at midspan so that the maximum response could be measured. Electrical resistance strain gages were bonded to the primary reinforcement and concrete deck. Transducers were attached to the panel stems to measure vertical deflection and differential deflection across the panel joints.

Legally loaded tandem axle dump trucks or a truck tractor-simitrailer combination was used to load the bridges. The trucks crossed each bridge in three designated lanes (left, right, and center) at a slow speed. Data were continuously recorded by an electronic data acquisition system (DAS); tape switches connected to the DAS identified the longitudinal location of the truck.

TEST RESULTS

The field test results were very useful for accessing two specific areas of performance. First, deflection and strain data from a given panel were analyzed to determine if the loads placed on the panel induced stresses above allowable limits. Second, deflection data from all of the panels for certain transverse and longitudinal load positions were used to calculate transverse load distribution factors.

Maximum tensile steel strains for all four PCBs ranged from 110 microstrain for Bridge 3 to 208 microstrain for Bridge 2. Taking Young's modulus of the reinforcement as 200 MPa relates to maximum stress levels ranging between 0.022 MPa and 0.042 MPa. The Iowa Department of Transportation standard by which the PCBs were constructed stipulates that stress in the reinforcement not exceed 0.138 MPa. Including the stress caused by the self weight of the panel (approximately 0.034 MPa) results in a total stress in the reinforcement that is slightly more than one half of the allowable stress. This indicates that the legally loaded trucks induced a safe level stresses in the PCBs. Similarly, live load deflections were well below acceptable limits given by AASHTO [2]. In terms of live load deflection-to-span ratios, Bridge 2 had the largest ratio (L/1525) and Bridge 3 had the smallest ratio (L/2213). AASHTO recommends that the live load deflection be limited to L/800.

Due to weather and traffic conditions, accurate concrete strain measurements could only be taken on Bridge 2. The largest recorded concrete compressive strain was 110 microstrain. The corresponding calculated live load concrete stress was 3.05 kPa and the dead load stress was 4.62 kPa. As was the case for the reinforcement stress and live load deflection, the total concrete stress, 7.67 kPa was well below the specified maximum, 13.8 kPa.

Load fractions, a measure of transverse load distribution, were calculated for each bridge from the midspan deflection data. The load fraction for a panel represents the fraction of a wheel line carried by that particular panel. In most cases, the two panels on which the truck was tracking on had the greatest load fractions of all the panels. Hereafter, the greatest load fraction for each bridge will be simply referred to as the load fraction for that bridge. All load fractions were calculated when the tandem axles were longitudinally centered at midspan.

The load fractions varied greatly from bridge to bridge. This was due to variations in the connections between adjacent panels. Inspection of all four bridges revealed that the concrete-filled pipes were not in place on Bridge 2 and grout was not packed in the keyways on Bridge 4. The keyways on Bridge 4 contained only gravel from the gravel wearing surface.

Bridge 3, with its fully grouted keyways, had the lowest load fraction for all PCBs tested. Its load fraction, 0.42, was also well below the design one lane load fraction, 0.58, calculated in accordance to AASHTO. The load fraction for Bridge 1 was 0.49 and was also below the design one lane load fraction, 0.57. These test results verify the effectiveness of the shear connection when installed properly.

When the shear connectors were not properly installed, the load fractions were greater than the design values. For Bridge 4, this difference was only marginal. The gravel in the shear keyways apparently aided in the transfer of shear to some degree. A larger difference occurred in Bridge 2 where the load fraction was 0.68, which was considerably greater than the design load fraction of 0.56. This is an important deficiency since a panel is supporting 21% more load than what it was designed to resist.

The effectiveness of these shear connectors is graphically show in Figure 5. This plot shows midspan deflection for Bridge 3 and 4 when the center lane was loaded. As one can see, the grout in the keyways of Bridge 3 prevented differential displacement across the panel joints. When the keyways were not grouted, significant joint slip occurred. Also, rigid joints transfer more load to neighboring panels and thus the load fractions for Bridge 3 were less than the load fractions for Bridge 4.

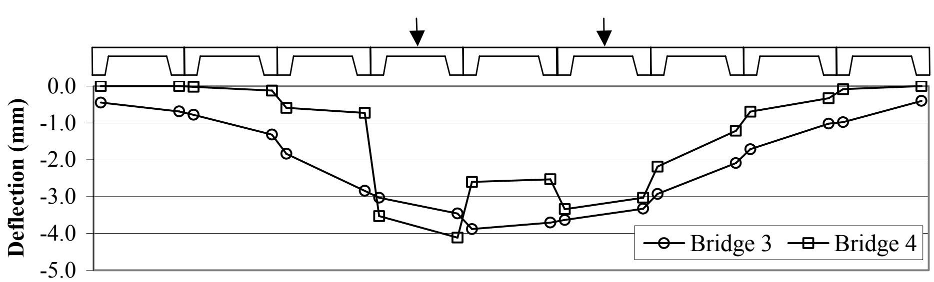

FIGURE 5. Transverse Centerline Deflections: Center Lane Loading

LABORATORY TESTING

A total of twelve deteriorated PCB panels from three different bridge replacement projects throughout Iowa were tested for ultimate flexural strength. The panels ranged in length from 7.6m to 11.0m and varied in the amount of deterioration. Some had relatively minor spalling and corrosion of the reinforcement while on others the majority of the primary reinforcement was exposed and heavily corroded. This variation was very useful since the effects of the deterioration could be seen. Additional information gained from the laboratory testing included panel strength, failure mode, stiffness, and strength of the concrete and reinforcement.

TESTING METHOD

Instrumentation used in the laboratory tests was similar to the instrumentation used in the field. Strain gages were bonded to the primary reinforcement and concrete deck at midspan. Transducers were used to measure vertical deflection at midspan and also at the quarterpoints. Each panel was loaded in a four point bending arrangement with 1.8m separating the middle load points. Load was applied by hydraulic actuators and was gradually increased until a failure occurred. During each load test, strain, deflection and load magnitude data were recorded by the DAS at predetermined levels of applied load. Following the failure, concrete cores and lengths of reinforcement were removed from undamaged portions of the panels and tested to determine the yield strength and the compressive strength of the reinforcement and concrete, respectively.

TEST RESULTS

The PCB panels performed well given their deteriorated state. Experimental ultimate strengths were found to generally exceed the theoretical ultimate strengths. Two factors contributed to this performance. First, large hooks on the ends of the bottom pair of bundled reinforcing bars effectively eliminated the need for development bond throughout the span. Secondly, the yield strength of the reinforcement was found to be considerably greater than the specified yield strength. Likewise, the concrete strength was found to also exceed its specified strength. The failure mode of all panels was a compression failure of the concrete deck preceded by excessive deflection.

Experimental and theoretical ultimate strengths for each panel are presented in Table 2; names indicate the county from where various panels were obtained. Cedar 1-3 were the shortest panels tested and therefore theoretically had the lowest ultimate strength. However, due to high concrete strength and reinforcement that experienced considerable strain hardening, the Cedar panels actually had the highest ultimate strength of the panels tested. Cedar 4 was the only panel tested with an attached curb. This curb increased the experimental strength of the panel by approximately 31% over the other Cedar panels.

As shown in Table 2, the experimental strength of the PCB panels exceeded their theoretical strength in all but one case: Butler 3. Butler 3 was by far the most deteriorated panel. In addition to heavy spalling and corrosion of the primary reinforcement, approximately 50% of deck surface had delaminated and spalled. The uncommon form of deterioration weakened the panel. With less deck available to resist compressive stresses, the remaining deck became overstressed at a lower load.

TABLE 2. Experimental and Theoretical Ultimate Strengths for Laboratory Tested Panels

Panel	Ultimate Strength (kN*m) - Experimental/Theoretical			
	1	2	3	4
Cedar	632/285	662/285	636/285	842/476
Butler	473/362	494/362	334/362	441/362
Black Hawk	521[1]/405	488[1]/405	583/405	549/405

[1]Failure not reached due to loading system limitations; value is midspan moment at maximum applied load.

Although the extent of the deterioration varied within a group of like panels, the experimental ultimate strengths for the various groups were relatively close. For Cedar 1-3, ultimate strength varied by only 30 kN*m and ultimate strength of the Butler panels varied by only 53 kN*m when the heavily damaged Butler 3 is excluded. Similar variation occurred for the two failed Black Hawk panels. This is an indication that the ultimate strength of these panels is only affected by extreme deterioration.

Midspan moment verses midspan deflection plots for the Black Hawk PCB panels are presented in Figure 6. As one can see, well defined regions of elastic and plastic behavior existed. This behavior was typical for all PCB panels tested. Elastic behavior occurred up to the point when the stress in the primary reinforcing exceeded its yield stress. The panels then deflected excessively while supporting only a small increase in load. This behavior continued until a compression failure occurred in the deck. Also shown in Figure 6 is the midspan moment induced by AASHTO HS20 loading. The magnitude of this loading is well within the elastic range and is considerably less than the ultimate strength of these panels.

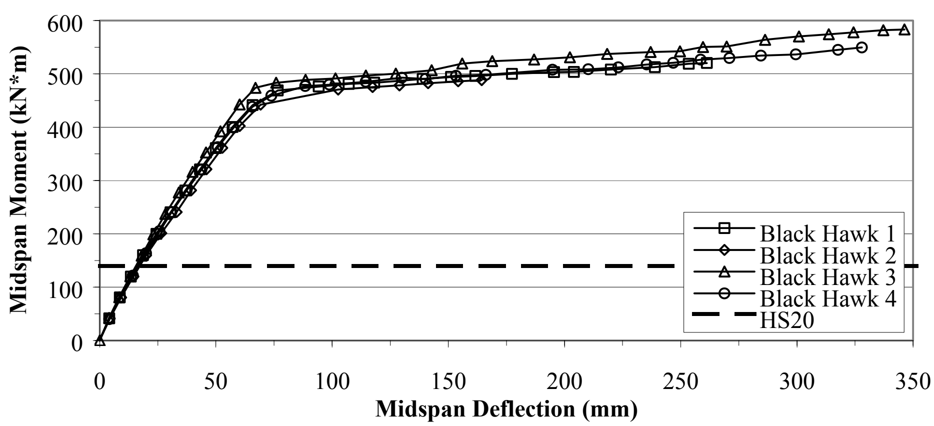

FIGURE 6. Moment Verses Deflection Plots For Black Hawk Beams

SUMMARY AND CONCLUSION

The most common form of deterioration found on PCBs was corrosion of the primary reinforcement and spalling concrete cover for this reinforcement. Through laboratory and field tests, it was determined that this deterioration had minimal effect on the performance of these bridges. Hooks on the ends of some of the primary reinforcement reduced the need for bond along the span. Another less common form of deterioration, deck delamination, did however cause a decrease in the ultimate strength of the panels.

The shear connection between panels was found to significantly affect the performance of the PCBs. When shear connectors were not properly installed, transverse load distribution was less than required by AASHTO.

ACKNOWLEDGEMENTS

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation.

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Quantitative Guidelines for Use of Thin Maintenance Surfaces

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ABSTRACT

Thin maintenance surfaces (TMS) extend the service life of bituminous and asphalt cement concrete roads—a task that has challenged road and highway agencies for years. Many of these agencies are aware of TMS as a maintenance treatment; however, selection of the proper TMS to use has been difficult. Guidelines for TMS selection were needed to improve the success of TMS.

The first phase of this research project, funded by the Iowa Department of Transportation and completed in April 1999, developed qualitative selection guidelines. For example, "Slurry seal and micro-surfacing are not recommended for badly cracked pavements; however, those treatments can be used to address a small amount of light cracking." However, the definitions of "badly cracked" and "light cracking" can vary from one person to another.

Therefore, quantitative standards for the selection of TMS were needed. The second phase of this research project refined the qualitative guidelines and developed quantitative guidelines for TMS selection. These new guidelines use the pavement condition index (PCI) rating developed by the U.S. Army Corps of Engineers. To avoid confusion with another index used in Iowa that is also referred to as the PCI, the index is called the surface condition index (SCI) herein.

The allowable distress is chosen by considering an appropriate SCI value for given treatments, traffic levels, and distresses. Users are expected to use judgment and to interpolate or to extrapolate to select a TMS for a particular traffic count. Transportation maintenance managers may find these guidelines useful for pavement management systems.

Key words: pavement management—thin maintenance surfaces

INTRODUCTION

In recent years there has been a renewal of interest in preventative maintenance techniques designed to extend pavement life of and to ensure low life-cycle costs for our nation's road infrastructure network. Thin maintenance surfaces (TMS) can be an important part of a preventative maintenance program for asphaltic concrete or bituminous roads. The need to demonstrate the use of TMS in Iowa and to develop guidelines for TMS use that are specific to Iowa spawned this research project. The Iowa Department of Transportation (Iowa DOT) and the Iowa Highway Research Board sponsored the project in two phases.

Phase One of the TMS research project included the following:

1. a survey of local transportation officials to determine current practices in Iowa
2. construction and monitoring of test sections (U.S. 151 and U.S. 30 in 1997 and U.S. 69 in 1998)
3. development of interim qualitative guidelines to help transportation officials select TMS

Before developing an interim set of guidelines, the researchers reviewed the literature, examined the results of the survey of local transportation officials, reviewed test section performance, and held discussions with the research advisory committee. Transportation decision makers are guided to select thin maintenance surfaces using a step-by-step process, beginning with an assessment of the condition of the road network. The second step is to identify technically feasible treatments by using a table based on the pavement surface condition and traffic load of the candidate road. The remaining steps result in a choice between technically feasible alternatives by considering past practices, cost, durability, user preferences, neighbor preferences, and other factors that are difficult to quantify.

The interim guidelines improved upon the scattered information that previously existed. However, because the guidelines required further improvement by providing better-defined decision points and guidance on when to use various types of aggregates and binders, a second phase of research was proposed to the Iowa Highway Research Board and subsequently approved.

Phase Two of the TMS research project included the following objectives:

1. continued monitoring for previously placed test sections
2. construction and monitoring of additional test sections (U.S. 218 in 1999)
3. evaluation of design processes for seal coats and recommendation of one for statewide use
4. further investigation of TMS aggregates
5. investigation between TMS and winter maintenance activities
6. refinement of the guidelines for TMS developed in Phase One

Objective 6 of Phase Two produced a set of quantitative guidelines. The allowable quantity of each type of distress was selected by considering an appropriate surface condition index (SCI) value for given treatments, traffic levels, and distresses. After selection of the SCI level, a permissible amount of distress was back-calculated. Three levels of traffic were considered: 5,000; 2,000; and 200 AADT.

Users are expected to exercise their judgment and to interpolate or to extrapolate to investigate treatment selection for a particular traffic count. In general, treatments that are the most appropriate for particular types of distress will be recommended for pavements with relatively low SCI values (indicating larger amounts of distress). Conversely, treatments that are least appropriate for a particular type of distress will not be recommended unless the pavement has a relatively high SCI value (indicating lesser amounts of distress). The ultimate product of Phase Two was a set of quantitative guidelines for TMS selection.

This paper will concentrate on objective 6 of Phase Two, the development of quantitative guidelines for the selection of TMS. However, the qualitative methods developed during Phase One of the research project are described to provide background.

METHODOLOGY

The research methodology was developed in consultation with a research advisory committee. Four sets of TMS test sections were constructed and monitored over a three-year period. The surface condition index for each test section was calculated before construction. The SCI was calculated by observing the cracks in each test section and performing calculations as described in greater detail herein. After construction, observations were made to calculate the SCI at regular intervals.

The SCI results for each test section were plotted versus time. Several treatments were recorded on the same graph so as to permit visual comparisons of the performance of the various test sections. Since the test sections were placed adjacent to each other on the same route, the traffic counts and vehicle loads essentially are constant throughout the test sections. Therefore, the only difference between one test section and another would be the initial SCI and any potential differences in the condition of the subgrade below the pavement. After comparing this performance data to the amount of cracking before application of the TMS and the traffic levels, the researchers developed guidelines based upon the literature review and experience gained from test section observation.

RESULTS

Phase One Interim Qualitative Guidelines *(1)*

A fundamental knowledge of the interim qualitative guidelines resulting from Phase One is essential to understanding the development of the quantitative guidelines resulting from Phase Two. An example of a qualitative guideline follows: “Slurry seal and micro-surfacing are not recommended for badly cracked pavements; however, those treatments can be used to address a small amount of light cracking.”

The Phase One interim qualitative guidelines provide a five-step TMS decision procedure:

Step 1. Collect Information on Candidate Roads

The transportation decision maker conducts a distress survey to assess the magnitude and the type of distress that the road is suffering in order to supply data for SCI calculations.

Step 2. Identify Feasible Treatments

Table 1 makes recommendations for the use of seal coats, slurry seal, and micro-surfacing *(2)*. The Phase One report provides additional guidance for selecting treatments for roads where rutting is the primary distress *(3)*. It should be noted that filling will serve as only a temporary remedy for those ruts that are caused by instability of the asphalt cement concrete or subgrade.

TABLE 1. Thin Maintenance Surfaces for Various Traffic Volumes and Distress Types

	Seal Coat	Slurry Seal	Micro-surfacing
Traffic volume:			
AADT < 2,000	Recommended	Recommended	Recommended
2,000 > AADT < 5,000	Marginal*	Marginal*	Recommended
AADT > 5,000	Not recommended	Not recommended	Recommended
Bleeding	Recommended	Recommended	Recommended
Rutting	Not recommended	Recommended	Recommended
Raveling	Recommended	Recommended	Recommended
Few tight cracks	Recommended	Recommended	Recommended
Extensive cracking	Recommended	Not recommended	Not recommended
Low friction	May improve	May improve	May improve**
Snowplow damage	Most susceptible	Moderately susceptible	Least susceptible

* There is a greater likelihood of success when used in lower-speed traffic.
** Micro-surfacing reportedly retains high friction for a longer period of time.

Step 3. Consider Other Factors

The Phase One report provides a table (see Table 2) of other factors that should be considered before making a final selection regarding seal coats, slurry seals, and micro-surfacing *(2)*. If previous investigation indicates multiple treatments are feasible, this table will indicate the preferred method.

TABLE 2. Other Factors Impacting Thin Maintenance Surface Decisions

	Seal Coat	Slurry Seal	Micro-surfacing
Past practices	Most officials prefer not to change successful past practice unless there is 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.		
Funding and cost	Least expensive option → less funding is required.	More expensive than seal coat and less expensive than micro-surfacing.	Most expensive option → more funding is required.
Durability	Dependent of aggregate type, binder type, and application technique.	Less durable than micro-surfacing.	More durable than slurry seal.
Turning and stopping traffic	Can be flushed by turning and stopping traffic.	Can hold turning and stopping traffic.	Best wear in turning and stopping traffic.
Dust and fly rock	Considerable dust possible during construction.*	Little dust possible during construction.	
Curing time**	Road can be opened after rolling is completed and speed should be limited to about 20 mph for 2 hours.	Road can be opened after 2 hours in warm weather and 6–12 hours in cold weather.	Road can be opened after 1 hour.
Noise and surface texture	Fairly noisy surface, open surface texture, and many loose rocks immediately after construction.	Less noise and dense surface texture (close to hot-mix surface).	
Availability of contractors	13 contractors in Iowa.	3 contractors in Iowa.	2 contractors in Iowa.
Use of local aggregates	Maximum flexibility: - Can use somewhat dusty aggregates with cutback binder. - Can use emulsion or cutbacks. - Rock chips, pea gravel, and sand may be used.	Less flexibility.	Least flexible. The binder is highly reactive (break time is affected by clay content).

* Dust is mitigated by using washed, hard, or pre-coated aggregate.
** Federal Highway Administration.

Step 4. Consider Timing

The construction of TMS must be properly timed. Most experts suggest applying TMS to a road seven to ten years after initial construction. Geoffroy *(4)* surveyed 60 transportation agencies regarding TMS life expectancy and reported that the expected lifespan of the treatment is five to ten years.

Transportation officials with successful TMS programs usually apply the first surface treatment when fine aggregate begins to ravel from the road surface. Raveling often occurs seven to twelve years after construction. Roads consisting of several layers of seal coat may require maintenance more often because less pavement structure is available to support loads.

Step 5. Consider Cost

As costs will vary from one area to another, users must research this locally.

Phase Two Quantitative Guidelines *(5)*

While the Phase One qualitative guidelines were an improvement, quantitative guidelines were desirable to limit the variation in application between users. The main objective of this phase of the project was to develop a framework for guidelines that are more quantitative. The framework is based on the surface condition index (pavement condition index) as described by Shahin *(6)* and the primary author's experience accumulated while executing both phases of this research project. The resulting guidelines are more quantitative than the ones developed in Phase One, but could be improved with further research.

The allowable quantity of each type of distress was selected by considering an appropriate SCI value for given treatments, traffic levels, and distresses. After the SCI level was chosen, a permissible amount of distress was back-calculated. Three levels of traffic were considered:

- **5,000 AADT:** Typical of a high-volume, two-lane, rural primary highway that may be a candidate for conversion into a four-lane highway
- **2,000 AADT:** Transition point from a high-volume primary rural highway to a low-volume primary rural highway
- **200 AADT:** Transition point between paved and graveled rural roads

Users will be expected to exercise their judgment and interpolate or extrapolate to investigate treatment selection for a particular traffic count as they follow the guidelines.

The guideline for cracks serves as an example (see Table 3).

TABLE 3. SCI Values for Maintenance Activity Types

Maintenance Activity	SCI Value	Deduct Value
Routine	60–95	5–40
Preventive	50–75	25–50
Rehabilitation	25–60	40–75
Rebuilding	0–40	60–100

TMS normally will be used for preventive maintenance, so the expectation is that the SCI value will range from 50 to 75 at the time of treatment.

Guidelines based upon cracking and traffic are described in Table 4 for four surface treatments and various crack lengths on a 24-foot-wide by 100-foot-long section of roadway. Crack lengths range from 300 to 1,500 feet in increments of 150 feet, except for a final 300-foot increment. SCI and deduct values were calculated as described by Shahin *(6)*, with the assumption that light longitudinal and transverse (L&T) cracking was the only distress present. Note that Shahin's method does not provide SCI calculations for L&T crack lengths that exceed 720 feet (30 percent distress). It may be that distress densities that exceed this amount are considered block cracking or some other type of distress in this method, but the author did not offer any further explanation. All cracks (except alligator cracks) are converted into an equivalent length of light cracking. Table 4 suggests adjustments to measured crack lengths for repairs and utility patches.

TABLE 4. Thin Maintenance Surface Guidelines Based on Amount of Cracking and Annual Average Daily Traffic

Feet of Cracking*	300	450	600	750	900	1,050	1,200	1,500
SCI basis**	80	78	73	71	***	***	***	***
Deduct basis**	20	22	27	29	***	***	***	***
					AADT			
Micro/slurry	5,000		2,000		200			
Seal coat (1/4 inches)		5,000		2,000		200		
Seal coat (1/2 inches)			5,000		2,000		200	
Double seal coat				5,000		2,000		200

Note: Based on 100 feet of road 24 feet wide.
* Medium-intensity cracks require joint sealing or slurry strip repair before surface treatment is placed. High-intensity cracks require patching before treatment is placed. Therefore, one foot of high-intensity crack equals two feet of light-intensity crack. Consider utility cuts and patches as low-intensity cracks around the perimeter of the repairs.
** Based on light L&T cracking.
*** SCI basis and deduct are not given for more than 750 feet of light L&T crack.

The lower bound on the amount of cracking distress that would be addressed by thin maintenance surfaces was established by the consideration of the use of slurry seal or micro-surfacing. These techniques do not address cracking as well as other techniques, so the required SCI is set somewhat above the usual preventive range at 80 for high-volume primary roads (AADT = 5,000). If light L&T cracking is the only distress, the maximum allowable percent of distress is 12.5 percent for a deduct value of 20. For a 100-foot section of road 24 feet wide (2,400 ft^2), the maximum allowable feet of length of cracking is 12.5 percent of 2,400 ft^2, or 300 feet (see Figure 1). A road with four transverse joints in 100 feet, a completely cracked longitudinal joint at the centerline of road, and a partial (50 percent) crack in each mid-lane would yield slightly less than 300 feet of crack. In the first author's experience, this represents a reasonable amount of cracking to be addressed by micro-surfacing on a high-volume road.

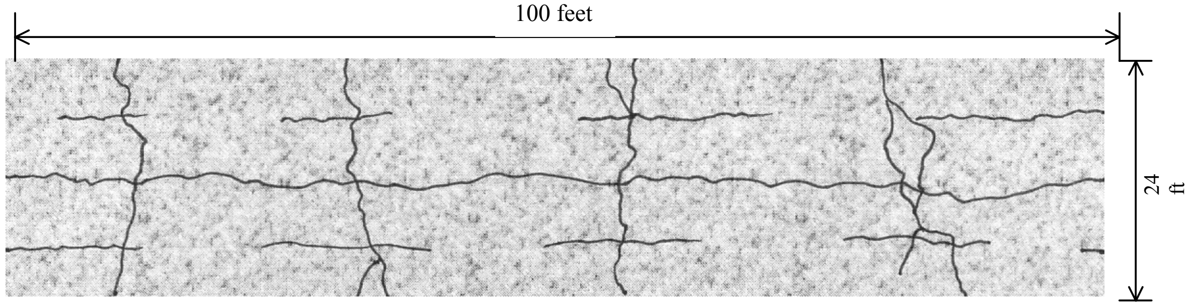
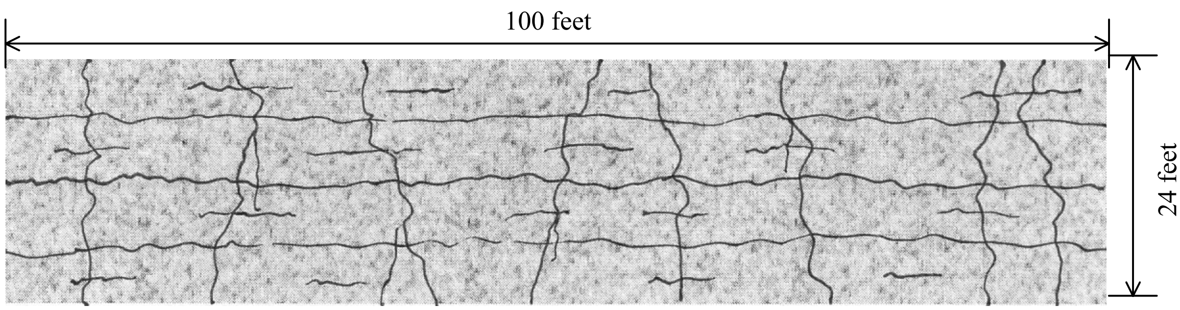


FIGURE 1. 2,400 ft^2 Section of Roadway with about 300 Feet of Cracking

If length of crack doubles, micro-surfacing would only be recommended if traffic is 2,000 or less AADT (see Table 4). This yields a SCI value of 73, inside the preventive range. Six hundred feet of crack could occur in a 100-foot section of 24-foot-wide road if there are eight transverse cracks, the centerline and both mid-lanes were cracked, and 25 percent of the wheel paths are cracked (see Figure 2). The start of wheel path cracks, as illustrated in Figure 2, may suggest incipient fatigue failure; however, at 2,000 AADT it is possible that the pavement may retain sufficient structural strength to last the life of the maintenance treatment (about seven years). However, TMS will do little to mitigate fatigue failure. Note that for 600 feet of light-intensity cracks on a higher volume road (5,000 AADT), 1/2-inch seal coat would be suggested, if the agency had a policy of seal coating such high-volume roads.



1 longitudinal joint	100 feet
2 mid-lane	200 feet
25% of 4 wheel paths	100 feet
8 foot × 24 foot transverse	192 feet
Miscellaneous	8 feet
Total	600 feet

FIGURE 2. 2,400 ft^2 Section of Roadway with about 600 Feet of Cracking (with calculations)

To establish an upper bound for the amount of cracking distress that could be addressed with TMS, the researchers considered a 3-foot by 3-foot crack pattern similar to block cracking, and a double seal coat was identified as a satisfactory treatment for roads with 200 or less AADT. This selection was made on the basis of anecdotal evidence that the first author collected where a road with a similar crack pattern was successfully treated in this way. It is important to note that the cracks cannot "work" up and down under load, and the road may not meet the usual standards for ride and appearance. However, the treatment might preserve a road with very light traffic.

Although alligator cracking frequently indicates a fatigue failure, guidelines were developed to address this condition with TMS. As stated previously, TMS do very little to address fatigue problems; however, TMS may reduce the amount of moisture entering the base and subgrade through the pavement, thereby stiffening the subgrade and reducing pavement stress, which would provide modest benefit. Also, the principal investigator has anecdotal evidence that low-volume roads, especially urban residential streets, can be candidates for TMS, if they have light alligator cracking due to small deflection fatigue. For a low-volume road, the thin maintenance surface may be sufficient "glue" to hold the alligator blocks in place, to reduce crack width, and to prevent spalling for a period of time, thereby extending the life of the pavement.

Guidelines for using TMS to address light intensity alligator cracking distress are given in Table 5. Zero percent distress is allowed for medium- and heavy-intensity cracking for roads with traffic volumes of 5,000 AADT. The SCI requirement for micro-surfacing and 2,000 AADT was set at 75, which is the upper limit of the usual range for preventive maintenance. Therefore, the maximum allowable alligator cracked area would be 5 percent. This level was chosen because micro-surfacing/slurry seal is not a preferred treatment for addressing cracking distress. The required SCI for 2,000 AADT and 1/4-inch seal coat, 1/2-inch seal coat, and double seal coat are 70, 65, and 60, respectively, based on the primary author's judgment. For each treatment, compared to the requirement for 2,000 AADT, the SCI requirement is 10 points less for 200 AADT.

TABLE 5. Thin Maintenance Surface Guidelines Based on Amount of Alligator Cracking and Annual Average Daily Traffic

	Micro/Slurry			**Seal Coat (1/4 inches)**		
AADT	5,000	2,000	200	5,000	2,000	200
SCI basis	*	75	65	*	70	60
Deduct basis	*	25	35	*	40	50
Light cracking**	*	5%	12%	*	8%	1%
	Seal Coat (1/2 inches)			**Double Seal Coat**		
AADT	5,000	2,000	200	5,000	2,000	200
SCI basis	*	65	55	*	60	50
Deduct basis	*	35	55	*	40	50
Light cracking**	*	12%	22%	*	18%	40%

Note: Based on 100 feet of road 24 feet wide.
* TMS are not recommended for alligator cracking on roadways with 5,000 or greater AADT.
** Applies to alligator cracking caused by fatigue due to advanced age combined with moderate deflection on firm subgrade. Do not use TMS for fatigue cause by severe deflections on soft subgrade.
Note: TMS are not recommended for medium or heavy alligator cracking.

Guidelines were refined to address bleeding as well (see Table 6). Independent guidelines for slurry seal and micro-surfacing were generated. The minimum SCI requirement for 5,000 AADT and micro-surfacing was set at 80; while for the same traffic and seal coat the SCI was set at 60. As traffic decreases, 10-point increments are allowed between each category. The SCI requirement was set high for micro-surfacing and slurry seal because it is difficult to decrease the quantity of binder in the mix design to compensate for bleeding from the substrate. For seal coat, an SCI requirement of 60 was selected because the amount of binder can be adjusted downward to compensate for bleeding. The SCI of 60 is near the middle of the preventive maintenance range (see Table 3). If a seal coat is used, the chances of success can be increased by using one-size aggregate that will allow excess void space to accommodate additional oil from the bleeding surface.

TABLE 6. Thin Maintenance Surface Guidelines Based on Amount of Bleeding and Annual Average Daily Traffic (based on 100 feet of road 24 feet wide)

	Micro/Slurry			Seal Coat*		
AADT	5,000	2,000	200	5,000	2,000	200
SCI basis	80	70	60	60	50	40
Deduct basis	20	30	40	40	50	60
Light bleeding	100%	100%	100%	100%	100%	100%
Medium bleeding	23%	55%	100%	100%**	100%**	100%**
Heavy bleeding	8%	15%	25%	25%**	40%**	60%**

* Consider using clean, one-size cover aggregate to provide more void space for excess oil and reducing binder application rate (especially for medium to heavy bleeding).
** Consider using 1/2-inch cover aggregate (more void space for excess oil).

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

This research project has demonstrated that it is possible to develop quantitative guidelines for use in the selection of TMS using the concept of the surface condition index. It should be noted that the results of this study are empirical in nature. By back-calculating an acceptable level of distress after selecting an SCI, the user will be guided to a specific TMS.

The guidelines were developed based on observations from four sets of test sections placed over three years, as well as a literature review, anecdotal evidence from conversations with government and industry employees, and observations by the authors.

Recommendations

Transportation decision makers should try both the qualitative and the quantitative guidelines. For many users, the qualitative guidelines may be adequate. These users are expected to use their experience and judgment when applying the guidelines. For others, the quantitative guidelines may be more appropriate. These users may be required to use interpolation or extrapolation at times. This system provides a definitive TMS selection such that all users should come to a standardized conclusion.

The quantitative guidelines may lend themselves to integration into a computerized pavement management system. Users may wish to compare the results from these guidelines to other systems that may already be in use. The guidelines should be further refined as more experience is collected.

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The research project advisory committee included the following members:

- Iowa DOT Office of Construction: Dave Jensen, P.E., and later Jeff Schmitt, P.E.
- Iowa DOT Office of Maintenance: John Selmer, P.E., and Francis Todey, P.E.
- Iowa DOT Office of Materials: John Heggen, P.E., and later Mike Heitzman, P.E.
- Carroll County: David Paulson, P.E.
- Kossuth County: Richard Scheik, P.E., L.S.
- City of Carroll: Randy Krauel, P.E.
- City of Newton: Neil Guess, P.E.
- Fort Dodge Asphalt: William Dunshee
- Koch Materials, Inc.: Bill Ballou (Dan Staebell, alternate)
- Sta-Bilt Construction Co.: Richard Burchett

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Pilot Study on Laser Scanning Technology for Transportation Projects

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ABSTRACT

This paper describes the results from a pilot study to investigate the use of laser scanning for the Iowa Department of Transportation in delivering projects in a safer and more efficient manner. Laser scanning is a terrestrial laser-imaging system that quickly creates a highly accurate three-dimensional image of an object for use in standard computer-aided design software packages. Included in this paper is a description of the technology, discussion of the pilot tests, lessons learned, and results. It appears that laser scanning technology may be an alternative tool for design and construction of transportation projects. Based on results from the study, laser scanning can be used cost effectively for preliminary surveys. It also appears that this technique can be used quite effectively for construction measurement in a safer and quicker fashion compared to conventional approaches. Laser scanning technology would be quite beneficial for determining volume calculations.

Key words: computer-aided design—data collection—laser scanning—preliminary survey—three-dimensional image

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INTRODUCTION

As transportation projects become more complex to design and build, it is important to take advantage of appropriate innovative technologies for reducing project cycle time. Laser scanning is one such technology that has potential benefits over standard surveying techniques such as aerial photogrammetry for providing accurate measurements. Laser scanning is a terrestrial laser-imaging system that quickly creates a highly accurate three-dimensional (3D) image of an object for use in standard computer-aided design (CAD) software packages. The laser's visible green beam is moved across a target in a raster scan. The horizontal and vertical angles of the beam are measured for each point, as well as the time of flight of the pulses. Figure 1 shows the Cyrax 2500 system used for the pilot study *(1).* Once an object is encountered, the laser is reflected back to the unit with the time of flight, which generates a measurement of distance. These measurements produce an impact location, which in return displays a cloud of points.

Measurements taken from the "cloud" can be used to do interference detection and constructability studies. Each point has embedded x, y, z data, so it can be directly loaded into a CAD program without any need of digitizing. Less than 6 mm (1/4") accuracy can be obtained using this technology (*2*).

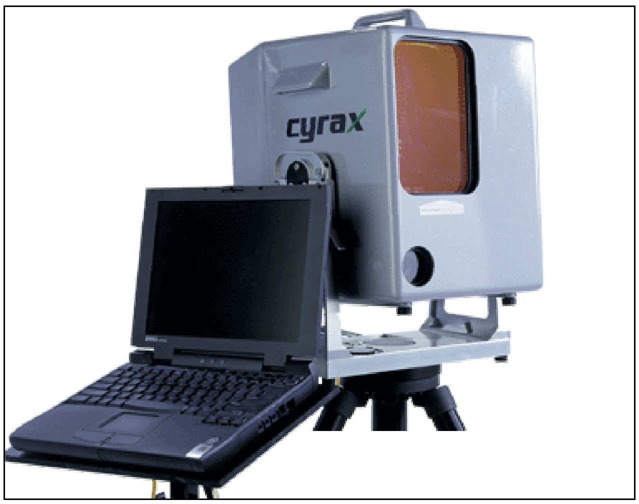

FIGURE 1. Cyrax 2500 Laser Scanning Unit

As an object is being scanned, each 3D measurement appears immediately as a graphical 3D point image on the laptop screen. Exporting scanned data into a CAD application, such as AutoCAD, MicroStation or 3D StudioMax, is also possible. This technology has been successfully demonstrated on numerous projects related to developing bridge as-builts, highway widening, power plant retrofits, refinery expansion projects, water utility construction project archives, rock face surveys, dam foundation surveys, cave scanning, rebar inspection, and visual effects for movies *(1)*. In several cases, field and office time were reduced using laser scanning compared to conventional methods.

PILOT STUDY OBJECTIVES

This study involves a pilot test to investigate the use of laser scanning to assist the Iowa Department of Transportation (Iowa DOT) in delivering projects in a safer and more efficient manner. The objectives are as follows:

1. Learn about how to use the laser scanner and software.
2. Select appropriate pilot projects to test the capabilities of this technology.
3. Determine the benefits and costs associated with using this technology compared to conventional approaches.
4. Provide recommendations regarding the future use of laser scanning for the Iowa DOT.

PROJECTS INVOLVED

In total, there were six test areas involved in this pilot study: (1) an intersection including a railroad bridge, (2) a section of highway including a pair of bridges, (3) new concrete pavement, (4) bridge beams on an unfinished bridge structure, (5) a stockpile, and (6) a borrow pit. These projects were selected because they were of particular interest to the Iowa DOT as areas where greater efficiencies could be attained. Table 1 summarizes the purpose for each pilot project. More specific information can be found from the study report *(3)*. There were some difficulties registering all of the scanworlds associated with the borrow pit pilot test. Consequently, it was decided to discard this pilot project and rely on volume-measuring capabilities using only the stockpile pilot project.

TABLE 1. Purpose of Pilot Projects

Pilot Project	Purpose
Intersection and railroad bridge	Learn about the Cyrax 2500 scanner and Cyclone software (training exercise).
Section of highway and pair of bridges	Determine surface elevation of highway and compare to aerial photogrammetry. Also, determine the level of bridge detail available using laser scanner.
New concrete pavement	Determine smoothness of freshly paved concrete.
Bridge beams on unfinished bridge	Assess camber on bridge beams for determining optimal loading requirements.
Stockpile	Determine volume of stockpile and compare to traditional methods.
Borrow pit	Determine volume of borrow pit and compare to traditional methods.

FIELD OPERATIONS AND LESSONS LEARNED

Several lessons were learned during the two week training and field scanning process as they related to the set-up and operation of the equipment.

Field Operation

Field operations involved two major tasks: (1) set up survey control points and targets (i.e., globe targets mounted on tripods) and (2) scan the desired objects and acquire targets. The research team consisted of two separate crews, surveying crew and scanning crew, for these two activities. The scanning operation involved several activities related to properly using the Cyclone software such as creating a database, operating scan control window, and target acquisition.

The survey crew consisted of five surveyors and one coordinator. The survey crew used traditional methods to set up targets and tie them into the Iowa state plane coordinate system. Thus, different scans could be registered and matched to each other with a high degree of accuracy. The surveying time was not specifically tracked but should be similar to the scanning time. This is because the surveyors worked the same hours as the rest of the team.

The scanning crew consisted of two operators. Table 2 shows the basic information related to the number of scans and duration of each scan. Scanning time defines the difference between start and end times of scanning. Start time is when the scanner begins to take the point cloud image while end time is when the computer is disconnected from the scanner. Scanning times varied per scan primarily because scans were performed using different resolutions.

TABLE 2. Field Scanning Information for Pilot Projects

Pilot Project	No. of Scans	Total Scanning Time (hrs.)	Average Scanning Time (min.)
Intersection and railroad bridge	—	—	—
Section of highway and pair of bridges	30	14	28
New concrete pavement	3	1.6	32
Bridge beams on unfinished bridge	5	2.9	34.8
Stockpile	3	1.5	30
Borrow pit	17	4.4	15.5

Lessons Learned

Since the research team had adequate control on each target, it was not necessary to have common targets in every scan. Overlapping targets are necessary when there is no knowledge of the x, y, z coordinates for each location. To obtain proper registration, at least three targets need to be common in each scan when control is not established on the targets.

Target acquisition is a critical issue for scanning and registration later. During the scanning process a few different types of mistakes were made that created additional work in the field and office. Some of the more common examples are listed below:

1. Targets were completely missed during the initial scan, reducing the number of targets in the scanworld and causing difficulty during the registration process.
2. Failure to scan the correct targets. This is typically detected during the acquisition process and requires the operator to reacquire the correct target.
3. Paired targets were mislabeled (switched). This could be corrected during the registration process.
4. Targets with labels that do not exist in the control files. This could be corrected during the registration process by including the correct coordinates.
5. Targets without labels. This lead to difficulties during the registration process because it is hard to tell which target is being used.
6. Targets with double labels. This is happens because the same two targets were acquired during the acquisition process using different labels. This problem is corrected during the registration process.

It was found that vibrations or scanner movement during the scanning operation makes it very difficult to align images during the registration process. This is because the scanned image becomes distorted once the scanner is moved from its initial position. Thus, it is important that the laser scanner be mounted on a stationary, nonvibrating surface.

DATA PROCESSING AND CONCERNS

The primary procedures involved with analyzing the scanned data from the field include importing coordinate data, registration, image fitting and editing, mesh editing, contouring, and using the virtual surveyor routine *(4)*. Not all of the projects required each of these steps as the requirements were dependent upon the desired outcome. Sometimes special steps were necessary in order to meet the unique requirements of the pilot tests. Details related to the image processing for each of the pilot projects (except for the intersection and railroad bridge training exercise) can be found in the research report *(3)*.

Registration Concern

The project of Highway and Pair of bridges was the first one that the research team did on its own after training. As a result, there were many mistakes related to correct target acquisition, which made the registration process more time consuming. A total of nine targets in eight scanworlds had target problems. It was found that checking and measuring target locations and distances between targets in the control space is an efficient and effective way to identify the problems once large errors appear in the registration window. The correction action, however, was performed in model space and then a new control space was created from model space. Because some mistakes were made with the first few scanworlds, extra work was required to minimize the registration errors. After registration and related cleanup work had been successfully finished, there were still some targets with errors slightly larger than the original tolerance of 0.009 meters. Most of targets with errors ranged from zero to 0.007 meters. Those errors could be caused by either a physical setting deficiency of targets or distortion of the laser beam but not by the targets labels. Because the greatest errors were in pairs of targets with distances greater than 50 meters, distortion is most likely the reason.

Fitting and Editing Concern

The process of removing the noise and modifying the registered scanworlds went smoothly for I-235 and primarily involved removal of superfluous data representing traffic on the roadway surface. Because the scanned images are 3D objects, different perspective views had to be checked in order to make sure the traffic noise was completely removed. Although the research team expended a significant amount of effort on mesh editing the point cloud file, this step was not really necessary since elevations could be measured directly using the virtual surveyor routine.

Vegetation removal on the stockpile was the most difficult part of the editing process. After numerous trials, a set of parameters was determined as a best solution to remove the brush and vegetation with minimal disruption to the stockpile. After applying the region growing routine, some leftover target tripods still required removal using a manual approach. This usually also deleted some of the stockpile but did not influence the final result because the density of the point cloud was sufficiently high. Figure 2 shows the point cloud after final point cloud cleaning.

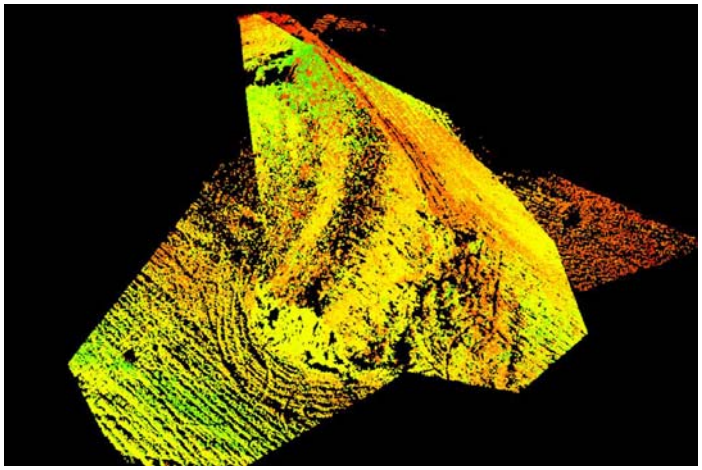

FIGURE 2. Cleaned Up Point Cloud Image of Stockpile

Special Concerns

A few new special features were applied to the Hardin County Bridge because the beams were not parallel to the reference plane axis, and establishing the true top of beam surface was challenging because of the protruding steel reinforcing loops present on the top surface. To be able to use the virtual surveyor routine along the beam, a new coordinate system was created by drawing a line on the beam, which was set as a new x-axis instead of the default system. Also, one end of that beam was set as the new origin. The new x-z plane was used as new reference plane to cut the beam into slices. By defining a proper thickness of each slice, the top boundary line can clearly be determined by the front view of the beam slice. After this step, the normal virtual surveying process can be applied.

To measure the volume of stockpile correctly, the mesh volume had to be measured taking into consideration the sloping ground below the stockpile. Since it was not possible to establish a curved reference plane that follows the upward sloping stockpile, it was necessary to create two separate meshes with one reference plane. The top mesh is based on the entire cleaned point cloud. The bottom mesh is based on the surrounding area of this point cloud. The desired volume of the stockpile is calculated by taking the volume difference between the top mesh relative to an arbitrary reference plane and bottom mesh relative to the same reference plane. The reference plane can be randomly chosen but must be below the top of the bottom mesh in order to simplify the calculation.

PILOT STUDY RESULTS

The results include information related to the technical results from the pilot tests, time expended to perform the pilot tests, and a cost comparison between aerial photogrammetry and laser scanning.

Technical Results

Elevation measurements were taken of the I-235 roadway centerline, lane edges, and shoulders using the Cyclone virtual surveyor. The measured results were exported into an ASCII format file. To determine the accuracy of those data points and to compare them with those from other surveying methods such as helicopter aerial photogrammetry, an ASCII file was converted into a MicroStation and GEOPAK file to create the plan views of I-235. While the difference between those methods can be clearly seen from the

MicroStation file, a detailed accurate comparison was also conducted. The average difference for measuring elevation at the lane edges between traditional surveying and Cyra laser scanning ranged from 0.001 meters to 0.009 meters, while the difference ranged from -0.006 meters to -0.023 meters between aerial photogrammetry and Cyra laser scanning. This comparison demonstrates that much more accurate measurements can be obtained from Cyra laser scanning technology than from the photogrammetry method.

The stockpile volume is calculated assuming that the reference plane is established at 300 meters. Using this assumption, the top and bottom mesh volumes are 2,176.849 and 1,669.78 cubic meters, respectively. Thus, the stockpile volume is 507.07 cubic meters. The volume of this stockpile was calculated using a traditional surveying approach and GEOPAK software and was found to be 512.96 cubic meters. It can be seen that the results of both surveying methods are fairly close to one another (1.2 percent difference, or 6 cubic meters).

Results show that it is possible to measure bridge beam camber using the virtual surveyor routine in the Cyclone software. However, it was not possible to adequately determine the smoothness of the pavement surface. The primary reason is because the laser scanner has an accuracy of two to three millimeters. Most smoothness irregularities will fall within or below the accuracy range of a laser scanner. Therefore, the application of this new technology, in its current state, is not sufficiently sensitive to monitor the smoothness of freshly paved concrete.

Time Requirements

Overall, a total of approximately 870 hours (15.1 hours per scan) were spent on this pilot study, including 403.1 hours for fieldwork, 153.5 hours for lab analysis, and 313 hours for training. Different groups of participants, including a training group, a scan crew, a survey crew, and lab analysts were involved in different phases of the learning process. Some people who attended the training course did not participate further with the project. Also, an assumption was made that the same field time was spent by the scan crew and the survey crew. All of these facts make the time tracking and analysis a complicated process.

Table 3 summarizes time spent on the entire project. In order to evaluate the project more accurately, the time spent by people who attended training but who were not involved in any other tasks was removed from the total hours, yielding the actual hours. Clearly, the learning time is more significant than may be expected.

In order to maximize production and efficiency, the size of the training and scan crew can be reduced to one scanning operator and one coordinator while the survey crew can be reduced to three surveyors and one coordinator (the same person as the scan coordinator) without diminishing work quantity or quality. Therefore, projected hours were calculated based on these crew sizes and are also listed in Table 3. The total hours are reduced to 477.5 from 805.6 (a 40 percent reduction). The field time, lab time, and learning time account for 55 percent, 17 percent, and 28 percent of the total hours, respectively. The total hours above can be converted into hours per scan. The actual hours per scan are 14 (7.0 in the field, 2.7 in the lab, and 4.3 for learning). Learning time is a one-time investment and will have less impact on total time as more projects are scanned and analyzed.

To analyze the project time more meaningfully, the time spent on each pilot project is discussed. For scanning equipment operations, the time per scan over four projects (excluding the borrow pit project) ranges from 3.5 to 4.0 hours with an average of 3.7 hours. For the lab analysis portion, the average time per scan is 2.5 hours for the total project and 0.8 hours for the final trial.

TABLE 3. Summary of Total Time Spent on Pilot Study

Type	Actual Time (hrs.)	Projected Time (hrs.)
Field time	**403.1**	**262.5**
Scanning operation	187.4	121
Transportation	114	75
Breaks	57	37.5
Setup	38	25
Support	6.7	4
Lab analysis time	**153.5**	**80**
Learning time	**249**	**135**
Training course	120	80
Reading and studying	40	20
Watching videos	30	15
Defining procedures	9	10
Discussion	20	10
Meetings	30	0
Total	**805.6**	**477.5**

Cost Comparison

According to the data from the Iowa DOT, helicopter aerial photogrammetry costs approximately $2.66 per linear foot. Based on the I-235 research, laser scanning costs $3.43 per foot. Although the laser scanning cost is approximately 30 percent higher than that for aerial photogrammetry, laser scanning offers advantages in terms of accuracy. Due to this characteristic, it may be possible to use laser scanning as alternative for the initial project planning and design phases. However, scanning would need to be carefully coordinated, as the scan makes no distinction between the differing surfaces involved. Aerial photogrammetry does offer some benefits here because features such as centerlines and shoulders can be visually identified. Laser scanning costs can be reduced if the scanner were to be mounted on platform vehicle, allowing both sides of the divided highway to be scanned at the same time. It is surmised that the costs would then be comparable to those found using helicopter aerial photogrammetry.

CONCLUSION AND RECOMMENDATIONS

Laser scanning appears to have its applications for transportation projects. Applications of greatest benefit using the strengths of this technology appear to be ones where there is a significant amount of detail that needs to be captured and/or applications where safety may be an issue such as providing accurate measurements on an active roadway. Laser scanning performed quite well on determining quantities of soil and rock. Laser scanning was also found to be particularly helpful in measuring bridge beam camber. This technique was able to determine the beam camber quite efficiently and accurately. It was also ascertained that the laser scanner is not suitable for measuring concrete pavement smoothness on newly paved concrete. It seems to take a significant effort to become proficient with this technology and then one needs to continue using it to maintain a level of sharpness. If there are sufficient opportunities to use this technology, then it is recommended that the user purchases the Cyclone software and purchase or rent the scanner. Initially, it may be prudent to rent the scanner. If there are very few occasions, then the user should use more traditional approaches to capturing these data or hire a consulting firm with this expertise to provide the laser scanning services.

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Synthesis of Best Practice for Increasing Protection and Visibility of Highway Maintenance Vehicles

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ABSTRACT

This paper presents current practices in enhancing visibility and protection of highway maintenance vehicles involved in moving operations such as snow removal, crack sealing, pothole patching, and shoulder operations. The most recent information for current moving operation practices throughout the country and the state of Iowa is discussed, better enabling transportation agencies to adequately assess the applicability and impact of each strategy to their best use and budgetary limitations.

A review of relevant literature, including the *Manual on Uniform Traffic Control Devices*, suggests that there are few concise current guidelines for moving work zones, resulting in a good deal of variability in states' practices. Furthermore, a sparsity of investigative studies of equipment and techniques used to enhance the safety of public and workers in moving maintenance activities is evident. This is in contrast to stationary work zone operations where researchers have significantly contributed to the field of knowledge throughout the years. A compilation of current moving work zone practices is believed to be beneficial in supplementing current literature resources and for identifying future research needs.

A national survey found that most states use amber warning lights on orange colored maintenance vehicles and almost all apply a form of retro-reflective material to enhance nighttime visibility. In general, snow removal vehicles use more warning lights and devices than standard maintenance equipment. All responding states indicated using shadow vehicles and/or truck mounted crash attenuators with routine maintenance moving operations. A supplemental Iowa county survey found that similar traffic control and warning devices are used on snow removal and standard maintenance vehicles. Mounted warning signs and rotating or strobe warning lights are common for routine operations while snow removal equipment utilizes retro-reflective tape, warning flags, rotating and strobe lights, and auxiliary headlamps.

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Key words: maintenance—*Manual on Uniform Traffic Control Devices*—moving operations——warning devices

INTRODUCTION

Despite the development and availability of many new accouterments such as crash attenuators, specialized warning lights, and retro-reflective materials, a large number of crashes are still attributed to inadequate visibility of maintenance vehicles and personnel in moving work zone operations. While conspicuity and subsequent safety have been greatly improved in recent years, still no precise summary of products and practices is available as reference for supervisors and workers in moving operations.

Current literature and available references provide few substantive guidelines for effective traffic control in moving work zones such as pavement marking application, snow removal activities, surface patching, crack filling, and shoulder operations. Consequently, considerable variability in practice exists among states. This paper will describe an effort to compile and summarize current practice for temporary traffic control measures used throughout the country to improve visibility and protection of maintenance workers and equipment in moving operations.

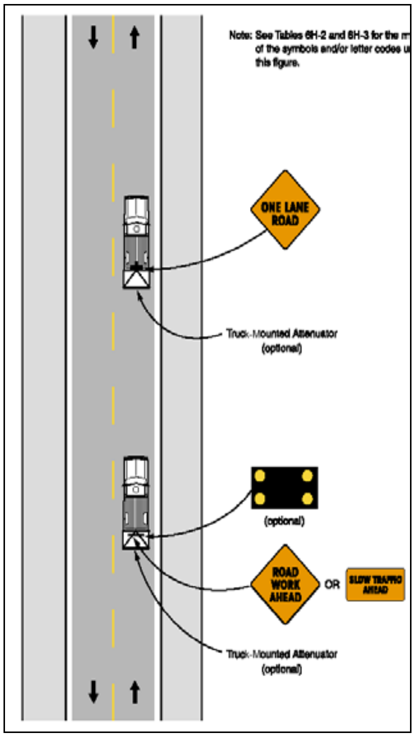
In addition to a review of current literature, both a national and Iowa county survey were conducted to determine current states' practices in improving visibility and safety in moving work zones. A review of current references, existing literature and responses to the surveys are described.

GUIDANCE FOR MOVING OPERATIONS (LITERATURE REVIEW)

Part 6 (Temporary Traffic Control) of the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*) presents several recommendations and standards of practice for moving operations. Specifically, Section 6G.02 includes guidance in the use of appropriate traffic control devices, shadow vehicles, and warning lights. Typical applications in Chapter 6H illustrate these applications for mobile work on shoulders (TA-4), two lane roads (TA-17), and multilane facilities (TA-35). These applications require or strongly recommend that work vehicles be equipped with rotating or strobe warning lights. Arrow panels and shadow vehicles are also recommended when work occupies a traffic lane. Truck mounted attenuators are presented as an option for additional protection. Refer to Figures 1 and 2.

In addition to the MUTCD, several studies and states' practices describe application of certain devices to better delineate work vehicles and activities in mobile operations.

Although the MUTCD does recommend the use of warning lights on moving vehicles, neither color nor configuration is detailed. Several states have adopted specific colors such as blue. (Alaska, Colorado, Minnesota, Texas, and others). The blue color is opined to signify a particularly hazardous exposure, demanding a higher level of driver alertness. (*2*, *3*). A study by the Texas Transportation Institute (TTI) found that a combination of colors and configurations was effective in alerting drivers and reducing speeds (*3*). Other studies have also considered various warning light designs, configurations, and applications including light bars, rotating beacons, strobes, and flashers. Two rotating lights supplemented with a flashing light were found most effective in some studies (*4*, *5*). The state of Missouri found that strobes were more effective than standard rotating lights and now use these in many maintenance operations (*6*).



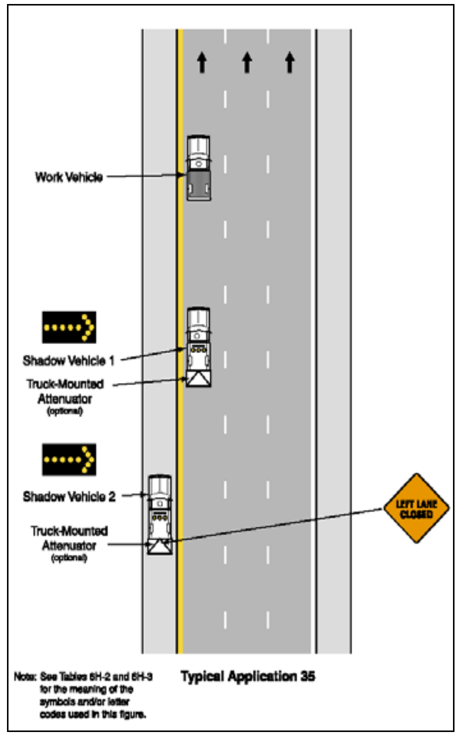


FIGURE 1. MUTCD TA-17 **FIGURE 2. MUTCD TA-35**

Shadow vehicles and truck-mounted attenuators (TMAs) can provide an extra level of protection for both workers and road users. While no known standards fully describing the use of these devices have been identified, some practices have been adopted in various states. Humphreys and Sullivan (*7*) developed guidelines for the use of shadow vehicles and TMAs in various mobile and short-term applications, and these are shown in Tables 1 and 2.

TABLE 1. Recommendations for the Assignment of Shadow Vehicles

Closure/Exposure Condition	Freeway	Non-Freeway with Speed Limit		
		>=50 mph	40–45 mph	<=35 mph
Shadow vehicle for no formal lane closure for operation involving exposed personnel	Very highly recommended	Very highly recommended	Very highly recommended	Very highly recommended
Shadow vehicle for no formal lane closure for operation **NOT** involving exposed personnel	May be justified*	May be justified*	May be justified*	May be justified*
Shadow vehicle for no formal shoulder closure for operation involving exposed personnel	Highly recommended	Highly recommended	Recommended	Recommended
Shadow vehicle for no formal shoulder closure for operation **NOT** involving exposed personnel	May be justified*	May be justified*	May be justified*	May be justified*

* May be justified on basis of special conditions encountered on an individual project.

TABLE 2. Recommendations for the Application of Truck-Mounted Attenuators

Closure/Exposure Condition	Freeway	Non-Freeway with Speed Limit		
		>=50 mph	40–45 mph	<=35 mph
Shadow vehicle for no formal lane closure for operation involving exposed personnel	Very highly recommended	Highly recommended	Recommended	Desirable
Shadow vehicle for no formal lane closure for operation **NOT** involving exposed personnel	Highly recommended	Highly recommended	Recommended	Desirable
Shadow vehicle for no formal shoulder closure for operation involving exposed personnel	Highly recommended	Recommended	Recommended	Recommended
Shadow vehicle for no formal shoulder closure for operation **NOT** involving exposed personnel	May be justified*	Recommended	Desirable	May be justified*

* May be justified on the basis of special conditions encountered on an individual project.

Retro-reflective markings are very effective in enhancing visibility of work equipment at night and in low light conditions. The National Highway Traffic Safety Administration (NHTSA) requires large commercial trailers to be treated with these materials and a resultant reduction in nighttime impacts have been noted (*8*). Several states also utilize retro-reflective tape to improve conspicuity of maintenance vehicles. New Jersey applied this highly visible material to maintenance vehicles in 1996 and followed with similar use on snowplows and emergency vehicles (*9*). Minnesota, Iowa, and several other states make effective use of these retro-reflective markings. A study by TTI found that retro-reflective magnetic strips applied to flagger vehicles were effective in improving night visibility and recommended further use (*10*).

Advanced vehicle control systems (AVCS) have been developed to improve safety for both workers and road users, particularly in higher speed and traffic volume exposures. Shadow vehicles and snow removal equipment pose special hazards for operators and ASCS applications have been employed to address these concerns. Two applications, a remote driven vehicle (RDV) and a fully autonomous shadow vehicle are designed to actually remove the operator from hazardous exposure by allowing remote operation (*11*, *12*) These systems have been tested in Minnesota under the Strategic Highway Research Program (SHRP) and may be fully available in the future. AVCS devices can also provide needed guidance assistance to snowplow operators during periods of severely reduce visibility. Using a magnetic marking system in the roadway, operators are provided edge of road and forward collision data to assist in steering. Improved safety and efficiency are expected through use of this innovation.

Visibility of snowplowing equipment is a particular concern in many states, considering the adverse effects of blowing snow, headlight glare, and obscured windows. A National Cooperative Highway Research Program (NCHRP) study investigated methods for improving conspicuity of snowplows and vision of operators (*13*). The study concluded that steady burning light bars mounted on the rear of the plows can be effective in increasing on-coming drivers' recognition of reduced vehicle speed and location. In addition the study concluded that side vanes mounted on the rear of trucks and deflectors on the front can reduce accumulation of snow on the vehicle thus improving performance of warning lights and overall vehicle visibility for road users. The Iowa Department of Transportation (Iowa DOT) in 1995 studied crashes involving snow removal equipment, concluding that reduced visibility was a major contributor to rear end collisions (*14*). The study recommended use of rear deflectors to reduce snow accumulation with improved lights and retro-reflective markings for better road user visibility. A follow up study in 1999 found the deflectors to be quite effective and recommended extended use as well as utilization of a new design, “scoop” deflector (*15*, *16*) In addition, the Iowa DOT examined the use of “Teflon” spray and taillight air blasters to avoid snow build-up on rear lights (*15*). Experiments with use of a section of snow fence augmented with retro-reflective tape affixed to the rear of snowplow trucks have been conducted by the Iowa DOT and future evaluations for the use of rear view cameras and dual speed displays have been recommended (*17*).

Although the MUTCD does address temporary traffic control measures for moving work operations, the recommendations and guidelines are not significantly definitive. For example, use of warning lights is required for some applications, but color is not specified. Consequently considerable variability in traffic control procedures and practices among the states is observed. Interest and concern is evident in this area among transportation agencies therefore additional guidance for safe and efficient temporary traffic control in mobile operations, particularly for night work, is needed.

SURVEYS

To assess common practice among the states in addressing traffic control needs for moving operations, a national survey was conducted. Forty-eight state departments of transportation were contacted either by email or telephone. Thirty-four state departments of transportation and three Wisconsin counties responded to the survey, an approximate 71 percent rate. In addition, to learn about maintenance work zone visibility practices in rural local agencies in Iowa, a survey was conducted in all 99 counties. This survey was sent to county engineers requesting information pertaining to temporary traffic control measures for maintenance vehicles during routine granular road maintenance and snow removal operations. Sixty-one counties, approximately 62 percent, provided information in response.

All responding state agencies use amber colored warning lights, either exclusively or with other colors. Alabama and Rhode Island use a mixture of white, amber, and red, while Alaska, Colorado, and Mississippi prefer amber and blue. Other states use different combinations but only Louisiana uses amber and red together. Strobe warning lights are preferred over rotating beacons in most responding states due to the perceived improved conspicuity. Several states are experimenting with light emitting diodes (LEDs) in various applications.

Retro-reflective marking material is applied to large trucks in most responding agencies, although the color varies. Over half use red and white tape, but California for example utilizes a six inch wide orange marking on both sides of large vehicle cabs. Idaho uses a retro-reflective yellow stripe on all vehicles and Massachusetts applies a blue and green combination. Small vehicles are also marked with retro-reflective materials in many agencies.

Vehicle color is important in providing recognition and distinction to maintenance vehicles. Orange and yellow appear to be the most popular colors although white is also specified in some agencies. Eau Claire County in Wisconsin has selected yellow-green as the color of choice, opining that this color provides more visibility under a wider range of lighting conditions.

Almost all responding states use shadow vehicles, arrow panels, and/truck mounted attenuators in at least some moving operations. Use and number of these devices varies among states. Georgia, Massachusetts, and New Hampshire report effective use of changeable message signs on some moving equipment. Kansas uses a mobile radio transmitter to advise approaching drivers of mobile painting operations.

Visibility of snow removal equipment is a major concern in many states and a variety of methods and materials were identified from survey responses. Generally more warning lights are employed on snowplowing vehicles than for standard maintenance equipment. Additional lights are added to both front and rear of vehicles and some states place warning lights on plow blades. Some agencies use different colored lights for snow removal than standard maintenance activities. Blue and white colors supplementing amber are displayed in some states. LED lights are also used by a few agencies. Retro-reflective markings are also increased on snow removal vehicles to improve visibility and supplemental flags are mounted on this equipment in some states such as Kentucky and Nevada. Deflectors and air foils are used to reduce air borne snow and subsequent accumulation in several agencies.

A unique survey of Iowa counties was undertaken to learn of common traffic control practices used by local agencies for routine mobile maintenance and snow removal operations. Of particular interest were measures employed during granular road maintenance using motor patrols. Survey responses indicate that most Iowa counties use amber rotating or strobe warning lights and vehicle mounted warning signs on motor patrols, however only eleven counties reported using ground mounted warning signs for these activities. Flags attached to the vehicles is a strategy in some counties and a few local agencies use strobe lights to good effect. For snow removal operations, 75 percent of responding counties reported using retro-reflective markings on vehicles, many applied directly to plow blades. A high percentage of counties also mount warning flags on snow removal equipment. Auxiliary headlamps and strobe lights exhibit common usage and about 39 percent of responding counties use snowplow deflectors.

CONCLUSIONS AND RECOMMENDATIONS

The surveys revealed a high interest in safety for workers and road users in moving operations among the states and Iowa counties, evidenced by increased use of warning lights and other devices. Some procedures are quite common, but a great deal of variation in practice was determined. Experimentation with new methods and materials for enhancing visibility of mobile equipment reinforces a common concern for the potential risks posed by a constantly moving work area.

A need for more definitive guidelines and recommendations on a national level is evident. As new materials are developed and improved methods discovered, focused studies should be undertaken to identify and present reliable guidance in effective traffic control measures for use by transportation agencies in these particularly hazardous exposures for both workers and road users.

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Effectiveness of Speed Advisory Sign Systems in Reducing Speeds Upstream of a Traffic Slowdown

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ABSTRACT

A primary safety concern associated with work zones on rural interstate highways is the increased crash potential when congestion occurs on the approach to a work zone. Depending on the traffic volume and capacity of the work zone, the queue of slow-moving or stopped vehicles caused by the congestion may extend rapidly upstream creating a high speed differential between the end of the queue and approaching traffic. The unexpectedly sudden encounter with congestion often makes it very difficult for some drivers to safely reduce their speeds and avoid colliding with other vehicles as they approach the end of the queue. One solution to this problem is to use some Intelligent Transportation System technology that can warn driver of possible hazard downstream. The D-25 Speed Advisory Sign System from MPH Industries is one of the technologies. It was evaluated as part of the Midwest States Smart Work Zone Deployment Initiative, a pooled-fund study sponsored by Iowa, Kansas, Missouri, Nebraska, and the Federal Highway Administration.

The system deployed for the purpose of this evaluation consists of a series of three MPH D-25 speed trailers that operates independently placed at approximately 1/4- to 1/2-mile intervals depending on the weather, terrain, and prevailing roadway and traffic conditions. Each trailer is equipped with: (1) an LED display with 25-inch speed digits, (2) directional radar directed toward downstream traffic, (3) two flashing strobes to warn drivers of downstream problems, (4) SPEED OF TRAFFIC AHEAD sign mounted over the speed display, and (5) USE EXTREME CAUTION WHEN FLASHING sign mounted beneath the speed display. The on-board radar monitored speeds downstream of the trailer. When a traffic slowdown was detected, the strobe lights began flashing. When there was no slowdown, the strobe lights were off and either the speed of traffic downstream or the work zone speed limit was displayed, whichever was lower. The messages were intended to warn drivers of stopped or slow-moving traffic ahead and thereby enable them to reduce their speeds and the potential of rear-end crashes.

The system was evaluated based on speed data measured by the MPH D-25 speed trailers. Traffic was also videotaped during peak hours to study driver behavior, conflicts, and braking activity. The initial results of the on-going evaluation indicated that the speed messages were effective in reducing speeds.

Key words: speed advisory sign systems—traffic congestion—traffic speed reduction—work zone safety

Note: Data collection and analysis were still in progress at the time of publication; final results will be presented at the symposium.

Bridge Prioritization for Installation of Automatic Anti-icing Systems in Nebraska

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ABSTRACT

During severe winter conditions bridges freeze before the surrounding roadways, often catching unsuspecting drivers off guard. Depending on weather and pavement conditions, automatic bridge deck anti-icing systems spray chemicals that prevent or minimize ice bonding to the pavement. The Nebraska Department of Roads (NDOR) is interested in installing such systems on various bridges statewide. However, the presence of 2,193 bridges in Nebraska and availability of limited funding required candidate bridge prioritization for installation of automatic anti-icing systems based on relevant criteria. This research was undertaken with the objective of developing a decision-aid tool that could aid NDOR with the prioritization of bridges for installation of automatic anti-icing systems.

The authors reviewed literature on automatic bridge anti-icing systems and the experiences of various transportation agencies. Factors considered important in the installation of automatic anti-icing systems included: accident history, bridge alignment, weather, traffic, and bridge distance from maintenance yard, among others. A methodology to build a database of relevant factors and a decision-aid tool that helped with narrowing the list of candidate bridges was developed. Database construction involved merging information from a variety of disparate sources in a geographic information system on factors deemed important in prioritization of bridge deck anti-icing system installations. Some of the sources included NDOR bridge inventory, NDOR accident database, archived weather data from the High Plains Regional Climate Center and the National Weather Service, Nebraska streets database, Nebraska rivers and streams database, and NDOR maintenance yard data. Four different decision-aid methods were initially incorporated in the decision-aid tool: benefit-cost ratio, cost-effectiveness, utility index, and composite programming. However, subsequent research indicated that for various reasons, the prioritization results from the four methods were not suitable for Nebraska. As such a fifth method, called the NDOR Preferred Method, was developed in close consultation with the project's technical advisory committee. The prioritization results from this method were accepted after extensive scrutiny. The data integration methodology and the processes developed in this research should be useful to other transportation agencies contemplating a methodical approach to installation of automatic bridge deck anti-icing systems for highway safety enhancement.

Key words: anti-icing—bridge—geographic information system—Nebraska—prioritization

BACKGROUND

A major safety issue during severe winter weather is the freezing of moisture on bridge decks before moisture-freeze on the rest of the roadway surface. Many traffic accidents occur when unsuspecting drivers lose control of their vehicles while traveling over frozen bridges. To mitigate this issue, some state transportation agencies have successfully used automatic bridge deck anti-icing systems. These systems apply deicing liquid chemicals to bridge decks when icing conditions are detected, thereby preventing moisture from freezing on the bridge deck (see Figure 1). The Nebraska Department of Roads (NDOR) is interested in installing automatic bridge deck anti-icing systems in Nebraska for safety enhancement. However, the presence of 2,193 bridges and the availability of limited funds necessitate candidate bridge be prioritization for installation of such systems. Therefore, a research project to prioritize bridges for the installation of anti-icing systems was initiated by the NDOR along with the University of Nebraska-Lincoln.

The objective of this research was to develop a decision-aid tool for prioritizing bridges for most effective installation of automatic bridge deck anti-icing systems in Nebraska. To achieve this objective, the authors extensively reviewed literature on automatic bridge deck anti-icing systems as well as experiences of various transportation agencies with such systems. Based on this review, a two-step methodology was developed to guide the construction of an appropriate database and the development of the decision-aid tool for bridge prioritization. Data from a variety of disparate sources were integrated in a geographic information system (GIS) to construct the needed database and a number of prioritization methods were incorporated in the decision-aid tool. Using the decision-aid tool, two candidate lists of bridges were developed for installation of automatic bridge deck anti-icing systems for Omaha and non-Omaha areas. This paper describes the findings from the literature review, the research methodology, construction of the database, development of the decision-aid tool, and prioritization of the candidate bridges for installation of automatic bridge deck anti-icing systems. For complete details on this project, the readers are referred to the report by Khattak et al., (*1*).

LITERATURE REVIEW

An automatic bridge deck anti-icing system sprays deicing chemicals on a bridge deck from nozzles installed in the pavement or bridge parapet/railing. Such a system is intended to prevent freezing of the bridge deck prior to the rest of the roadway, thus making it safer for users. Several studies have documented the benefits of automatic bridge deck anti-icing systems. Friar and Decker (*2*) reported a 64 percent reduction in the number of reported accidents due to installation of such a system in Utah. A study by Stowe (*3*) indicated a benefit-cost ratio of 2.36 and a net benefit of $1,179,274 for an automatic anti-icing system. The Minnesota Department of Transportation (MnDOT) reported on three automatic anti-icing system installations in Minnesota: Interstate 35 Bridge near Duluth, Truck Hwy 61 Bridge near Winona, and an intersection in Dresbach (*4*). MnDOT reported accident reductions of 56, 100, and 100 percent at the Duluth, Winona, and Dresbach locations, respectively. Benefit-cost ratios of 2.0, 3.1, and 2.7 were reported for these three locations. Another Minnesota-based study by Johnson (*5*) evaluated the I-35W and Mississippi River Bridge anti-icing system. The evaluation indicated a reduction of 68 percent in the number of winter season accidents on the bridge and a benefit/cost ratio of 3.4.

Barrett and Pigman (*6*) evaluated a bridge deck anti-icing system installed on southbound I-75 at the North interchange to Corbin, Kentucky. After four winter seasons, the system had minimal problems and worked efficiently. However, the system's effectiveness was limited by its location. Being on an interstate, the bridge was subject to significant winter maintenance activities anyway, and it was located in a part of the state that does not receive abundant precipitation. Barrett and Pigman recommended that the system be used in the following places: 1) accident-prone areas, 2) isolated bridges that require deicing trucks to travel an unreasonable distance to treat, 3) remote areas that are difficult to reach in bad weather, and 4) bridges over water, which may be more susceptible to freezing moisture.

Finally, NDOR conducted a survey (unpublished) to assess automatic anti-icing system usage among state transportation agencies and to determine the existence of any criteria and guidelines for deploying such systems. Of the 19 agencies that responded to the survey, eight (42 percent) indicated they either use or plan to use bridge deck anti-icing systems. Only two (10.5 percent) state agencies (Maryland and Wisconsin) indicated that they have prioritized plans for the installation of such systems. Major criteria used by Wisconsin include: accident history, bridge grade, locations susceptible to black ice or frost, super-elevated decks, average daily traffic (ADT), distance from the nearest salt stockpile, potential for moisture generation, high winds, and bridge span. The NDOR survey asked respondents what criteria they would use for prioritization of bridge locations. Some of the frequently cited criteria were accidents, bridge distance from maintenance yard, and adverse weather.

In summary, the literature review indicated the economic viability of automatic anti-icing systems. The benefit-cost ratios of such systems are in the range of 1.8 to 3.4 and accident frequency reduction varies from 25 to 100 percent. However, benefits from these systems are location-dependent. Very few state transportation agencies have bridge prioritization plans for installation of automatic bridge deck anti-icing systems. Some of the important factors in bridge prioritization include accident history, bridge alignment, weather, and distance from maintenance yard. The next section presents a brief narrative of the methodology adopted by the authors for this research.

METHODOLOGY

Figure 2 presents the methodology used for database construction from various sources and development of the decision-aid tool. Database construction was accomplished in a GIS while the decision-aid tool was developed in a spreadsheet. Data from several sources were integrated in a GIS using a variety of processes. The data utilized included bridge inventory, state accident data, weather information, traffic information, maintenance yard information, and Nebraska streets, rivers, and streams data. Additional elements were added to the integrated data to enhance its effectiveness for usage by the decision-aid tool. The decision-support tool utilized the integrated database to provide prioritized lists of candidate bridges for the installation of automatic bridge deck anti-icing systems. A description of the database construction is given next.

DATABASE CONSTRUCTION

Data from several sources were integrated in a GIS to construct the needed database. The data sources included: state accident and traffic information from the NDOR traffic division, bridge inventory from the NDOR bridge division, and weather-related data from the High Plains Regional Climate Center and the National Weather Service. Information on maintenance yards was obtained from the NDOR maintenance division while data on Nebraska street, rivers, and streams were obtained from commercial sources. A brief description of each source, its contribution, and the integration process are given below.

The NDOR Bridge Division provided data on 2,193 bridges in the state. These data contained information on the following elements:

- structure number
- route number
- location (latitude and longitude)
- functional classification
- length of bridge
- approach roadway width
- alignment
- age
- wearing surface

NDOR maintains separate records for pairs of bridges on the same route if they are physically separate. Since the automatic bridge deck anti-icing system will be installed in both directions therefore, for this study the records of such bridge pairs were aggregated. For example, the number of lanes on both bridges were added, the number of crashes on both bridges were added, and so on. Some elements that were common among bridge pairs were not added; e.g., functional classification, bridge length, type of railing, etc.

Thirteen-year (1988-2000) state accident data were obtained from the NDOR traffic division, which maintains all police-reported accidents in Nebraska. Accident attributes included elements such as date and time of accident, road surface conditions, route number, and injury severity of those involved in the accident. Surface condition was categorized into dry, wet, snow/icy and the injuries were measured on the KABCO scale – Killed, A-type injury (incapacitating), B-type injury (evident), C-Type injury (complaint of pain), and Property-Damage-Only. Using GIS, the accident data were first imported into the GIS and then spatially merged with the bridge data. Accidents reported on or within 300 feet of either end of each bridge in the state were extracted. This effectively excluded non bridge-related accidents from the analysis. The accident data were further culled by limiting to those accidents that were reported during snowy, icy, or frosty surface conditions (information available on the accident report) or when the minimum average temperature for the day was less than 320 F and precipitation was present. The use of the average daily minimum temperature in the selection of crashes for subsequent analysis ensured that all possible crashes that might have occurred under conditions that could be ameliorated by the automatic bridge deck anti-icing system were taken into consideration. Calculation of the average minimum daily temperature is described along with the weather information.

The NDOR traffic division provided Year 2000 statewide traffic data (ADT and truck percentage), which were integrated into the database. However, the use of only Year 2000 traffic data results in bias in some of the calculations. For example, its use in calculation of bridge crash rates prior to Year 2000 results in under-estimation of crash rates, assuming ADT increases over time. The use of yearly ADT (1988, 1989, …, 2000) would overcome such a problem. However, more specific historical traffic data were not readily available. Since bridge prioritization is based on consideration of bridges relative to each other, the bias effect may be limited if the ADT growth is somewhat similar across the bridges.

Two sources were used to obtain relevant weather information: the High Plains Regional Climate Center and the National Weather Service Automated Weather Data Network (AWDN). Access to archived electronic Nebraska temperature data for the 13-year study period (1988-2000) was obtained from the High Plains Regional Climate Center. These data included the daily maximum and minimum air temperatures from 316 weather stations located throughout Nebraska. The extracted temperature data along with the weather station locations were input to the GIS. These extracted temperature data were used in the accident selection process (accidents that occurred on days when the minimum temperature was below freezing). To find temperature at the accident site on the day of its occurrence, a boundary of 25-mile radius was drawn around each bridge. Weather stations located within this boundary provided data for estimation of average minimum and average maximum temperature for the day of the accident. The 25-mile radius was selected to ensure that all accidents had at least one weather station for temperature estimation. The National Weather Service AWDN provided information on minimum and maximum wind speed and daily precipitation (this information was not available in the High Plains Regional Climate Center data). However, the National Weather Service data were limited to only forty weather stations distributed through out Nebraska. These weather stations along with their respective data were input to the GIS. Since the number of stations was less, it was assumed that the weather conditions at any bridge were similar to those reported at the nearest weather station, i.e., data from multiple stations were not averaged. The National Weather Service data also contained temperature information, which was used to estimate the number of days that each bridge experienced below freezing temperatures. Note that the weather information at each bridge was found by assuming that conditions were similar to those recorded at the National Weather Service station. Other options such as inverse distance weighted interpolation, spline interpolation, etc., could be utilized for bridge weather evaluation from the National Weather Service weather stations instead of making the assumption mentioned above. These options were not used for parsimony in this study. Also, the process described above results in the estimation of air temperature and not bridge-deck temperature. The temperature of a bridge-deck may be different than the surrounding air temperature. However, bridge-deck temperature data are rarely available and were not available for this research.

No electronic file containing location information on NDOR maintenance yards in Nebraska was available that could be promptly used in this study. GIS capability to match postal addresses to a street database was used along with postal addresses of the maintenance yards to obtain the appropriate locations of maintenance yards in the database. Information on the service areas of each maintenance yard was obtained from NDOR and manually input to the GIS. Travel times and distances from each maintenance yard to each bridge in its respective service area were calculated using the maintenance yard locations, their respective service areas, the Nebraska street database, and bridge locations. GIS capability to find the shortest path based on travel time or travel distance was utilized.

Commercially available Nebraska street data were obtained from the Environmental Systems Research Institute (ESRI) and utilized for street network and postal address matching. Travel time on each link was calculated by first finding the link length and then the posted link speed limit. The assumption in calculation of travel time is that maintenance vehicles will travel at the speed limit. However, it is possible that travel time during adverse weather may be different than the time calculated based on speed limit. Commercially available Nebraska rivers and streams geographic data were obtained from ESRI and utilized to determine the locations of rivers and streams in Nebraska with respect to the bridges. Using the "select by theme" capability of GIS, bridges located on rivers and streams were selected and identified as bridges with water flowing underneath.

Additional Elements

Additional elements were added to the database including unit accident costs for computing the cost of the accidents reported at each bridge and automatic bridge deck anti-icing system cost for each bridge. Using these costs, the loss ($) during the study period at each bridge was calculated. The addition of the above information to the database completed the database integration process in the GIS. The completed database contained information on various elements that included:

- Accident frequency under during snowy, icy, or frosty surface conditions on or within300 feet either side of bridges
- Bridge accident rate
- Accident loss ($)
- Distance of bridge from its respective maintenance yard
- Travel time to bridge from its respective maintenance yard
- Bridge age (since original construction as well as since last reconstruction)
- Number of lanes
- Bridge functional classification
- Bridge Year 2000 ADT and truck percentage
- Bridge span
- Bridge approach width
- Presence of water under bridge
- Number of days with wind speed greater than 15 mph at the bridge location
- Number of days with precipitation at the bridge
- Number of days with average minimum air temperature below freezing at the bridge
- Number of days with average maximum air temperature below freezing at the bridge

These elements were subsequently utilized in the development of the bridge prioritization decision-aid tool. However, before its usage, a number of tests were conducted to verify the efficacy of the database. These tests and the corresponding remedial measures undertaken are briefly described next.

DATABASE INTEGRITY

Tests were conducted on the database to validate its integrity and effectiveness. Minimum and maximum values for each factor were looked at to ensure that they were with in reasonable range. This test indicated unacceptably high accidents at some bridges on relatively minor routes. Investigation showed that these were overpass bridges that were mistakenly assigned the main route accidents. All overhead bridges were checked and corrected in the database. ADT values at certain locations looked suspiciously low or high. These were verified from NDOR and corrected. A test on speed limit values of various functional class highways indicated that some of the routes in the database did not have correct speed limit values. Again, these were corrected based on information obtained from NDOR. Travel times and travel distances between maintenance yards and various bridges were validated by contacting NDOR personnel who were familiar with bridge and maintenance yard locations. Weather data were validated by looking at general weather patterns in Nebraska and by comparing the values obtained from the analysis to those general weather patterns. These tests ensured, to a certain degree, that the database was ready for use in the decision-aid tool. Development of the decision-aid tool is described next.

BRIDGE PRIORITIZATION DECISION-AID TOOL

The bridge prioritization decision-aid tool was developed in a spreadsheet and it incorporated five different prioritization methods. The five methods were: 1) benefit-cost, 2) cost-effectiveness, 3) utility index, 4) composite programming, and 5) NDOR preferred method. A brief description of each method follows; for complete details of these methods the reader is referred to (*1*).

Benefit-Cost Method

Bridges were prioritized based on the ratio of benefits generated from the installation of anti-icing systems and the associated costs. Bridges with higher benefit-cost ratios were given higher priority for the installation of an automatic anti-icing system. Benefits and costs were quantified in monetary terms; estimation of benefits involved looking at avoided accidents due to installation of automatic anti-icing systems while estimation of costs was based on the purchase cost of such systems. Based on information gleaned from the literature, it was assumed that installation of anti-icing systems would result in a 60 percent reduction in accidents. Benefits were then calculated by using accident costs for different injury levels. Avoided traffic delays due to fewer accidents would also contribute to benefits; however, data required to estimate traffic delay due to accidents were not readily available and therefore, benefits from avoided traffic delays were not included in this method.

Cost-Effectiveness Method

The cost-effectiveness method evaluated each bridge based on the expected safety improvement per unit cost due to installation of an anti-icing system. The expected safety improvement of an anti-icing system was expressed in terms of a safety improvement index (SII), which was a function of the expected reduction in accident frequency and severity. Using different injury levels, SII was calculated by assuming a 60 percent reduction in accidents due to installation of the automatic anti-icing systems. The weights expressing the relative importance of accidents with different injury severity levels were proportional to the accident costs involving different injuries. The system cost was determined for each bridge based on the costs estimates used in the benefit-cost analysis. The SII was divided by the system cost to determine the safety improvement index per unit cost. Bridges were prioritized on the basis of this ratio with higher priority given to those with higher values.

Utility Index

The utility index method involved assigning weights to various criteria that were important in the comparative evaluation of bridges. A panel of experts from the NDOR was asked to provide weights to different criteria used in the Utility Index method. The weighted score for each bridge was then normalized on the 0-1 interval and used in prioritizing bridges.

Composite Programming

The Composite Programming method is somewhat similar to successive application of the Utility Index method. Individual criteria were weighted and then grouped. The groups were then assigned weights to reflect their importance relative to each other. Successive weightings and groupings produced a final score that was normalized on the 0-1 interval. The individual criteria and group weights were assigned by the panel of experts from the NDOR.

NDOR Preferred Method

The NDOR Preferred Method was developed because prioritization results from the earlier four methods were not sufficiently discriminating amongst the various criteria considered important in the installation of automatic anti-icing systems. This was due to the fact that most of the criteria were given relatively high scores by the panel of experts. Also, the project technical advisory committee placed high importance on a method that was simple and straightforward. To this end, various scenarios were created where bridges were prioritized based on different criteria. These included prioritization based on all reported accidents within +/- 300 ft of the bridges, bridge accident rate, bridge environmental considerations, geographic considerations, etc. Depending on the scenario, the high traffic experienced by the Omaha bridges resulted in skewed prioritized bridge candidate list. To avoid skewed results, Omaha bridges were separated from other bridges and two candidate priority lists developed. It was also decided to limit candidate bridges to those experiencing 13 or more accidents during the study period.

Results from the various scenarios were closely inspected; the prioritization by simple accident frequency provided the most realistic and useful results for Nebraska. As such, the decision-aid tool was modified to first limit candidate bridges to those that experienced 13 or more accidents during the study period and then prioritize those bridges on simple accident frequency. This method was simple and straight forward and provided results deemed appropriate for this research by the project’s Technical Advisory Committee and the research team. Two priority lists, one each for Omaha and non-Omaha bridges, were generated based on this method. NDOR will consider bridges at the top of these two lists for the installation of automatic bridge deck anti-icing systems.

CONCLUSIONS

This research indicated that data from a variety of sources can be combined for decision-aid in installation of automatic bridge deck anti-icing systems. Several methods were incorporated in the decision-aid tool that developed in this research. This decision-aid tool provided two priority lists of candidate bridges based on criteria that were deemed most pertinent for Nebraska. Even though a relatively straight forward ranking by accident frequency was eventually used in the production of the two lists via the NDOR Preferred Method, the other methods incorporated in the decision-aid tool provide options for conducting analyses based on different considerations. Regarding the two prioritization lists, the authors recommend that they are not the final word in terms of which bridges should be installed with the automatic anti-icing systems in Nebraska. Institutional decision making is a complex process that must consider a variety of issues that are impossible to capture by a computerized process. As such, the use of these priority lists should be restricted to narrowing the list of candidate bridges to a manageable few. The authors recommended that the final decision on the bridges that would receive an automatic anti-icing system from amongst the prioritized bridges should be made by the NDOR by taking into consideration additional issues that could not be captured by the decision-aid tool.

The methodology and database integration processes presented in this paper should be useful to transportation agencies contemplating installation of automatic bridge deck anti-icing systems for highway safety enhancement. Some aspects of this research can be further refined in future studies. Better estimates of weather conditions can be obtained by using more sophisticated analyses techniques. The analyses can be improved by using pavement temperature rather than air temperature or by establishing a relationship between air and pavement temperature on a local basis (the relationship might vary from place to place). Finally, other states may have additional data sources that might be relevant in the bridge prioritization process. It is recommended that those additional data sources should be utilized to the full to help make better decisions.

ACKNOWLDGMENTS

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FIGURE 1. A Bridge-Deck Anti-Icing System in Operation (Source: Odin Systems International, Inc.)

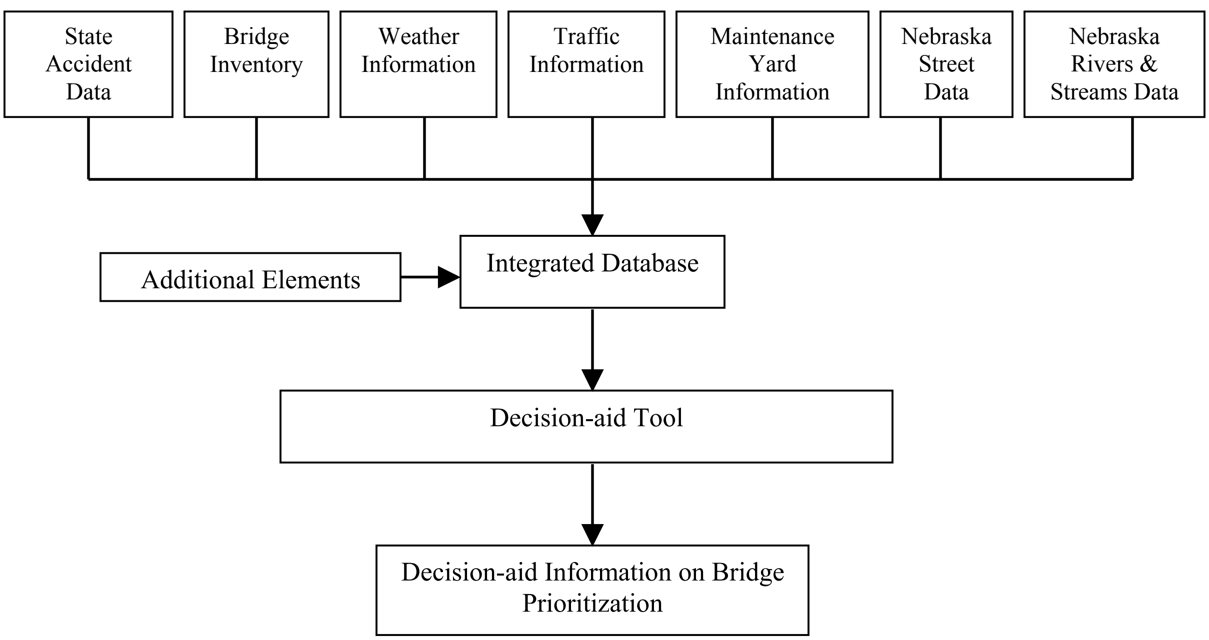

FIGURE 2. Adopted Research Methodology

Repair of Damaged Prestressed Concrete Bridges Using CFRP

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ABSTRACT

Every year many prestressed concrete (P/C) girder bridges in Iowa are damaged by overheight vehicles. Traditional P/C girder repair strategies include welded steel jackets, internal strand splices, and external post-tensioning. Unfortunately, these types of repairs are both labor intensive and vulnerable to future corrosion. One possible alternative to these traditional repair techniques is to repair/strengthen impact damaged P/C girders with carbon fiber reinforced polymers (CFRP). These types of materials have the advantage of large strength/weight ratios, excellent corrosion/fatigue properties, and are relatively simple to install. This paper will present experimental results from one of the three damaged P/C girder bridges that were repaired using CFRP.

The results from testing this bridge before and after being repaired with CFRP will be presented. Data from these tests verified that CFRP is an effective method for repairing/strengthening damaged P/C girder bridges.

Key words: carbon fiber reinforced polymers—damaged prestressed concrete bridges—field testing repair—strengthening

INTRODUCTION

Each year numerous highway bridges in Iowa are damaged by impacts due to over height vehicles, which result in costly repairs and traffic disruptions. Iowa State University, in conjunction with the Iowa Department of Transportation (Iowa DOT) is investigating the use of externally bonded carbon fiber reinforced polymers (CFRP) to repair and strengthen damaged prestressed concrete (P/C) girders. Though different types of repair methods, materials, and elements are being considered, the objective of this research is to investigate the effectiveness of CFRP as a material to repair damaged P/C bridge girders.

The repair and strengthening of reinforced concrete (R/C) structures with CFRP is a generally accepted practice. However, there has been minimal research on the use of CFRP to strengthen and repair impact damaged P/C bridge girders. Current accepted methods to repair P/C girders include the use of steel jackets, tendon splices, exterior prestressing or the replacement of the damaged girders. Drawbacks of these methods include relatively high costs, corrosion, poor fatigue performance, and long periods of traffic disruption. The advantages of using CFRP include reduced installation time, corrosion resistance, and ease of application. To determine the effectiveness of CFRP as a repair method for P/C bridge girders, laboratory as well as field tests are being undertaken.

Review of Previous Work

An extensive literature search has been conducted to investigate the use of CFRP in concrete repairs. Included in this search was the investigation of methods for repairing vehicle impacted P/C bridges.

A comprehensive survey was developed and sent to state and Canadian highway officials to determine what policies and procedures are being followed in the case of impact damaged P/C bridges. The questionnaire also addressed whether CFRP had been used as a repair material or was being considered for use. Of the agencies that replied, seven indicated using CFRP to repair various highway structures and only one agency noted it would not consider using CFRP as a repair material.

In HR-397, "Field/Laboratory Testing of Damaged Prestressed Concrete Girder Bridges" *(1)*, a damaged P/C girder bridge (one exterior and one interior P/C beams were seriously damaged) was tested in the damaged state and after the two damaged beams had been replaced. The two P/C girders that were removed were also tested – one in the damaged condition and the other after being repaired with CFRP. Results of this investigation verified the effectiveness of CFRP in repairs. Additional details of this investigation may be found in Ref. 2 as well as in the final report to HR-397*(1)*.

In TR-428, "Effective Structural Concrete Repairs" *(3,4)* research on the P/C bridge girders is continuing. In this Iowa DOT sponsored investigation, which is still in progress, several procedures for structurally repairing damaged reinforced or prestressed concrete elements are being investigated. Only the portion of this study related to the use of CFRP on P/C girders is presented in this paper. Although this portion of the study involved both laboratory and field testing, only a portion of the field work will be presented in this paper.

Field Demonstration Projects

To date, three field demonstration projects have been completed. CFRP was used to strengthen all three of these bridges based upon the results and installation procedures established in the laboratory. As previously noted, only one of the damaged P/C girder bridges, which was repaired using CFRP, will be presented in this paper.

The bridge that was damaged was on the south bound portion of the IA Highway 65 bridge that crosses IA Highway 6 in the vicinity of Altoona, Iowa. This four span bridge consists of two 96.5 ft main spans and two approach spans – the north one 36 ft long and the south one 46 ft long. A cross-section of this bridge is presented in Fig. 1. All six of the LXD P/C girders in one of the main spans were damaged by an overheight vehicle traveling east on IA Highway 6. Although, as illustrated in Fig. 2, all 6 girders were damaged approximately 30 ft from the center pier, most of the damage was to the first two girders. Damage to Beam 1 consisted of spalling of concrete from the bottom flange and the severing of one strand. Damage to Beam 2 (shown in Fig. 3) was the most severe in that significantly more concrete spalled from the bottom flange exposing five strands two of which were severed.

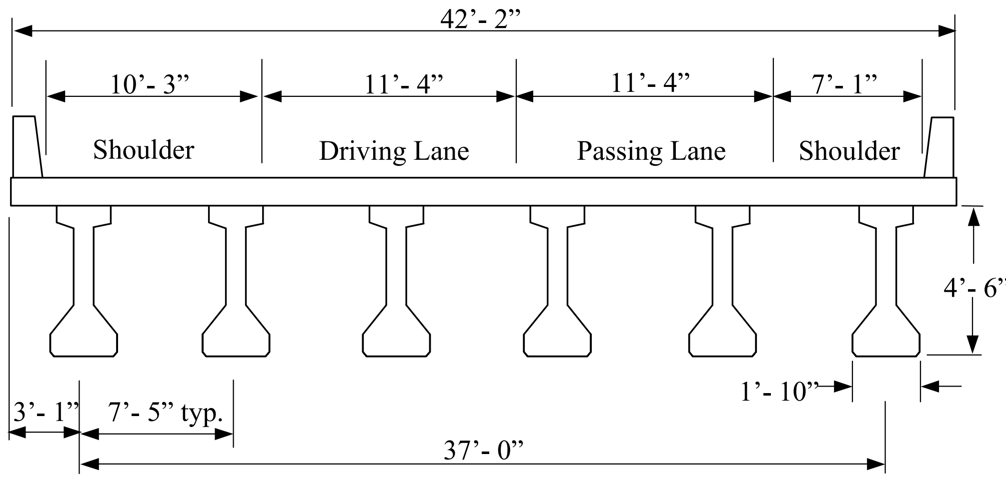

FIGURE 1. Cross section of Altoona Bridge

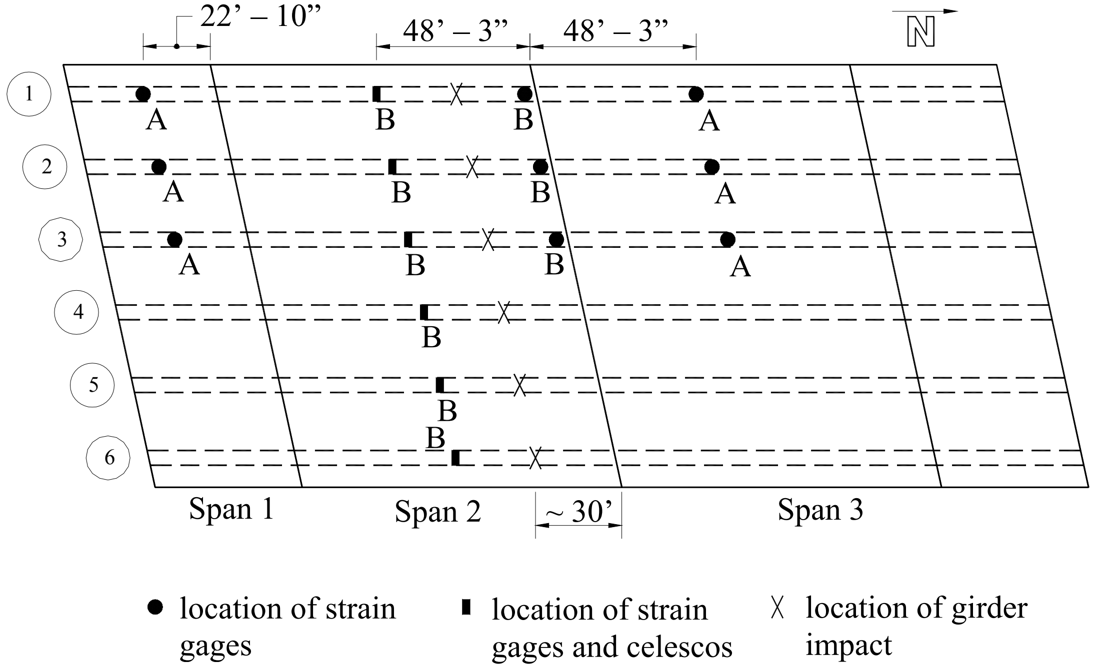

FIGURE 2. Location of Damage, Strain Gages, and Deflection Transducers on the Altoona Bridge

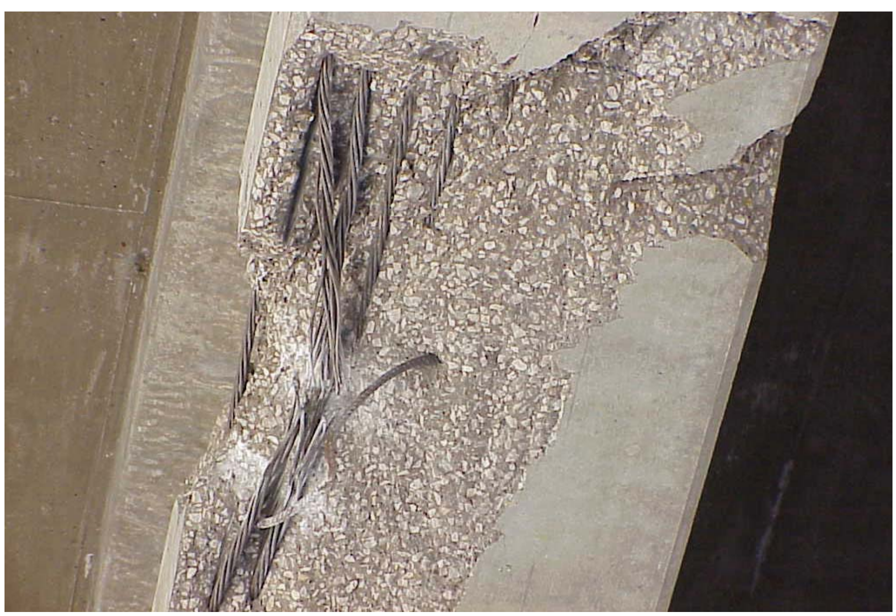

FIGURE 3. Photograph of the Damage to Beam 2 in the Altoona Bridge

FIELD TESTING

As previously stated, this bridge was tested before and after the CFRP was installed so that the effectiveness of the strengthening system could be determined. Instrumentation consisted of strain gages and deflection transducers located as shown in Fig. 2. A total of 24 gages were used on the bridge – one on the side of the bottom flange at Section A in Fig. 2 and two at Section B in Fig. 2 - one on the side of the top and one on the side of the bottom flanges. Deflections were measured at the centerline of all six beams in Span 2. Two Iowa DOT trucks (rear tandem) were used in the testing of the bridge. The two trucks (rear tandem) used in the testing prior to the repairs had an average weight of 51,100 lbs, while the average weight of the two trucks used in the final tests after repairs was 46,700 lbs. Static as well as dynamic load tests were performed on the bridge. Thirty-two different load tests, with the trucks positioned to produce the largest positive and negative moments in the various girders were completed in the static portion of the test. Dynamic data were obtained with one of the trucks traveling at three different speeds: 3-8 mph, 30-35 mph and 65-70 mph. For additional details on the position of the truck(s) in the various tests, the reader is referred to Ref 4.

FIELD REPAIRS

Prior to installation of the CFRP, the manufacturer's recommended patching material and procedures were used to repair the spalled concrete on the various beams, CFRP plates were attached to the most heavily damaged beam – Beam 2 after the patch material had cured. Four 4-in. wide by 75 ft long protruded CFRP laminates (shown in Fig. 4) were installed on the bottom flange of Beam 2 using an epoxy-resin which was applied to both the laminate and the P/C girder. A rubber roller was used to enhance the bond. The tensile design strength of the CFRP laminates was 406 ksi. After installation of the four laminates, a CFRP wrap was installed in the vicinity of the patch (80 in. of the girder was wrapped) to confine the patch and to prevent any plate debonding. Five CFRP strips (approximately 6 ft long which was sufficient length to cover the bottom flange and all but approximately the top 1 in. of the web) were installed. Similar CFRP sheets were installed at the location of the patches in the other beams to assist in confining the patches.

FIGURE 4. Photograph of CFRP on Beam 2 of Altoona Bridge

TEST RESULTS

The Altoona Bridge was tested before (Fall, 2000) and after (Spring, 2001) the bridge was repaired with CFRP. Although both strain and deflection data were recorded during the two tests, since the strains and deflections describe the same change in behavior, only the deflection data are presented in this paper. Deflections at the midspan of Span 2 for two load cases (LC1 and LC2) are presented in Figs. 5 and 6, respectively. In LC1, two trucks – one in Lane 1 (see Fig. 7) and the other in Lane 2 – were positioned at the midspan of Span 2. A photograph of the trucks in this position is shown in Fig. 8. In LC2, two trucks are end-to-end in Lane 1 centered at the midspan of Span 2. The combined weight of the two trucks in LC1 was 102,200 lbs and 93,400 lbs in LC2. Data presented in Figs. 5 and 6 have been normalized to the larger weight (102,200 lbs) so that a comparison of results from LC1 and LC2 can be made. Of the numerous different load cases, LC1 and LC2 are presented because they produce the largest deflections and strains in Beams 1 and 2.

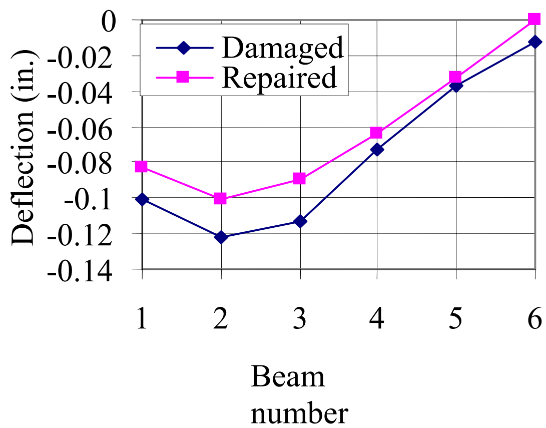

FIGURE 5. Comparison of LC1 deflections in Altoona Bridge

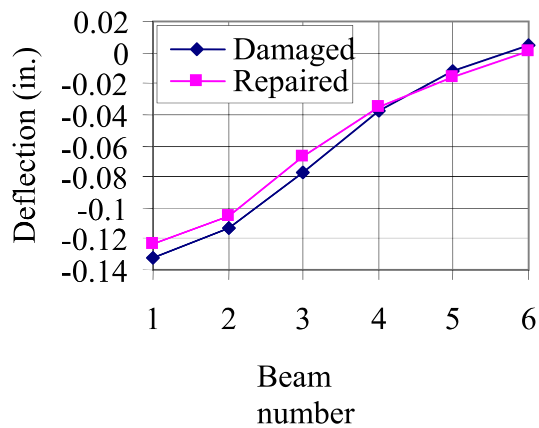

FIGURE 6. Comparison of LC2 deflections in Altoona Bridge

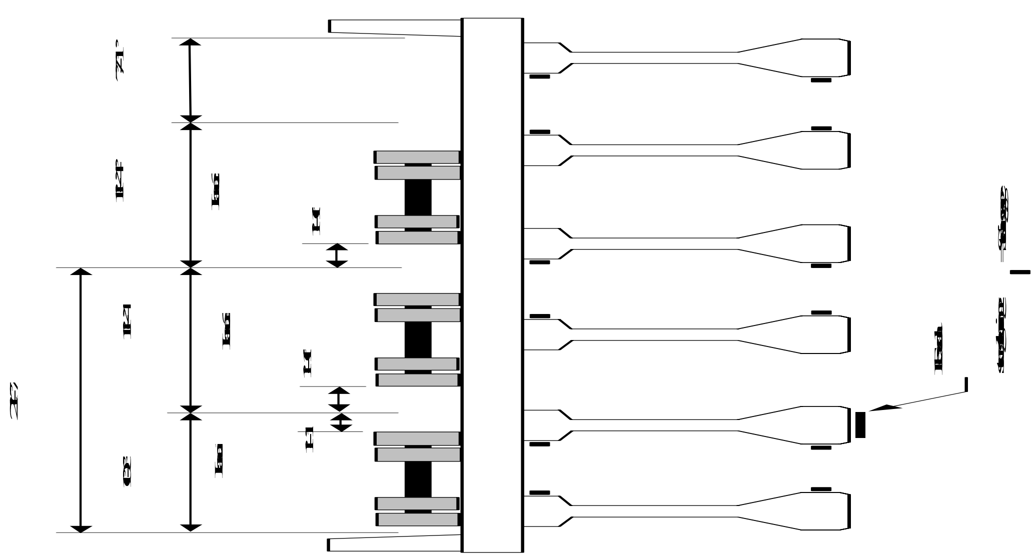

FIGURE 7. Truck lanes used in the Altoona Bridge tests

FIGURE 8. Photograph of Trucks in LC1 Tests

The effect of the CFRP strengthening system is apparent in both Figs. 5 and 6. After the CFRP was installed (which increased the moment of inertia of Beam 2 by approximately 5 percent), deflections at the midspan of Span 2 decreased for both LC1 and LC2. Strains in Beam 2 due to the slight increase Beam 2 stiffness increased a small amount but were still in the 50-60 MII range.

SUMMARY AND CONCLUSIONS

Although only one of the field tests completed in this study has been presented in this paper, based on this field test and other laboratory and field tests, the authors are confident in the CFRP strengthening system. Repair procedures using CFRP were developed to restore the moment capacity to P/C girders that have severed strand(s). CFRP also has the functional capacity of confining patches so that if some of the patch material becomes loose, it does not fall on passing vehicles.

Based upon this study:

- Flexural strengthening of impact damaged P/C girders is feasible when approximately 15 percent of the strands are severed.
- CFRP sheets restore a portion of the flexural strength lost when P/C girders are damaged.
- Transverse CFRP sheets assist in the development of the longitudinal CFRP plates and prevent debonding. Such jackets also confine patch material.
- CFRP reduced beam deflections in some cases by as much as 20%. Actual deflections measured, however, were very small.

ACKNOWLEDGEMENTS

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Urban Four-Lane Undivided to Three-Lane Roadway Conversion Guidelines

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ABSTRACT

In April 2001 guidelines for the conversion of four-lane undivided roadways to three-lane facilities were developed. The content of these guidelines are summarized in this paper. Several successful examples of this type of conversion are identified in the guidelines along with the expected operational impacts and factors that should be considered in the conversion of four-lane undivided to three lanes. The focus of this paper is the factors and the operational analysis results in the guidelines. In addition, results from a University of Wisconsin extension of the guideline operational analysis are discussed. A CORridor SIMulation (CORSIM) software package sensitivity analysis approach was used in two theses to approximate the difference in the operation of similar roadways with either a four-lane undivided or three-lane cross-section. The variables considered in the analyses were total entering traffic volume (up to 2,300 vehicles per hour), and different levels of left-turn traffic, access point densities, percent heavy vehicles, and bus stop activities (e.g., bus dwell times and headways). An investigation of signalized side-street delays was also completed, and the average arterial travel speed impacts of this type of conversion during non-peak-hours compared. It has been found that in some cases a four-lane undivided to three-lane conversion can improve roadway safety with only a small reduction in operations. Key words: safety, simulation, and operations.

Key words: arterials—CORSIM—lane conversion

INTRODUCTION

The conversion of four-lane undivided roadways to three lanes has occurred throughout the United States (See Figure 1). This is true despite the fact that the safety and operational benefits of this type of cross sectional conversion are not as clearly understood as roadway widening. In April 2001, the "Guidelines for the Conversion of Urban Four-Lane Undivided Roadways to Three-Lane Two-Way Left-Turn Lane Facilities" were produced for the Iowa Department of Transportation (IaDOT) to begin addressing this knowledge gap (*1*). The IaDOT guidelines are available at www.ctre.iastate.edu/pubs/ trafficsafety.htm, and include a series of evaluative questions and chapters on past research, case study results, simulation of comparable four-lane undivided and three-lane operations, and feasibility determination factors (*1*). The operational suggestions in this guideline document were supported and/or extended by results from case study applications and CORridor SIMulation (CORSIM) sensitivity analyses (*2*, *3*).

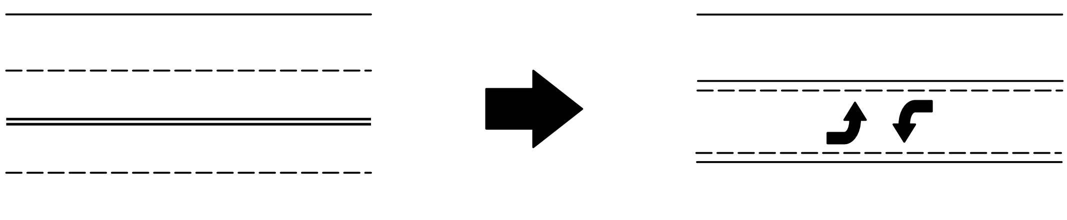

FIGURE 1. Four-lane Undivided to Three-Lane Conversion

CASE STUDY LOCATIONS AND RESULTS

Several references were used to create the list of case study locations in the IaDOT guidelines (*4*, *5*, *6*, *7*, *8*, *9*, *10*). In addition, a number of other conversions have been identified in Georgia, Washington, and Florida since the guideline publication. The list of guideline case study locations, along with their before-and-after operational and safety observations, are shown in Table 1. The data for these case study locations, except Sioux Center, Iowa, were all collected by others. Conversions are also known to have occurred in Alaska, Colorado, Pennsylvania, Michigan, Oregon, Massachusetts, Maryland, and Texas.

The thirteen roadway conversions in Table 1 had average daily traffic (ADT) volumes of 8,400 to 14,000 vehicles per day (vpd) in Iowa ,and 9,200 to 24,000 vpd elsewhere. The reviewed case study conversions appeared to result in a reduction of average or 85th percentile speeds (typically less than 5 miles per hour), and a relatively dramatic reduction in excessive speeding (a 60 to 70 percent reduction in the number of vehicles traveling 5 miles per hour faster than the posted speed limit was measured in two cases). Percent reductions in total crashes ranged from 17 to 62 percent for the case studies listed. However, Huang, et al. will present information at the Transportation Research Board Urban Street Symposium in July 2003 that took a more statistically valid approach to the evaluation of conversion safety and found the percent of total crashes occurring after a conversion was only about 6 percent lower than that of comparison sites (*11*). Additional analysis by Huang, et al. that also controlled for factors like volume and study period showed no impact due to the difference in cross section, and no significant difference in crash severity and crash type "before" and "after" this type of conversion (*11*).

FEASIBILITY DETERMINATION FACTORS

Four-lane undivided to three-lane conversions should only be considered (i.e., compared to other alternatives) at locations where it might be a feasible option. The guidelines identify and discuss more than 20 feasibility determination factors (*1*). These factors are described in this paper, but should not be considered exhaustive. Questions were also suggested in the guidelines to assist in the evaluation of each factor. The existing and expected (i.e., design period) status of the following factors should be evaluated.

ROADWAY FUNCTION AND ENVIRONMENT

The function of a roadway is defined by its amount of vehicular access and mobility activity. The objective of any design change should be to match the roadway environment with the actual roadway function. Turning volumes and/or vehicles patterns, for example, can produce a four-lane undivided cross section that actually operates as a "defacto" three-lane roadway (i.e., most of the through flow is in the outside lane, and the inside lane is used almost exclusively by turning traffic) (See Figure 2). The existing and intended function of the candidate roadway should be thoroughly understood.

OVERALL TRAFFIC VOLUME AND LEVEL OF SERVICE

One argument for widening two-lane undivided roadways to four lanes was that it would serve more through traffic. Many urban four-lane undivided roadways operate both efficiently and safely in this manner, but the existing and/or design period traffic flow capabilities of a four-lane undivided and a three-lane cross section need to be compared for conversion feasibility. One basic measure of comparison is the magnitude of existing and forecast ADT and peak-hour volumes the cross sections appear to be capable of serving. The ADT of the case studies in Table 1 ranged from 8,500 to 24,000 vpd, and according to the American Association of State Highway Transportation Officials (AASHTO) the peak-hour volumes along this type of roadway typically represent 8 to 12 percent of their ADT (*12*). For an ADT of 8,500 to 24,000 vpd these percentages represent a bidirectional peak-hour volume of 680 to 2,880 vehicles. However, there have been "successful" conversions in the United States along roadways with much higher daily volumes than those studied.

TABLE 1. Case Study Analysis Results (*4, 5, 6, 7, 8, 9, 10)***

Location	APPROX. ADT	Safety	OPERATIONS
MONTANA			
Billings – 17th Street West	9,200-10,000	62 percent total crash reduction (20 months of data)	No Notable Decrease**
Helena – U.S. 12	18,000	Improved**	No Notable Decrease**
MINNESOTA			
Duluth – 21st Avenue East	17,000	Improved**	No Notable Decrease**
Ramsey County – Rice Street	18,700 Before 16,400 After	28 percent total crash reduction (3 years of data)	NA
IOWA			
Storm Lake – Flindt Drive	8,500	Improved**	No Notable Decrease**
Muscatine – Clay Street	8,400	Improved**	NA
Osceola – U.S. 34	11,000	Improved**	No Notable Decrease**
Sioux Center – U.S. 75	14,500	57 percent total crash reduction (1 year of data)	Overall travel speed decreased from 28-29 mph to 21 mph, and free-flow speed from 35 mph to 32 mph. There was a 70 Percent decrease in speeds greater than 5 mph over the posted speed limit.
Blue Grass	9,200-10,600	NA	85th percentile speed reduction up to 4 mph (two locations increased 1 to 2 mph in one direction). The change in percent vehicles speeding depended upon location and direction (see discussion).
Des Moines (Note: This was a conversion from multiple cross sections to a three-lane)	14,000	NA	Average travel speed increased from 21 to 25 mph
CALIFORNIA			
Oakland – High Street	22,000-24,000	17 percent in total crash reduction (1 year of data)	No notable change in vehicle speed
San Leandro – East 14th Street	16,000-19,300 Before 14,000-19,300 After	52 percent in total crash reduction (2 years of data)	Maximum of 3 to 4 mph spot speed reduction

WASHINGTON			
Seattle – Nine Locations	9,400-19,400 Before 9,800-20,300 After	34 percent avg. total crash reduction (1 year of data)	NA

*ADT = Average daily traffic. NA = Not Available. Safety data duration is for before/after conversion.
**Summarized results based on anecdotal information.

FIGURE 2. Four-lane Undivided Roadway/Intersection Operating as a "defacto" Three-Lane Cross Section

The sensitivity analyses completed as part of the IaDOT guidelines project included most of the volumes in the case studies (See Table 2). A simplified corridor was used in these analyses and is shown in Figure 3. The analysis compared average arterial travel speed, arterial LOS, and intersection LOS of similar four-lane undivided and three-lane roadways with the peak-hour volumes shown in Table 2 (*1*, *2*). The analysis found the smallest difference in average arterial travel speed for the two cross sections occurred at a peak-hour volume of 750 vphpd. However, the simulated difference between the average arterial travel speeds along the two cross section was always less then 4 miles per hour (mph), and differences greater than 1.9 mph were only experienced at 1,000 vphpd (*1*, *2*). The arterial and signalized intersection LOS were generally the same for each cross section except when the 875 and/or 1000 vphpd (depending an the arterial classification assumed) were simulated.

Additional simulations were also done with the same corridor for even larger volumes (*3*). The difference in operations for the four- and three-lane corridors with volumes up to 1,250 vphpd (assuming 20 access points per mile per side and a total access point turning volume equal to 25 percent of the mainline traffic) were considered, but the CORSIM results for volumes above 1,150 vphpd were not reliable and dropped from further consideration (*3*). At volumes of 1,000 vphpd or higher the reduction in arterial speed along the four-lane undivided roadway was larger than the three-lane roadway, but 75 percent of the three-lane arterial speed reduction occurred between 1,000 and 1,050 vphpd (*3*).

TABLE 2. Sensitivity Analysis Factors

Characteristic	Values Evaluated
Total Entering Volume (vehicles per hour per direction)	500, 750, 875, and 1,000
Access Point Left-Turn Volume (percent of through volume)*	10, 20, and 30
Access Point Density (points per mile per side)	0, 10, 20, 30, 40, and 50

*Left-turn volumes are evenly distributed among the access points.

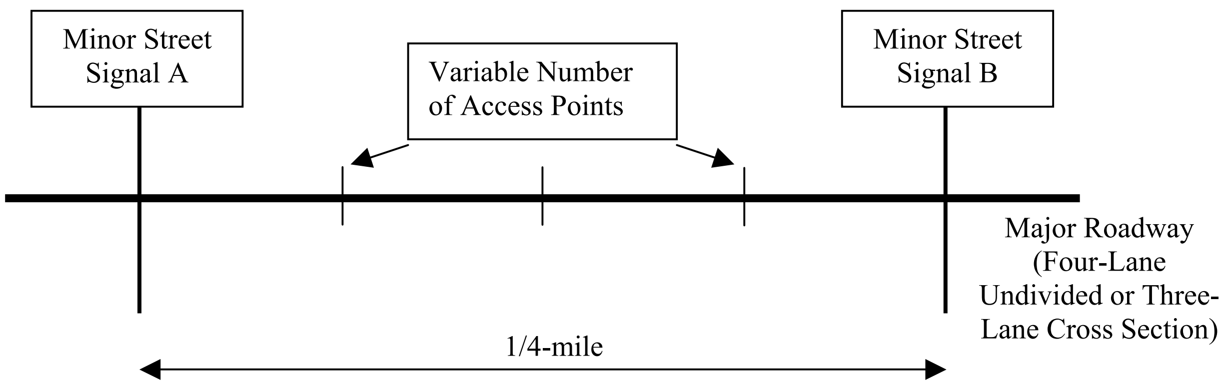

FIGURE 3. Simulated Case Study Corridor

Turn Volumes and Patterns

The sensitivity analyses completed as part of the guidelines project compared the simulated average arterial travel speed and LOS for four-lane undivided and three-lane roadways with a range of access point left-turn volumes and densities (See Table 2 and Figure 3). The analyses results indicated that, given optimized signal timing, the difference between the average arterial travel speeds for the two cross sections decreased as access point left-turn volumes increased, and as access point density increased (*1*, *2*). Arterial LOS for the two cross sections were only different at the highest access point left-turn volume and densities considered. In addition, average arterial travel speeds decreased as access point left-turn volumes increased along the four-lane undivided roadways, but increased along the three-lane roadways. However, the overall range of simulated average arterial travel speed differences for all the access point densities along the corridor considered was only 0.6 mph (*1, 2*).

Frequent-Stop and/or Slow-Moving Vehicles

The amount of frequent-stop and/or slow-moving traffic (e.g., agricultural vehicles, school bus student drop-off/pick-up, mail delivery vehicles, and buggies) that occurs along a roadway also needs to be considered. These types of vehicles have a greater impact on the operation of a three-lane roadway than a four-lane undivided cross section. An extension of the simulations completed for the IaDOT guidelines considered different percentages of heavy vehicles and bus activity were along the corridor shown in Figure 3 (*3*). For a main roadway volume of 750 vphpd (and an access density of 20 access points per mile per side) simulations were completed for heavy vehicles percentages from 0 to 30 percent. Not surprisingly, the results showed a reduction in average arterial travel speed along the three-lane roadway that was three times more than the four-lane undivided roadway reduction (*3*). Approximately 50 percent of the speed reduction, however, occurred at and above 20 percent heavy vehicles (*3*). The impacts of 1 and 2 bus stops (with buses arriving at 5 to 60 minutes headways and 30 to 60 second dwell times) were also simulated. Of course, the impact of the bus activities on average arterial travel speed was greater along the three-lane roadway, but the traffic volumes and corridor characteristics considered in this research did not allow more a more detailed conclusion (*3*).

Weaving, Speed, and Queues

The weaving, speed, and queuing of vehicles on a four-lane undivided roadway are different than those of a three-lane roadway. However, the change in some of these factors can be small if a four-lane undivided roadway is already operating as a "defacto" three-lane roadway (See Figure 2). Clearly, weaving or lane changing (other than vehicles entering the TWLTL) should not occur along a three-lane roadway. If this does occur (i.e., passing in the TWLTL), education and/or enforcement measures may be necessary.

The need to "calm" or reduce vehicle speeds is also often cited as a reason for converting a four-lane undivided roadway to a three-lane cross section. The case study results show that average vehicle speed and speed variability usually do decrease. Overall, the typical reduction in 85th percentile or average speed along the case study roadway segments was 3 to 5 miles per hour (mph). The sensitivity analysis output supported the case study results, and showed that the vehicle speed differences they experienced (i.e., 3 to 5 mph) are possible for a large range of total entering traffic, access point left-turn volumes, and access point densities.

Cumulative off-peak impacts on travel speed are also sometimes a concern, and a simulation of hourly volumes along the corridor in Figure 3 revealed that the largest difference in average arterial travel speed occurs during off-peak travel times (when the two cross sections would have the greatest difference in their general operation) (*3*). If appropriate, the cumulative average off-peak speed impacts during a typical day should be something to consider when determining the feasibility of a four-lane undivided to three- lane conversion.

The conversion of a four-lane undivided roadway to a three-lane cross section includes geometric changes that impact through-vehicle delay and queues. For example, through-vehicle delay related to left-turn traffic can be expected to decrease, but the reduction in through lanes may result in a larger increase of peak-hour segment and/or intersection through vehicle delay. One concern has been the potential increase in delay for minor roadway vehicles. A conversion may have the potential to decrease the number of acceptable gaps within the traffic flow (due to a general reduction in through lanes), and this should be considered in the determination of four-lane undivided to three-lane cross section conversion feasibility. Side street vehicle delay at the signalized intersections was considered in the extension of the IaDOT guidelines project, and the proportion of the total delay experienced by minor street vehicles was found to increase dramatically with main street volume if the number of signal phases was limited to two and the cycle lengths considered were also limited (*3*). Additional analysis is needed to evaluate the impacts of individual roadways.

Crash Type and Patterns

Based on past data and the case study results it is typically expected that a roadway with a three-lane cross section will have a lower crash frequency or rate than a similar four-lane undivided roadway. In fact, data from Minnesota indicate that three-lane roadways have a crash rate 27 percent lower than the rate for four-lane undivided roadways (*13*). The case study results also showed similar or higher decreases in total crashes, and these results were confirmed by Hummer (*14*). A more statistically robust analysis by Huang, et al., however, showed less of an safety improvement impact due to these conversions (*4*). These results were discussed in the case study section of this paper. The expected increase in safety that can apparently occur may be the result of the reduction in speed and speed variability observed along the roadway, a decrease in the number of conflict points between vehicles, and/or improved sight distance for the major-street left-turn vehicles.

Pedestrian and Bike Activity

The conversion of an urban four-lane undivided roadway to a three-lane cross section may have an impact on pedestrian and bike activity. These users (pedestrians and bicyclists) are not served well by urban four-lane undivided roadways, and anecdotal case study results appear to support the conclusion that pedestrians, bicyclists, and adjacent landowners typically prefer the corridor environment of a three-lane cross section. Bicycle lanes are also sometimes added when the conversion occurs.

Right-of-Way Availability, Cost, and Acquisition Impacts

Many urban four-lane undivided roadways have a limited amount of right-of-way. If a roadway in this environment is widened (through the addition of a TWLTL or raised median) the cost and acquisition impacts could be significant. Typically the conversion of a four-lane undivided roadway to a three-lane cross section does not require any additional right-of-way or the removal of trees and buildings. The existing curb-to-curb width is simply reallocated with pavement marking from four through lanes to two through lanes and a TWLTL (possibly including bicycle lanes).

General Characteristics

Parallel Roadways. The structure of the surrounding roadway system should be considered when evaluating the feasibility of a four-lane undivided to three-lane cross section conversion. The potential decrease in mobility (i.e., average arterial travel speed) that might occur after a conversion may induce some drivers to choose a different route. Parallel roadways in close proximity to the converted corridor are candidates for this alternative route. Planning level traffic flow analysis may be necessary.

Offset Minor Street Intersections

Minor street offset intersections along an arterial can be a poor design characteristic. The existence of offset minor streets or driveways with high turning and/or through volumes should be considered in the conversion feasibility determination. Overlapping volumes of heavily used offset minor streets or driveways can produce a situation where turning vehicles slow and possibly stop within the through lanes of a three-lane roadway. This is a situation that should be avoided.

Parallel Parking, Corner Radii, and At-Grade Railroad Crossings

Other roadway characteristics that should be considered include the amount and usage of the parallel parking spaces along the corridor, the length of each corner radii, and the impact of any at-grade railroad crossings. Parallel parking occurs along four-lane undivided and three-lane roadways. One parallel parking striping design that can reduce the impact of parking usage on the operations of the through lane traffic includes pairs of parking spaces that are spaced to allow parking movements to occur quickly. This type of design, however, will reduce the number of parking spaces available. Corner radii geometry and/or corner design impact the ability and speed of vehicle entering/exiting the minor cross street or driveway. The movements of these types of turns may be more important along a three-lane roadway, and radii or turn-lane improvements should be done on an as-needed basis. Finally, the impact of at-grade railroad crossings should be considered. In most cases, the queues at a railroad crossing can be expected to approximately double when a roadway is converted to a three-lane cross section. Drivers on a four-lane undivided roadway that approach a railroad crossing occupied by a train will typically choose the lane with the shortest queue (i.e., use both lanes evenly). The three-lane cross section does not provide this option.

RECOMMENDATIONS

The feasibility of replacing an urban four-lane undivided roadway with a three-lane cross section should be considered on a case-by-case basis. An investigation of community goals for the roadway and a comparison of the expected before-and-after safety and operational impacts to what is locally acceptable must be completed.

The existing and expected (e.g., design period) characteristics of a number of factors should be investigated further in future research and when considering the design period feasibility of an urban four-lane undivided to three-lane cross section conversion. These factors include:

- Roadway function and environment;
- Overall traffic volume and level of service;
- Turning volumes and patterns;
- Frequent-stop and/or slow-moving vehicles;
- Weaving, speed, and queues;
- Crash types and patterns;
- Pedestrian and bike activity;
- Right-of-way availability, cost, and acquisition impacts; and
- General characteristics: parallel roadways, offset minor street intersections, parallel parking, corner radii, and at-grade railroad crossings.

The results of the work summarized suggest that urban four-lane undivided to three-lane cross section conversions along roadways with peak-hour volumes less than 750 vphpd may experience few operational impacts, but that more caution should be exercised when the roadway has a peak-hour volume between 750 and 875 vphpd. At and above 875 vphpd, the simulations indicated a more severe reduction in average arterial travel speed and greater operational concerns.

The sensitivity of the results appear to indicate that an urban four-lane undivided to three-lane conversion will be most successful if the factors that define the roadway environment remain stable during the design period (e.g., traffic volumes won't increase dramatically), and if the current four-lane undivided roadway is already operating as a "defacto" three-lane roadway.

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Deer-Vehicle Crash Countermeasures Effectiveness Research Review

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ABSTRACT

During the last two years an extensive review of deer-vehicle crash (DVC) countermeasure documentation has been completed. Research and/or documents related to 16 different countermeasures were reviewed and are currently being summarized in a DVC Countermeasures Toolbox. An example of some of the countermeasure research reviewed includes documents related to deer whistles, warning signs and technologies, and roadside reflectors. The results of the ongoing countermeasures review contained in the toolbox are summarized in this paper. The toolbox is one of the first products from the Deer-Vehicle Crash Information Clearinghouse (DVCIC). The DVCIC is funded by the Wisconsin Department of Transportation and five states in the Upper Midwest (i.e., Michigan, Minnesota, Illinois, Iowa, and Wisconsin) are currently involved with the project. The final version of the DVC Countermeasures Toolbox, and the results of a DVC data management and characteristics survey should be completed by the end of 2003. These products, along with suggested standards for DVC countermeasure research and a regional data summary, are expected to be resources for transportation decision-makers.

Key words: countermeasures—deer-vehicle crashes—research review

INTRODUCTION

It has been estimated that over 1.5 million deer-vehicle crashes (DVCs) occur each year in the United States, but less than half of them are reported (*1*). In Wisconsin, approximately one in seven reported crashes are DVCs. A summary of the reported DVC and/or animal-vehicle crashes for the Upper Midwest region is show in Table 1.

In July 2001, the Wisconsin Department of Transportation (WisDOT) initiated a regional DVC Information Clearinghouse (DVCIC). Five states in the Upper Midwest (i.e., Michigan, Minnesota, Illinois, Iowa, and Wisconsin) are involved with this project. During the last two years the clearinghouse staff has combed through hundreds of documents that summarize the current state of the knowledge related to DVC countermeasure effectiveness. The creation of this DVC Countermeasures Toolbox is ongoing and should be finalized this year. The goal is to provide a resource to transportation professionals that can assist them with their decisions related to the reduction of DVCs.

This paper summarizes the preliminary results of the toolbox document review that were available in June 2003. Additional reviews will be completed between June 2003 and the time of the Mid-Continent Transportation Symposium in August 2003. The results of the ongoing reviews will be presented at the symposium. Draft and final results of the reviews, as they become available, are also available at www.deercrash.com.

THE DVC COUNTERMEASURE TOOLBOX

The development of a DVC Countermeasures Toolbox is an ongoing task of the DVCIC staff. The objective of the toolbox is to provide information to decision-makers about the current state of the knowledge related to the effectiveness of DVC-reduction measures. The focus of the investigation is documented and published research, if available, about the relationship of 16 DVC countermeasures and their direct DVC impact. The characteristics of each measure that may impact its DVC-reduction effectiveness are also being identified. However, documentation about the "effectiveness" of the DVC countermeasures also ranges from the anecdotal to a comparably few peer-reviewed research journal publications. DVC countermeasure studies that are poorly documented, questionably designed, and/or invalid or unrepeatable in their statistical validity are also common. This situation is most likely the result of the variability, diversity, and complexity of the problem.

TABLE 1. 2000/01Upper Midwest Deer-Vehicle Crashes

State	Pre-Hunt Numbers in Deer Herd	Deer-Vehicle Crashes	Deaths	Injuries	Vehicle Damage**
Michigan	**1,800,000**	**67,000**	**11**	**2,100**	**$114 mil**
Wisconsin	**1,500,000**	**19,900**	**9**	**800**	**$34 mil**
Minnesota*	**960,000**	**19,000**	**2**	**450**	**$32 mil**
Illinois	**750,000 (est.)**	**22,900**	**5**	**920**	**$39 mil**
Iowa*	**210,000**	**7,800**	**3**	**600**	**$13 mil**
Total	**5,220,000**	**136,600**	**30**	**4,870**	**$232 mil**

*2000 Reported deer-vehicle or animal-vehicle crashes.
**Damage estimate assumes $1,700 property damage per reported crash.

The toolbox attempts to summarize the current state of the knowledge about the DVC-reduction capabilities of the countermeasures listed below. Those countermeasures in the list that are in italics are currently being summarized. The remainder of the summaries are in draft form. The reader is referred to the webpage (www.deercrash.com) for these draft summaries, and a complete listing of the references used.

- Noise/sound/whistle devices;
- Reflectors/mirrors;
- Deer crossing signs;
- Intercept feeding;
- Speed limit reduction;
- Highway lighting;
- Repellents;
- Deer flagging models;
- Deicing salt alternatives;
- In-vehicle technologies;
- Wildlife grade separations and crossings (e.g., overpasses, underpasses, and at-grade);
- Vegetation/roadside management;

- Hunting or herd management;
- Fences/barriers/one-way gates;
- Highway planning; and
- Public education/awareness.

NOISE/SOUND/WHISTLE DEVICES

The DVC reduction effectiveness of air-activated deer whistles has been investigated through the use of non-scientific before-and-after studies and some documented research into the hearing capabilities of deer. In general, the relatively poor design (e.g., sample size) and/or documentation of the before-and-after studies have produced conflicting results. No conclusions can be drawn from the studies reviewed as a whole, and better designs and documentation are recommended for future studies of this nature.

A small amount of documented/published research has been completed in the area of deer auditory capabilities and their reaction to air-activated whistles. In fact, a summary of a recently published study, and possibly the documentation of a county-level study in California, will be added to the upcoming final version of the toolbox. For the most part, however, it has been found that deer hear in the same range of sound as humans. It has also generally been concluded in the studies reviewed that deer did not react to vehicle-mounted air-activated deer whistles. The ability of whistles to produce the advertised level of sound at an adequate distance has also been questioned. Additional scientifically defined research about the effectiveness of air-activated deer whistles and similar non-air-activated devices is recommended.

REFLECTORS/MIRRORS

The reflector/mirror studies and literature reviewed for the toolbox were segmented into four categories. Past reflector/mirror research typically used either a cover/uncover, before-and-after, or control/treatment study approach to evaluate their impact. Researchers have also either observed deer movements as they evaluated the impact of roadside reflectors/mirrors on deer roadkill and/or DVCs or specifically considered deer behavior toward reflected light. Many of the studies summarized (which represent a sample of the many documents available), whether they focused on deer roadkill and DVC impacts or deer behavior, had conflicting results. Overall, 5 of the 10 studies summarized for the toolbox had conclusions that indicted roadside reflectors did not appear to impact deer roadkill or DVCs, and 2 of the 10 concluded that they did. Three of the 10 studies summarized appeared to reach inconclusive or mixed results. Most of the studies that evaluated deer behavior (many dealing with captive deer) were also inconclusive or concluded that the deer either did not appear to react to the light from the reflectors and/or quickly became habituated to the light. Unfortunately, the experimental designs and details of all the studies varied (their details are included in the draft toolbox at www.deercrash.com). Speculative and anecdotal information that exists about roadside reflector/mirror DVC-reduction effectiveness was not included in the summary.

At this point in time it is difficult to conclude the roadkill- or DVC-reduction effectiveness of roadside reflector/mirror devices due to the conflicting results of the studies summarized. It is recommended that the completion of a definitive roadside reflector/mirror DVC-reduction effectiveness study be considered. A well-designed widespread long-term statistically valid study of comparable and well-defined roadside reflector treatment and control roadway segments (with consideration given to local deer travel patterns) is suggested.

DEER CROSSING SIGNS

Two studies were summarized that implied there were speed reduction impacts related to the lighted deer crossing sign design improvements they were evaluating. However, the outcome of a more in-depth study by some of the same researchers of a lighted and animated deer crossing sign did not appear to indicate that the resultant vehicle speed reduction resulted in a reduction of the number of deer roadkill (i.e., DVCs). Unfortunately, these study results are based on only 15 weeks of data and the variability in DVCs and the factors that impact their occurrence also limits their validity and transferability. It is proposed that additional and more long-term research be completed to support or refute the speed- and DVC-reduction impacts of existing and proposed improvements to deer crossing warning signs.

A number of systems that combine dynamic signs and sensors are also being considered or have been installed (e.g., Montana, Indiana, Minnesota, and Wyoming). Several of these systems are briefly described in draft toolbox at www.deercrash.com. The recent development of these systems requires an initial evaluation of their activation reliability. One key to the successful application of these systems is the minimization of false activations. The operation and effectiveness of some of the systems described in the draft toolbox are currently being studied, but only the Nugget Canyon, Wyoming system analysis appears to have been documented at this time. The researchers doing the evaluation concluded that when the system worked properly it produced a small, but statistically significant, reduction in average vehicle speeds. However, they did not believe the observed average vehicle speed reduction would reduce DVCs. Reductions in average vehicle speeds were also found when the lights on the signs were continuously flashed and/or a deer decoy was introduced on the roadside. In fact, the largest average vehicle speed reduction calculated was when the lights were flashing and the deer decoy was present.

INTERCEPT FEEDING

Intercept feeding involves the provision of feeding stations outside the roadway area. The objective is to divert animals to the feeding areas before they cross the roadway. One study was found that attempted to evaluate the impact of this DVC countermeasure. The researchers generally concluded that intercept feeding might be an effective short-term mitigation measure that could reduce DVCs by 50 percent or less. However, the study results actually described appeared to be contradictory. In addition, there was no documentation of the number of DVCs that occurred along the roadway segments evaluated before the intercept feeding stations were in operation, and it was acknowledged by the researchers that the amount of deer roadkill counted along the segments were not proportional to the deer population near each segment. In general, the study investigators were of the opinion that the potential for a short-term reduction in DVC of 50 percent or less was not sufficient enough to justify the amount of work and funding necessary for the implementation of intercept feeding. It was suggested that intercept feeding might be combined with other countermeasures to increase its effectiveness. Two problems that might occur with the implementation of this countermeasure are that deer may become dependent on the food supply and more deer than typical might be drawn to the general vicinity of the roadway and the area. The appearance of chronic wasting disease in Wisconsin also makes this approach undesirable.

SPEED LIMIT REDUCTION

Two studies that evaluated speed limit reduction as a potential DVC countermeasure were reviewed for the toolbox. In both cases the researchers suggested that there was a relationship between animal-vehicle collisions and posted speed limits. In certain instances, but not all, their research results appear to show a less then expected number of animal-vehicle collisions along roadway segments with lower posted speed limits. To reach this conclusion, one study statistically compared the proportion of roadway mileage with a particular posted speed limit to the proportion of animals killed along those segments. The other study compared the frequency and rate per roadway length of animal-vehicle collisions before and after a posted speed limit changes. No studies were found that focused on the number of white-tailed DVCs and posted speed limit.

There are several limitations to the research results presented in this summary. Overall, like the analysis of many other animal-vehicle crash countermeasures, the two studies summarized do not address, and/or attempt to control for, a number of factors that could impact the validity and usefulness of their conclusions. For example, neither study quantitatively considered the increase in traffic volume or adjacent animal population variability along the segments considered. The comparison of the proportion of animal-vehicle collisions to the proportion of roadway mileage also assumes a uniform distribution of animal population, and also tends to ignore any positive or negative correlations that might exist between roadway design, topography, posted speed limit, operating speed, and animal habitat. Effectively determining and defining a relationship (if any) between reduced posted speed limits (or operating speeds) and the number of animal-vehicle collisions along a roadway segment will require additional research studies that attempt to address, control for, and/or quantify the impact and potential interaction of these and other factors.

One of the studies summarized also concluded that the choice of vehicle operating speed appeared to be primarily impacted by the roadway and roadside design features (versus posted speed limit). This is a conclusion that is generally accepted in the transportation profession, and primarily supports the idea that a reduction in posted speed limit that is not considered reasonable by the driving public will generally be ignored (without significant enforcement presence). This type of situation has been shown to increase the general possibility of a crash (not DVCs) between two vehicles along a roadway because some drivers will slow and others will not.

HIGHWAY LIGHTING

One study was found that attempted to directly relate the existence of roadway lighting to a reduction in DVCs. This study also investigated the changes in deer crossing patterns and average vehicle speeds that might occur with the addition of lighting. The study researchers concluded that the addition of lighting did not appear to have an impact on DVCs, deer crossing patterns, or average vehicle speeds. However, they made this conclusion despite the fact that a reduction in the number of crashes per deer crossing appeared to decrease by about 18 percent with the addition of lighting along the roadway test segment. It is assumed, but it was not documented, that the investigators believed that this reduction was within the normal variability of the data evaluated. The addition of a taxidermy-mounted full-size deer in the emergency lane of the roadway segment did produce a reduction in average speed of about 8 mph when the lights were activated. However, not enough speed data were available to validate these results. Additional research may be appropriate to evaluate the focused effectiveness of lighting as a DVC-reduction tool (versus a speed reduction tool).

REPELLENTS

A large number of studies, with varied approaches, have attempted to evaluate the effectiveness of numerous repellents on the feeding patterns of several different types of captive animals. The studies summarized in the draft toolbox investigated repellent impacts on white-tailed deer, mule deer, caribou, and elk. No studies were found that documented an attempt to test repellent effectiveness on deterring wild animals from approaching a roadside and roadway to feed.

Some of the factors evaluated in the studies summarized include type and number of repellents (e.g., predator urine, brand, odor, taste, etc.), status or application of repellent (e.g., spray, paste, etc.), concentration of repellent, animal hunger level, food type, and amount of rain or water occurrence after repellent application. All of the studies reviewed did find some type of feeding reduction with one or more of the repellents considered, but the variability and/or non-repeatability of the studies makes a direct comparison of their results difficult. One paper that was reviewed did attempt to discover some overall trends in the numerous repellent studies available. The repellent effectiveness results of twelve studies were ranked (i.e., 0 = ineffective to 4 = highly effective) and analyzed by two experts. It was concluded that Big Game Repellent™ and predator odors were typically the most effective of all the repellents. In addition, no significant difference was found in the ranking of area (i.e., primarily odor) and contact (i.e., spray or dust) repellents, or in the reactions to repellents between deer and elk (although white-tailed and mule deer appeared to react differently to predator odor).   These results may be useful when choosing a repellent, but should also be used with the understanding that the comparison required a subjective, but expert, ranking to be completed.

The effective and economical application of repellents to potentially reduce roadside browsing of white-tailed deer would need to consider several factors. Some of these factors include how the repellent is applied, at what time intervals, cost, animal habituation, and the locations to which is it applied.   Like most of the other countermeasures already summarized, the application of repellents as a DVC reduction tool would most likely need to be focused on “high” DVC locations rather than widespread. However, white-tailed deer (or other animals) may also just shift their browsing location if repellents are not applied in a widespread manner. The application of repellents in combination with other DVC reduction tools at “high” crash locations might be most appropriate.

DEER FLAGGING MODELS

Some experts believe that white-tailed deer raise their tails to expose their white undersurface as a warning communication between their species. Deer flagging involves the use of wood silhouettes of models of this warning stance on the roadside to warn deer away from the roadway. Only one study was found that investigated this approach to DVC reduction. None of the experiments in the study appeared to yield conclusive results that the addition of flagging models reduced the number of white-tailed deer that were observed and/or crossed the study roadway right-of-way. In some cases fewer deer were seen along the treatment segments than in the control segments, but in others the number of deer observed increased after the models were installed. The general fluctuations in deer movements and the variability in data observation approaches (and time periods) also appeared to confound attempts, at least in some of the experiments, to connect deer behavior to the presence or absence of the flagging models. The researchers involved with the study generally concluded that they had failed to demonstrate that the use of deer flagging models was an effective method of reducing the number of deer observed along the highway right-of-way. They did not recommend their use. A similar and well-designed study in the future might be considered to validate or refute the results of this study.

DEICING SALT ALTERNATIVES

It has been speculated that the use of salt on the roadway for winter maintenance purposes has increased the number of DVCs. However, no research on the impact of salt or salt alternative use on DVCs was found. In the past, suggestions and/or studies of sodium chloride and its alternatives have typically focused on the water-related environmental impacts of these chemicals (e.g., surface runoff) rather than their potential DVC impact. A study that focuses on this subject is needed. Some areas in the country do not allow or have restricted the use of roadway salt and could be useful in this analysis. Any evaluation of roadway salt and/or salt alternatives on DVCs, however, would also need to consider their effectiveness at clearing the roadway pavement (which increases general safety) and their other benefits and costs.

IN-VEHICLE TECHNOLOGIES

No published studies were found that evaluated the DVC-reduction capabilities of in-vehicle sensor or vision technologies. However, the application of these technologies in the general vehicle population is very recent and the ability to do this type of large-scale study probably has not been possible. An evaluation of the DVC reduction capabilities of these technologies for a wide range of drivers would be of interest. Their potential to reduce the number of DVCs (if properly used) appears to exist. Currently, the cost of in-vehicle vision systems is high, but it may decrease if demand and competition increases.

ONGOING REVIEWS

A preliminary scan of several documents related to wildlife grade separations/crossings and fencing/barriers/one-way gates reveals that these two measures have been widely implemented, and appear to have been studied to a larger extent than some of the other countermeasures previously discussed. These two measures are also commonly and appropriately implemented together. Determining the DVC-reduction effectiveness of one or the other may be difficult. In addition, the effectiveness of wildlife separations/crossings is also often measured by whether or not it is used by the animals for which it is built rather than the change in DVCs that might have occurred. Studies that focus on the effectiveness of different deer fencing heights have also been documented. It appears that these studies sometimes have conflicting results. Several studies focus on the use of fencing to protect of valuable crops. A fencing height of 8 to 10 feet is often suggested, but documentation about what percentage of white-tailed deer are removed from the right-of-way due to different fencing heights is still being pursued. Key factors related to deer fencing installation include location, length, height, surrounding topography, its relationship with grade separations (to allow migration), and the need for continuous maintenance.

The documentation available about the potential DVC-reduction impacts of vegetation/roadside management, hunting and herd management, highway planning, and public education/awareness on DVCs need further evaluation. Roadside vegetation management documents typically do not discuss the potential that policies or plantings have to attract white-tailed deer. Most of the documents found focus on vegetation choices in gardens to avoid deer damage. The content o these documents may have some limited transferability to roadside management and choices. As expected, except on a small scale, the impact of hunting and heard management policies (and their changes) and/or public awareness/education campaigns on DVCs is difficult to prove. Overall, it has been shown in several studies that there is a relationship between the trends in deer populations and the reported number of DVCs, but the cause and effect relationship between these two factors needs additional investigation. The existence of a public education program in the area of roadway safety is also a key component (along with engineering and enforcement). Highway planning that focuses on locating roadways to minimize their impact on animal migration patterns is also a good objective. The use of this DVC countermeasure would most likely be focused on new roadway construction or complete reconstruction, but it might also be used to identify “segments of DVC concern” along existing roadways.

CONCLUSIONS

The DVC countermeasures toolbox is being created to provide the level of information to transportation professionals that allow them to make reasonable and knowledgeable DVC countermeasure application choices and decisions. The objective is to provide more than an annotated bibliography of the documents available, and allow the reader to reach their own conclusions about the current state of the knowledge related to the countermeasures being considered.

A review of the information in this paper and the more detailed summaries at www.deercrash.com lead to the following conclusions. One, there are few, if any, definitive “research-oriented” countermeasure effectiveness studies. This situation has probably occurred because of the complexity of the DVC problem (e.g., the variability and/or validity of the data), and a lack of resources (e.g., time and funding). Two, a common solution to the DVC problem is unlikely, and it is expected that one or more countermeasures at select locations will be the key to DVC reduction. Three, the countermeasures used should be matched with roadway locations or segments that have the greatest potential to produce a DVC reduction and not just shift the problem. Four, additional and definitive research projects must be designed for many of the DVC countermeasures considered. These studies will most likely need to be broad in range (e.g., multi-state) and time (e.g., multi-year), and will require significant funding. It is suggested that these evaluations start with some of the measures that have been used in the United States and elsewhere for decades. Five, the proper completion of the activities above needs accurate DVC reporting by location, and easy access to that crash report information. As indicated previously, it is assumed that up to 50 percent of all DVCs may be unreported and undocumented.

DISCLAIMER

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 reflect those of the Wisconsin Department of Transportation or the Federal Highway Administration.

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Construction of Laboratory and Field Demonstration Modified Beam-in-Slab Bridges

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ABSTRACT

Repairing/replacing deficient bridges is a major challenge for transportation system managers and is magnified for Low Volume Road (LVR) systems where inadequate structures need replacing due to structural and functional deficiencies. To maximize limited replacement funds, some local governments employ in-house forces to reduce construction costs, however most lack the resources to construct traditional bridge systems.

The beam-in-slab bridge (BISB) system is a cost competitive alternative designed specifically for LVR systems. Consisting of W sections spaced 24 in. (640 mm) on centers and filled with concrete, the BISB system has been shown to be a sound replacement option, but however in some instances it is not structural efficient. To improve efficiency, composite action was obtained with an alternative shear connector (ASC); a transverse arch was utilized to distribute the wheel loads and reduce the self-weight. Both the section spacing and depth were increased resulting in the Modified Beam-in-Slab Bridge (MBISB) system.

A full-scale laboratory test bridge (L = 31 ft. (9.45 m), W = 20 ft. (6.1 m)) was constructed to quantify the construction process, load distribution and failure modes of the MBISB. Unique to the construction was the transverse arch between the longitudinal girders formed with custom rolled re-useable formwork and minimal deck reinforcement.

A MBISB demonstration bridge (L = 70 ft. (21.34 m), W = 32 ft. (9.75 m)) was designed and constructed without the use of specialized equipment in Tama County, Iowa. The bridge will be load tested in the summer of 2003 to compare its behavior with predicted values.

Key words: beam-in-slab bridge—composite action—low volume road bridge—replacement alternative

INTRODUCTION/PROBLEM STATEMENT

In Iowa, county governments are charged with maintaining and replacing bridge structures on the off system roads. Off system roads are those roads not cared for by Federal, State, or City forces; since Iowa is a rural state, a majority of the off system roads are also LVR (Low Volume Roads) with far less than 400 ADT (Average Daily Traffic). As reported by the National Bridge Inventory, approximately 30% of the 19,949 bridge structures found on Iowa's off system roads are either structural deficient or functionally obsolete *(1)*. County governments, and more specifically, county engineers are faced with the challenge of upgrading or replacing deficient structures. Due to limited resources and the costs associated with maintaining an aging and deteriorating bridge population, county engineers have expressed an interest in innovative methods to extend available replacement funds.

Many counties in Iowa employ full time bridge crews to maintain and repair deficient structures. However, due to the advanced levels of deterioration and functional obsolescence, replacement is sometimes the most cost effective solution. Constructing traditional bridges is sometimes beyond the capability of some county bridge crews; such designs contain features, such as Traffic Level Four (TL-4) barrier rails that are not required for LVR applications *(2)*. As a result, alternative bridge replacements specifically designed for a LVR application have been investigated.

While not being exposed to high traffic volumes, structures found on LVRs are subjected to heavy loads due to agricultural and off road equipment and must therefore still meet strength and serviceability conditions similar to on system structures. The Iowa State University Bridge Engineering Center, in conjunction with the Iowa Highway Research Board, and numerous Iowa county engineers for several years have been working to develop various alternative bridge replacement designs for LVR.

Beam-in-Slab Bridge System (BISB)

One of the more successful and widely used alternative replacement designs is the BISB system, a design originating in Benton County, Iowa in the mid-1970's. There are approximately 80+ of these structures in service today and more are being constructed, (Lyle Brehm, former Assistant Benton County Engineer, unpublished data). The original BISB design consists of simply supported W12 x 79 girders spaced 24 in. (610 mm) on center. Steel confining straps are welded to the bottom flanges at the longitudinal quarter points to provide confinement during the placement of the concrete. Plywood "floor" formwork rests on the top of the bottom flanges, leaving space between the web and the formwork allowing for the concrete to be in contact with the bottom flange. Concrete fills the void and is struck off even with the top flange completing the BISB system; a typical BISB cross-section is presented in Figure 1.

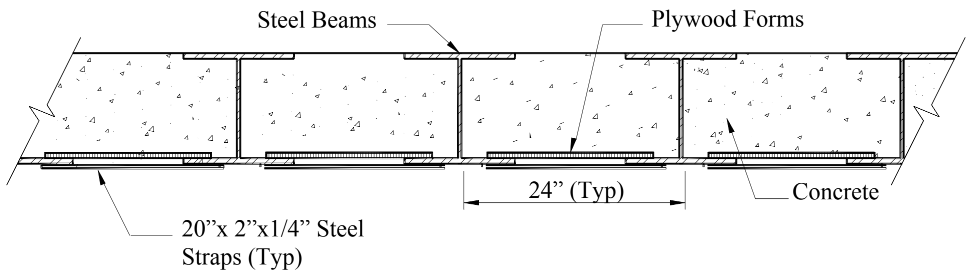


FIGURE 1. Typical Cross Section of BISB

The system is simple to construct with in house forces and does not require the use of specialized equipment. When comparing the costs of the BISB to more traditional systems, a savings of over 20% is possible (Tom Schoellen, Assistant Blackhawk County Engineer, unpublished data). The structural behavior and capacity of the BISB has been studied and verified to be sufficient for Iowa legal loads through both field and laboratory testing completed by Iowa State University researchers (*3*), (*4*).

In spite of the positive qualities of the original BISB system, the design's applicability is limited due to several structural inefficiencies. This includes a large self-weight due to the volume of concrete filling the space between beams and a lack of composite action between the steel girders and the surrounding concrete.

OBJECTIVE

Two major modifications were proposed for the original BISB design in an effort to improve the overall structural efficiency while at the same time maintain its simplicity of construction. The first proposed improvement was to gain composite action between the steel girders and the surrounding concrete increasing the flexural rigidity of the section. The second proposed improvement was to remove the ineffective concrete from the tension side of the flexural section. The combination of integral action and reduced self-weight will allow for deeper girder sections, wider girder spacing and longer overall spans.

Developing Composite Action/ASC

Developing composite action was the first proposed improvement; the standard method of obtaining composite action in steel girder/concrete deck bridges is through the use of shear studs. Applying shear studs requires the use of a specialized welder, a piece of equipment that most counties lack. Thus, the development of an alternative shear connector that can be readily implemented by county forces was undertaken at Iowa State University

After extensive testing, a satisfactory Alternative Shear Connector (ASC) was developed (3), (5). The final design for the ASC consists of 1 1/4 in. diameter holes that are either torched or drilled through the web of a girder (see Figure 2). The holes are centered one diameter below the bottom of the top flange and set on a 3 in. longitudinal spacing for the length of the girder. The method of constructing the holes for the ASC was part of the original investigation. Both torched and drilled holes were investigated, and minimal difference was found between the methods of construction.

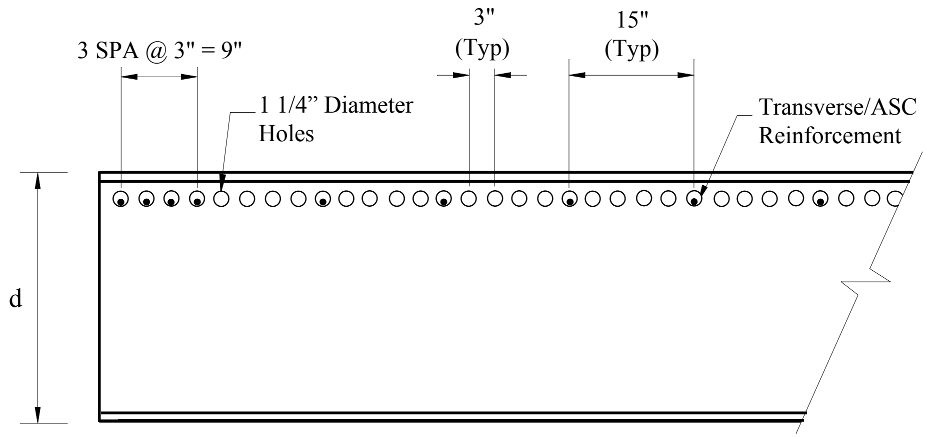


FIGURE 2. Typical Layout of the Alternative Shear Connector

Concrete flows through the holes in the web forming dowels to provide a mechanical connection between the slab and the girders. Transverse reinforcement is required through every 5th hole to provide lateral confinement of the shear dowels (5).

The ASC thus requires the concrete to be below the top flange and in contact with the web, ruling out the use of traditional steel girder/concrete deck forming systems. However, this method of obtaining composite action is readily applicable to the BISB system where the concrete is placed between the girders and is in direct contact with both flanges and the web.

Reducing Self-weight – Transverse Arch.

The span length of the original BISB system is limited by the excessive self-weight of the concrete that fills the void space between the girders. In addition, all the concrete on the tension side of the neutral axis does not contribute to the flexural strength. Thus, a design feature to reduce the self-weight while allowing for an increased girder depth, spacing and span length was desired. A transverse arch, resting on the bottom flange removes a large portion of the ineffective concrete. In addition to reducing the self-weight of the section, the transverse arch allows for a reduction in the deck reinforcement because of the resulting arching action.
Testing the Modifications

Four laboratory specimens were designed and constructed to test the proposed modifications and develop construction guidelines for future bridges. The objective of constructing and testing the four specimens was the quantification of the following parameters:

- Ultimate strength of the arched section
- Failure mode of the section
- Load distribution

- Ease of construction

All the laboratory specimens were constructed from recycled W21 x 62 girders and were outfitted with the ASC to gain composite action. An oxy-acetylene torch was used to construct the ASC holes. In the first three specimens, the only reinforcement was the transverse reinforcing steel needed for the ASC; in the 4th specimen, a layer of #3 reinforcing steel was placed transversely across the girders to prevent possible spalling over the embedded girders. Transverse steel straps were welded to the bottom flanges of the girders in all the specimens to restrain the girders during concrete placement and loading.

Forming the transverse arch posed a challenge because the formwork system needed to be structurally sufficient to support the plastic concrete, remain cost effective and preferably be both removable and re-useable. Several materials and geometric configurations were investigated as possible formwork systems.

POLYETHYLENE PIPE

Polyethylene drainage pipe was used as the formwork for the first arched laboratory test specimen (Figure 3a) *(5)*. Since the girder spacing was 42 in. (1,067 mm), the formwork was made from a section of 42 in. (1,067 mm) diameter pipe. The circular section reduced the amount of concrete needed by 36% and was freestanding. Similar to the plywood in the original BISB, the polyethylene pipe was a single use, stay in place formwork system. For larger girder spacings, polyethylene pipe is not an effective option due to increased material costs and limited geometries.

Arched Plywood

Arching plywood between the girders was investigated as a possible formwork system. This was attempted with W21 x 62 girders spaced 72 in. (1,830 mm) apart with minimal success (Figure 3b). Two layers of plywood with a thickness of 1/4 in. (6.4 mm) each were tested to see if a structurally adequate arch shape could be obtained. In spite of previous testing and analytical modeling, when filled with concrete, excessive deformations required the formwork to be shored, ruling out arched plywood as a possible formwork solution.

Culverts

Corrugated Metal Pipe (CMP) was used successfully as the arched formwork for a demonstration bridge not addressed in this paper. The girders were spaced 24 in. (610 mm) on center requiring a minimal span of 18 in. (457 mm). A 24 in. (610 mm) diameter, 16 gauge CMP was cut into thirds and placed between the girders, removing 17% of the concrete needed between the girders. The CMP was a single use stay in place corrosive formwork system; such systems do not follow standard Iowa Department of Transportation practice.

Custom Rolled Steel Sections

Circular sections, while readily available, are not the most efficient shape especially at larger girder spacings and are limited to standard sizes; thus, an alternative to the circular section was sought.

CMP is rolled from 25 1/2 in. (648 mm) wide steel sections and then riveted together to form the pipe. Based on this procedure, the concept of custom rolled arched steel formwork sections was developed. Two designs were chosen (small radius specimen and large radius specimen) and test specimens of each configuration were constructed to confirm the analytical analysis of the sections.

The small radius (15 in. (380 mm)) formwork for W21 x 62 girders spaced at 72 in. (1,830 mm) was constructed from 14 gauge galvanized steel with a 2 2/3 in. (67.5 mm) x 1/2 in. (12.7 mm) corrugation pattern. The large radius (27 in. (661 mm)) formwork specimen for W21 x 62 girders spaced at 72 in. (1,830 mm) was also made from 14 gauge galvanized steel with the same corrugation pattern.

The large radius formwork removed 45% of the concrete needed to fill the section while the small radius formwork removed over 52% of the concrete volume resulting in a significant reduction in self-weight and material. The sections were made to be recoverable and thus could be used in other bridges. Upon the selection of the formwork system, the specimens were designed, constructed and tested.

First Arch Specimen

A cross section of the first specimen is presented in Figure 3a. Following the original BISB design, the top flanges of the girder were exposed and used to strike off the concrete. The first specimen had the following properties: two girders on 42 in. (1,067 mm) centers, a length of 33.5 ft (10.21 m), and polyethylene pipe formwork. A single point load was applied at the center of the simply supported specimen to model a worst-case wheel loading.

The steel girders began to yield at a load of 126 kips (560 kN), indicating that the arched deck had a higher capacity than the girders in flexure. To investigate the punching shear capacity of the arched deck, the girders were blocked up and the specimen was reloaded. After blocking, the specimen failed in a splitting/punching shear mode when the bottom flange straps failed at a load of 177 kips (787 kN) *(5)*.

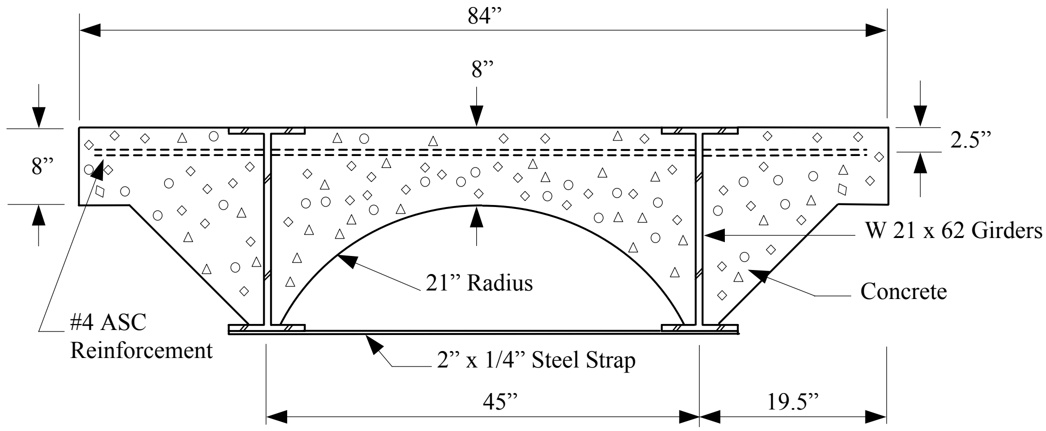
Second Arch Specimen

A cross section of the second specimen is presented in Figure 3b and has the following properties, two girders on 72 in. (1,830 mm) centers, a length of 14.5 ft (4.42 m), and arched plywood formwork. The girders were fully embedded with 3 in. (76 mm) of cover over the top flanges. This modification was made to lower the transverse steel, increase the moment of inertia of the section and provide for a more skid resistant deck. The simply supported specimen was subjected to a single point load at the center of the specimen simulating a worst-case wheel loading.

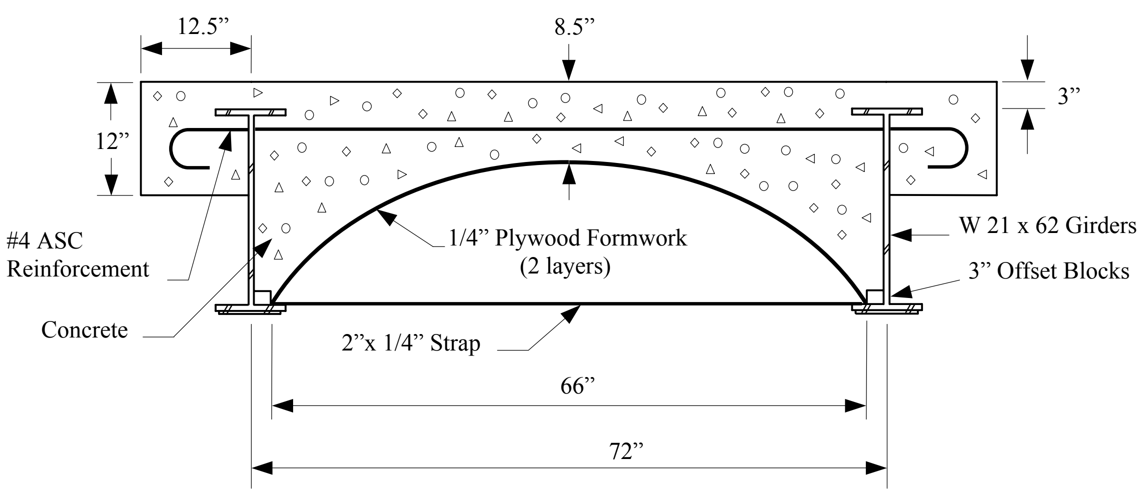
The short clear span was selected to force a punching failure mode prior to the girders yielding in flexure. The second specimen was tested similarly to the first and at an ultimate load of 155 kips (758 kN) a splitting/punching shear failure occurred in the deck when a bottom flange strap failed.

Third Arch Specimen

The third specimen was constructed to investigate improvements based on the results of the first two specimens. Large radius custom rolled corrugated steel sections were used as the formwork since the arched plywood did not perform adequately. A photograph of the third specimen being tested is presented in Figure 4. The third specimen had the following properties: two simply supported girders set on 72 in. (1,830 mm) centers and a length of 14.5 ft (4.42 m). A single point load was applied at the center, similar to the first two specimens.



(a) Cross section of the first arched specimen, polyethylene pipe formwork



(b) Cross section of the second arched specimen, arched plywood formwork

FIGURE 3. Arched Deck Specimens 1 and 2

The corrugated steel formwork, which was removed prior to testing, performed flawlessly and became the preferred formwork system. Larger confining straps were used on the third specimen and a punching failure occurred at 260 kips (1,157 kN). This test provided assurance that the transverse arch section in combination with the ASC provided for a sufficiently strong system with the capacity to resist legal Iowa loads.

To this point, only single bay specimens had been constructed and tested in an effort to quantify the capacity and mode of resistance of the composite transverse arched section. The previous specimens provided minimal information on the behavior of the section in an actual bridge application where load would be distributed to adjacent bays. A fourth specimen was designed and constructed to obtain distribution data as well as ultimate strength data on the composite arch system when it was used in a bridge.

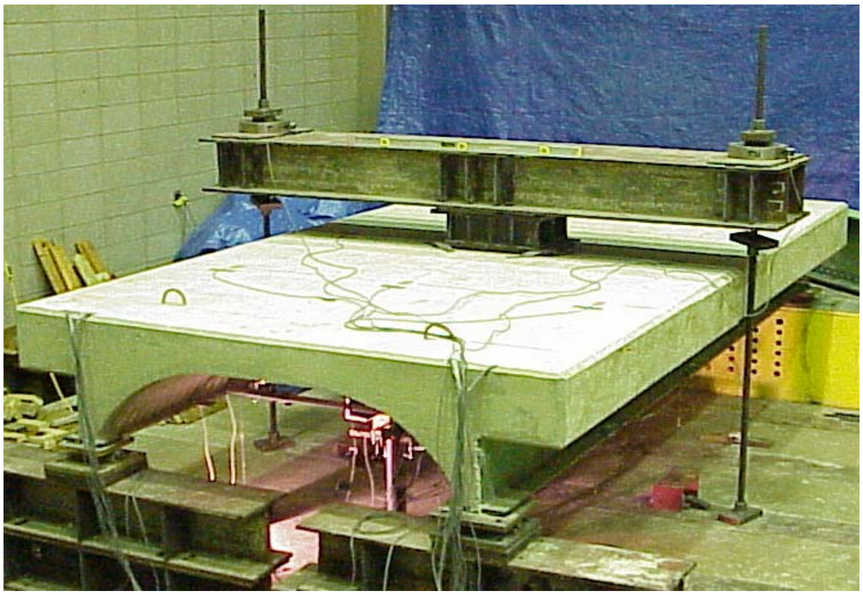


FIGURE 4. Test Setup for the Third Arched Specimen

Fourth Arched Specimen

The fourth specimen consisted of four simply supported girders on 72 in. (1,830 mm) spacing, forming three monolithic bays. The overall dimensions of the specimen were L = 31 ft (9.45 m) W = 20 ft (6.1 m). Small radius custom rolled steel sections were used for the formwork. An overall view of the specimen during construction with the arched formwork in place is presented in Figure 5a. The finished structure with the service level loading system in place is presented in Figure 5b.

(a) Formwork and reinforcing before concrete placement

(b) Typical service level loading setup

FIGURE 5. Construction and Service Level Koading of Specimen 4

The specimen was subjected to service level loads representing a single wheel load at several locations. The load was applied at six loading points transversely across the structure at the 1/4 span and the midspan plus three load points at the 3/4 span to quantify the lateral load distribution. Two series of service level tests were performed, the first with the confining straps in place and the second with the confining straps released. It was found that at service level loads, releasing the transverse straps had little effect on the behavior of the structure.

Midspan deflections due to a concentrated load placed at the mid point of the specimen are presented in Figure 6. For this load configuration it can be observed that the difference between the straps being in place or removed has a minimal effect; the largest change in deformation for this load case was 0.009 in. (0.23 mm). The displaced shape also indicates a reasonably symmetric transverse load distribution. The largest difference in deflection for any of the service load cases was approximately 0.018 in. (0.46 mm).

The arched specimen also exhibited excellent lateral load distribution under the service level load. For the load case represented in Figure 6, the maximum percentage of the total load carried by a single interior girder is 32% based on the moment fraction calculated from strain data due to flexure. This result is significantly less that the 40% calculated for an interior girder subjected to a single lane loading based on the 1994 AASHTO LRFD Bridge Specification for beam and slab decked bridges *(6)*.

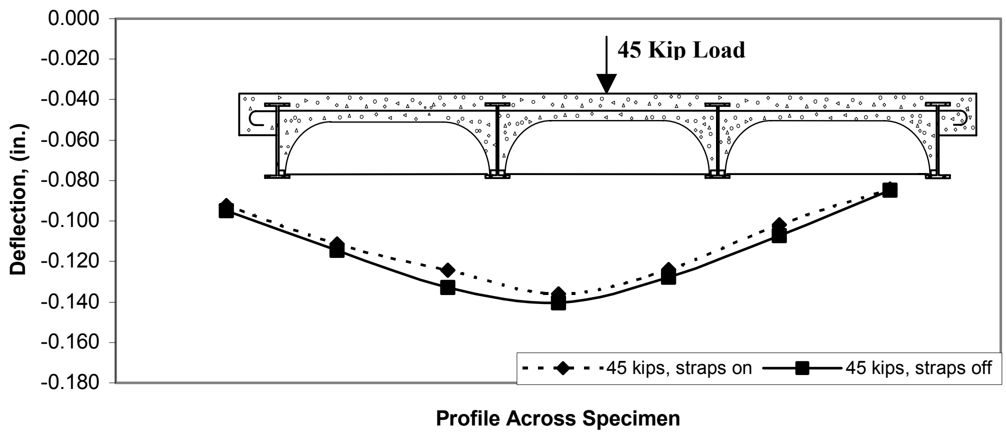


FIGURE 6. Plot of Midspan Deflections with and without Confining Straps

Upon the completion of the service level tests, the specimen was subjected to ultimate loading. The specimen failed in flexure with the longitudinal girders yielding and deflecting to a maximum of 7.1 in. (180 mm) at a load of 302 kips (1,343 kN) prior to the termination of the test. The data gathered and observed behavior of Specimen 4 under service and ultimate loading provided verification that the modifications to the original BISB had more than sufficient strength to be used in a field application. A demonstration bridge utilizing both modifications was first designed and then constructed.

MODIFIED BEAM-IN-SLAB DEMONSTRATION BRIDGE

Design

The design of the demonstration bridge followed the 1994 AASHTO LRFD Bridge Specification (6). The structure was designed to meet applicable strength and serviceability requirements for an HS-20-44 design vehicle. To ensure a conservative design for the first bridge of this type, the steel girder/concrete deck slab load distribution factors were used to calculate lateral load distribution to the girders.

Six W27 x 129 Grade 50 steel girders spaced 72 in. (1,830 mm) on center with the ASC were required for the 70 ft. (21.34 m) span. Custom rolled corrugated formwork with a radius of 20 1/2 in. (521 mm) were constructed to form four of the five bays. The remaining bay was formed by modifying and reusing small radius formwork sections used in Specimen 4 (laboratory bridge).

Two lines of diaphragms of recycled S18 x 55 sections were placed at 24 ft (7.31 m) from either end of the structure. The diaphragm spacing was necessary to prevent instability during the placement of the concrete deck. In order to obtain the needed development length in the transverse ASC reinforcement, a custom exterior formwork system was also designed.

Construction

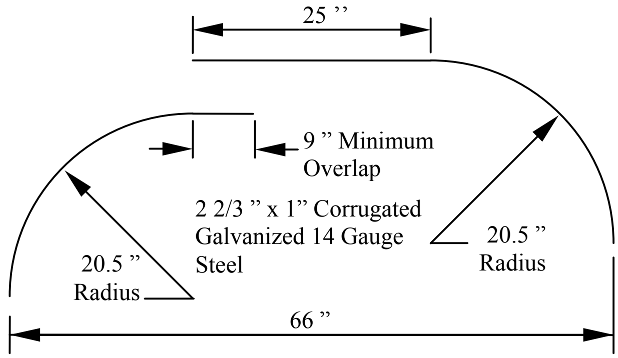
Construction of the demonstration bridge followed the traditional steel girder/concrete slab bridge construction format. Upon the completion of the abutment system, the cambered girders, complete with a drilled ASC, were fitted with diaphragm and exterior formwork brackets and 'swung' into place (Figure 7a). Transverse crown was introduced to the structure through the use of steel bearing pad of staggered thickness. The diaphragms were installed and a concrete backwall system placed, leaving placement of the arched formwork as the next step.

The custom rolled corrugated sections were assembled offsite prior to the arrival of the girders. A local culvert manufacturer rolled the sections to the specifications indicated in Figure 7b. The assembly process took the following sequence. The two partial sections were assembled into one piece by bolting the individuals together with 5 – 1/4 in. x 3/4 in. (6.25 mm x 19 mm) Grade 5 cap screws. The individual arched sections were then fitted together and bolted into 10 ft. (3.05 m) long segments and stored at the ISU Structures Laboratory. The completed segments were transported to the job site and placed directly into the bridge. Figure 7c documents the assembly of typical segments and Figure 7d gives a clear view of installing the segments. The segments of formwork were blocked into place using wooden spacer blocks and shimmed tight to the diaphragms and backwalls completing the interior formwork.

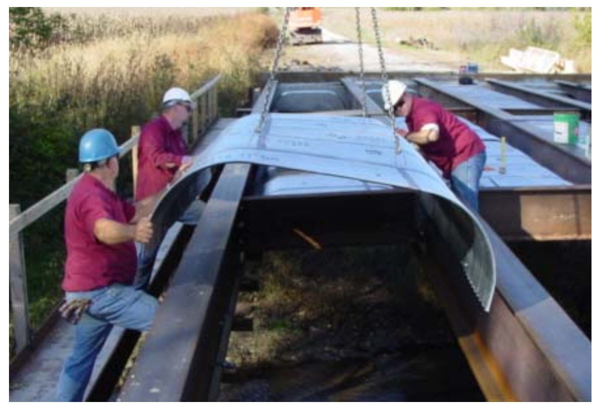
The structural reinforcement required in the transverse arched section, and additional temperature and shrinkage reinforcement was then set in place. The exterior formwork was set into place and a leveling rail installed to set the grade for the power screed to complete the bridge formwork.

After the concrete was placed and cured and a thrie beam guardrail system was installed, the bridge was opened to traffic. The completed structure can be viewed in Figure 8a. Four months after the concrete placement, county crews removed the custom rolled formwork leaving the underside of the Modified Beam in Slab Bridge readily viewable (see Figure 8b).

The structure was constructed completely by in house forces with a cost savings of approximately 10% in comparison to completing traditional structures. Field testing of the demonstration bridge is scheduled for mid-July 2003.



(a) Setting the girders into place **(b) Custom rolled corrugated arches**



(c) Assembling the segments **(d) Setting the arches into place**

FIGURE 7. Setting the Girders and Constructing the Arched Formwork

(a) Finished MBISB Demonstration Bridge **(b) Removing the arched formwork**

FIGURE 8. Completing the Construction of the Demonstration Bridge

SUMMARY/CONCLUSIONS

The original BISB design has served as an alternative replacement design for LVR in Iowa for over 25 years. The system can span openings of up to 50 ft (15.24 m) and both field and laboratory testing have verified the design as structurally adequate for legal loads. The system is simple to construct and requires minimal equipment. However, due to excessive self-weight, the system is limited in span, girder size and spacing.

Two modifications, the ASC and the transverse arched section, were proposed to improve the overall efficiency of the structure. It was hypothesized that the modifications would allow for a deeper girder sections, wider girder spacing and longer spans while making better use of materials. Research was undertaken to investigate the applicability of the modifications.

Four laboratory test specimens were constructed and tested to evaluate the performance of the modifications. Three single bay specimens were constructed to study the structural behavior of the transverse arch. Results from the three specimens suggested the system was applicable to a bridge application and implicated the custom rolled sections as the formwork system of choice. A 3-bay specimen was constructed to investigate the lateral load distribution and flexural behavior of the combined modifications in a bridge system. Results from service level loadings indicated superior load distribution and ultimate testing results provided evidence that the MBISB system could easily support Iowa legal loads.

With the success of the laboratory testing program, a demonstration bridge was designed and constructed utilizing both of the modifications. The resulting structure was constructed solely by county forces and resulted in a cost savings for the county government. The structure has been in service for seven months and will be load tested this summer.

ACKNOWLEDGEMENTS

The research presented in this paper was funded by the Highway Research Board and the Highway Division, Iowa DOT, Ames, Iowa. The authors wish to thank numerous Iowa county engineers and undergraduate students who worked on this investigation.

DISCLAIMER

The opinions, findings, and conclusions expressed herein are those of the authors and not necessarily those of the Iowa DOT or the Iowa Highway Research Board.

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A Business Case for Winter Maintenance Technology Applications: Highway Maintenance Concept Vehicle

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ABSTRACT

The purpose of this paper is to demonstrate, from a business perspective, the benefits of using technology applications in winter maintenance operations. This paper documents the business case to be made for the technology applications on the Highway Maintenance Concept Vehicle (HMCV) project by examining the business implications of many benefits such as increased safety, reduced environmental impacts, and increased efficiency.

The use of commercial-off-the-shelf (COTS) and prototype technologies to improve winter maintenance operations has been in practice for several years; however, it has been very difficult to quantify the benefits achieved by adopting these technologies. A benefit-cost framework is established whereby the current methods of performing the analysis can be compared to other proposed winter maintenance technology improvements.

Applying new technology to winter maintenance operations can

- Reduce accidents
- Reduce chemical use
- Provide return on investment

The benefit-cost analysis demonstrated that the integration of the newer emerging technologies does indeed play a beneficial role in reducing accidents, increasing mobility, reducing adverse environmental impacts and having a direct bearing on the economic impacts in the area.

Key words: benefit-cost analysis—business case—technology—winter maintenance

Proceedings of the 2003 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2003.

INTRODUCTION

This report updates the Highway Maintenance Concept Vehicle, Phase IV report by providing a business case for technology applications on the Highway Maintenance Concept Vehicle. The case to be made examines many benefits such as increased safety, reduced environmental impacts, and increased efficiency. A benefit-cost framework is established whereby the current methods of performing the analysis can be compared to other proposed winter maintenance technology improvements.

The objectives of applying new technology to winter maintenance operations are:

- Reduction in accidents:
- Reduced chemical usage
- Return on investment

A literature review available on winter anti-icing operations using advanced technology by different agencies indicates reduction or eliminating of accident rates by 73-80 per cent. The Pennsylvania DOT reported an accident reduction of close to 100 percent using anti-icing techniques (*1*) but given certain allowances a presumed 80 percent or a .2 resultant factor rate was used (*1*).

Reduced chemical usage: The Benefit Cost Ratio also suggests that although costs savings are definitely possible with the usage of anti-icing technologies, the level of service to the travelers are increased with the same or less usage of materials. Some of the other benefits are in less chemical usage, less time on equipment and increase in the efficiency of the system.

A similar study was done at the City of Kamloops, British Columbia. Reduced chemical usage: The Benefit Cost Ratio also suggests that although costs savings are definitely possible with the usage of anti-icing technologies, the level of service to the travelers are increased with the same or less usage of materials. Return on investment: The sensitivity analysis parameters for accident reduction/elimination included a wide range of numbers – from the reported close to an 80% accident eliminated (Pennsylvania DOT and Kamloops, British Columbia study) to a range of 50% reduction. Even at the low 50% range, the Benefit Cost Ratio was still favorable at 2.31 and 2.37 indicating a 131% and 137% (depending upon the discount rate) rate of investment (ROI) return for the project.

Return on investment: The sensitivity analysis parameters for accident reduction/elimination included a wide range of numbers – from the reported close to an 80% accident eliminated (Pennsylvania DOT and Kamloops, British Columbia study) to a range of 50% reduction. Even at the low 50% range, the Benefit Cost Ratio was still favorable at 2.31 and 2.37 indicating a 131% and 137% (depending upon the discount rate) rate of investment (ROI) return for the project.

The investigation also examined some of the secondary benefits and risks associated with these technology applications to winter maintenance operations.

BACKGROUND

The entire Highway Maintenance Concept Vehicle project encompassed four phases over a six–year period. Phase I of this project focused on describing the desirable functions of a maintenance concept vehicle and evaluating its feasibility. Phase II was the proof of concept phase that included development, operation, and observation of three prototype vehicles. Phase III included, conducting the field evaluation of three prototype vehicles, and identifying method(s) and cost to integrate the data and information generated by the vehicle into the state's snow and ice control management process. The research quickly pointed out that investments in winter storm maintenance assets must be based on benefit/cost analysis and related to improving level of service. If the concept vehicle and data produced by the vehicle are used to support decision-making leading to reducing material usage and the average time by one hour, a reasonable benefit/cost will result. In Phase IV we, at CTRE, as well as our partners at Iowa DOT, Wisconsin DOT, and Pennsylvania DOT, long considered the benefits of advanced technology to enhance winter maintenance activities. The HMCV demonstrated near term benefits of the technologies for winter maintenance operations.

WHY BUSINESS CASE?

A business case is one way to organize, evaluate, and present information about the actions that governments take to improve public safety. In this report, all benefits and costs each year between 2000 and 2001 are included in the benefit–cost analysis (BCA) and all values are discounted back to 2000 using both a 4 percent and a 7 percent real discount rate to calculate the present values of the benefits and costs in 2002 dollars.

The Benefit–Cost Ratio suggests that although costs savings are definitely possible with the usage of anti-icing technologies, the level of service to the travelers is increased with the same or less usage of materials.

DETAILED RESEARCH

The first step in developing the business case was to analyze the data and information technology in the Iowa DOT maintenance operations and determine their impact on the cost of conducting those activities. Reductions in resource costs, labor, trucks, and materials, are achieved by identifying cost factors and by taking actions to influence those factors. Certainly the severity of the winter affects winter maintenance costs. A "bad" winter is very expensive and requires using a large amount of labor, trucks, and materials to achieve an acceptable level of service.

Every year, state agencies spend an estimated two billion dollars plowing snow, sanding and spreading chemicals on icy roadways. Approximately 20,000,000 metric tons of sodium chloride (road salt) is used annually for deicing. The demand for salt has doubled in the last ten years. To keep roads clear and safe for travel, the Iowa DOT, for example, spends approximately $35 million every year on winter maintenance spending an average of $65,000 to $70,000 per hour to fight the winter storms, (*2*).

This Benefit Cost Analysis (BCA), conducted as part of the overall goal of the pooled fund study's Highway Maintenance Concept Vehicle project to "examine and test newly emerging technologies that have the potential for improving the level of service defined by policy during the winter season at the least cost to taxpayers."

Benefits of anti-icing techniques have been summarized in the table below. Based upon the perception of maintenance personnel, most benefits were related to improved safety and improved productivity (i.e., reduced maintenance costs), (*3*).

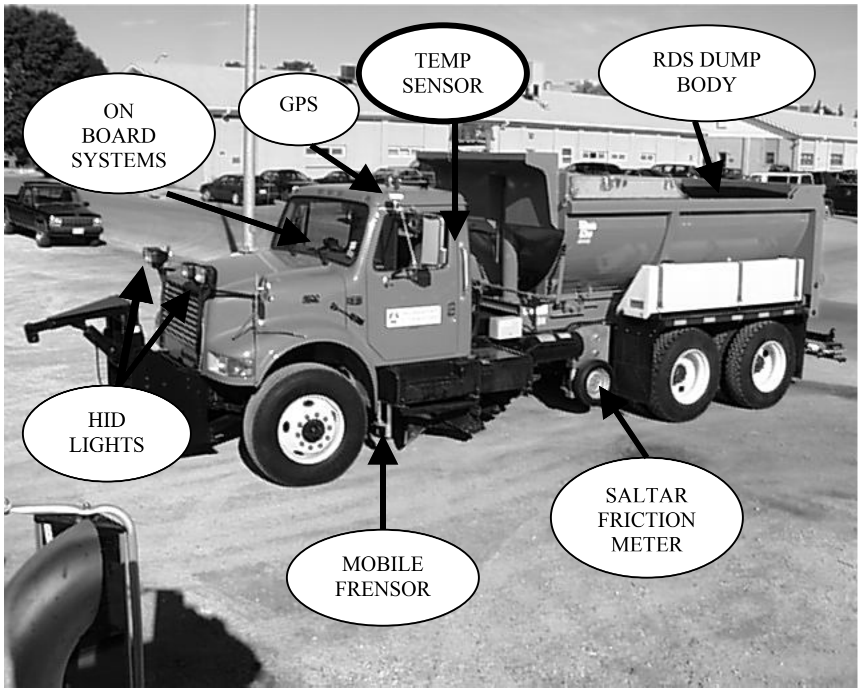

FIGURE 1: Phase IV Highway Maintenance Concept Vehicle (2001-2002)

METHODOLOGY

The methodology for conducting the BCA is based on the guidelines outlined for generally accepted practices for federal projects and on the Benefit/Cost Analysis of ITS Applications for Winter Maintenance conducted by Robert Stowe, P.E., previously with the Washington DOT. Using information obtained from the DOT officials from the Washington and Arizona DOT, it was established that the BC Worksheet for Collision Reduction was an appropriate and a valid tool in computing the benefits and costs for collision reduction.

The BCA considered two different alternatives for this study.

Do nothing Alternative. The first alternative is the 'do-nothing' alternative where the status quo approach of using traditional method of winter application was considered. Although this practice has worked successfully over the past years, this system is not very efficient in terms of safety, consumption of materials and labor. This inefficient system uses the outdated reactive deicing approach in solving winter road surface conditions. A study by the Iowa DOT suggests that 30 percent of the salt is wasted using existing techniques. Also, excessive chemical run-offs contaminate soil and ground water contributing to environmental degradation. Although the 'do-nothing' alternative has the advantage of low implementation cost, it carries high operational efficiencies and perhaps much higher costs in terms of societal benefits.

The second alternative considered was to study the benefit cost analysis of deploying and integrating anti-icing strategies on HMCVs during winter applications. A comprehensive BCA would include quantification and monetization of various elements of the analysis; safety in terms of accidents reduced or eliminated, reduction in salt, chemical and labor usage, environmental equity, savings in time due to increased mobility, etc. Since no historical or relevant data on environmental, mobility and labor usage was available, this BCA was conducted on two counts viz. Safety and Material usage (salt). The crash report was based on information available at the Iowa Traffic Safety Data Service and CTRE for crash severity data for years 1998-2000. The Iowa DOT provided the salt usage data at the Des Moines North Garage.

One of the basic concepts of Benefit–Cost analysis is not to consider sunk costs (money already spent). This appears to be consistent with one of the purposes of the HMCV, which is to determine whether or not to proceed with the project according to the plan outlay. Because this analysis is being done after the development costs have been incurred, the purpose of this BCA is not to determine development and operational costs of the system will be justified by the projected benefits, but rather to evaluate whether the projected costs and benefits (starting with fiscal year 2000) justify continuation of the project, (NIH. 1998).

The BCA analysis undertaken in this study differs marginally from the original model envisaged in Task 3 of the Phase IV study Work Plan dated February 28, 2002 for the following reasons.

1. Discounting: In the original model, discounting, where future cash flows are reduced to equivalent present-day values, is not addresses. Discounting is important element of a BCA whereby the costs and benefits of each year of the system cycle is estimated and is then converted to a common unit of measurement to properly compare competing alternatives.

2. Start –up costs or sunk costs are usually not part of the BCA analysis for an ongoing project.

Data were available on safety and materials, specifically, usage of salt. This provided the basis to conduct the BCA based on these data.

ASSUMPTIONS

BCAs should be explicit about the underlying assumptions used to arrive at estimates of future benefits and costs. For our analysis, these are some of the assumptions:

1. Wet conditions have been included in the element of surface conditions for winter driving conditions. Since anti-icing materials are applied to the roadway immediately before or at the beginning of a storm, the road becomes wet or slushy rather than icy. This wet road conditions are considered part of the surface condition under analysis.

2. The BCA time period should match the system life cycle. The system life cycle includes the following stages/phases: a) Feasibility study b) Design c) Development d) Implementation e) Operation. The system life cycle or technological obsolescence is considered five years.

3. The BCA computation has been done on two factors, that of Safety and salt usage. The Iowa DOT has specific information on crash severities. Literature reviews indicated that

specific data was available for crash severities and value of life matrix. Not much data was available for the mobility, efficiency, productivity and environmental quality of the areas under analysis.

4. Based on the data available, it was determined that accidents were eliminated by almost 80% when anti-icing techniques were used. This was the case in Pennsylvania and British Columbia, Canada. Resultant factor has been computed on different range of values. The first at 0.2 (80% accidents were eliminated) and the second at 0.5 (50% of the accidents were eliminated).

HMCV

A portion of the equipment used in this research was provided to the study, some had to be purchased and some was included in the cost of upgrading the snowplow that was done during the normal course of normal vehicle replacement schedule at Iowa DOT.

The chart below lists the costs of the equipment, along with the vendors that supplied those equipment items.

TABLE 1: Costs of HMCV

Equipment Item	Cost	Vendor
Chassis – International	$65,500	Monroe Snow & Ice Control
RDS Dump Box	5,500	Same
Front Plow	4,000	Same
Sander/Salter	2,600	Same
Underbody Blade	6,600	Same
Added Features for HMCV		
On board pre-wetting	2,500	Monroe
Anti Icing Spray bar	14,000	Monroe
Surface Temp. Sensor	800	Sprague
AMS 200 Data Management	2,500	Raven Industries
DCS 710 Ground Speed Controller	8,000	Raven Industries
Trakit AVL	13,000	IDA Corp
DGPS Antennae	1,400	Communications Systems International
HID Plow Lights	1,100	Speaker
Saltar Friction Meter	15,000	Norsemeter
Frensor Mobile Freeze Point Detection	10,500*	AeroTech-Telub
Total	**$153,000**	

BENEFIT COST CALCULATIONS

All benefits and costs each year between 1998 - 2000 are included in the BCA and all values are discounted back to 2000 using both a 4 percent and a 7 percent real discount rate to calculate the present values of the benefits and costs in 2000 dollars. Planners and economists have traditionally followed the use of 4 percent real discount rate in these benefit/cost calculations using the guidelines published in the 'red book,' by AASHTO. (A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements, 1977) published by the American Association of State Highway and Transportation Officials. The 7 percent real discount rate is a federal Office of Management and Budget guideline for calculating a BCA for federal projects.

DISCOUNTING

Future cash flows need to be reduced to equivalent present-day values, because the value of a dollar in the future is less than the value of a dollar today. This is referred to as discounting. After the costs and benefits for each year of the system cycle have been estimated, it is converted to a common unit of measurement to properly compare competing alternatives. That is accomplished by discounting future dollar values, which transforms future benefits and costs to their "present value." The PV (also referred to as the discounted value) of a future amount is calculated with the following formula:

$P = F\ (1 / (1 + I)^n$ where,

P= Present Value
F=Future Value
I=interest rate &
n=number of years

Government policies or projects typically produce streams of benefits and costs over time rather than in one-shot increments. Commonly, in fact, substantial portions of costs are incurred early in the life of the project, while benefits may extend for many years. Yet, because people prefer a dollar today than ten years from now, BCA typically discounts future benefits and costs back to present values. The system life cycle for this study is considered 5 years.

SENSITIVITY ANALYSIS

At its most rudimentary, a Sensitivity Analysis demonstrates the impact of variations on the discount rate on the final analysis. But to put in a broader perspective, a sensitivity analysis tests the impact of changes in input parameters on the results obtained from the benefit-cost analysis. For example, how much change in the value of the benefits is required before the costs of the proposed system exceed the benefit.

An appraisal should always be subject to a sensitivity test to assess how robust the result is to changes in the assumptions used in calculating it. In particular, a range of expected accident reductions should be assessed, since one can never be certain as to what the actual outcome will be; using a low and a high estimate of possible and realistic outcomes is always good practice. In our case study, although literature review suggests that the resultant factor of 0.2 is safe to use (80% of accidents were eliminated), the range was expanded to include a conservative value of 0.5 resultant factor (50% of the accidents were eliminated).

If the outcome is favorable even if a pessimistic forecast is used, we can be confident that the project is worthwhile. Conversely, if the outcome is unfavorable even with optimistic assumptions, we can be confident that the project is unlikely to be worthwhile. The middle ground―favorable under optimistic assumptions and unfavorable under pessimistic assumptions—requires us to do more work to try and get a better forecast.

CONCLUSIONS

The BCA demonstrates that the integration of the newer emerging technologies in the concept vehicle does indeed play a beneficial role in reducing accidents, increasing mobility, reducing adverse environmental impacts and having a direct bearing on the economic impact in the area. The sensitivity analysis parameters for accident reduction/elimination included a wide range of numbers – from the reported close to an 80% accident eliminated (Pennsylvania DOT and Kamloops, British Columbia study) to a range of 50% reduction. Even at the low 50% range, the Benefit Cost Ratio was still favorable at 2.31 and 2.37 indicating a 131% and 137% (depending upon the discount rate) rate of investment (ROI) return for the project.

The Benefit Cost Ratio also suggests that although costs savings are definitely possible with the usage of anti-icing technologies, the level of service to the travelers are increased with the same or less usage of materials. Some of the other benefits are in less sand and chemical usage, less time on equipment and increase in the efficiency of the system.

ACKNOWLEDGEMENTS

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APPENDIX A. BENEFIT COST WORKSHEET

For collision Reduction

Safety Improvement Location: Select Corridor I –35 Polk County, Iowa (appendix C)
Safety Improvement Description: Technology add-ons on HMCV

1. Initial Project Costs, I: $50,691
2. Net Annual Operations and Maintenance Costs, K: $ 5,000
3. Annual Safety Benefits in Number of Collisions:

Before (historic) Collision type	Nos.	Yrs.		- Rate	After (estimated) Resultant Rate Factor			= Annual Benefits
a) Fatality	0	3	=	0.0	.2	0.00	=	0.00
b) Major Injury	4	3	=	1.33	.2	0.27	=	1.06
c) Minor Injury	17	3	=	5.67	.2	1.13	=	4.54
d) Possible Injury	18	3	=	6.0	.2	1.20	=	4.80
e) Property Damage Only	51	3	=	17.0	.2	3.40	=	13.60

1. Costs Per Collision: 5. Annual Safety Benefits by Costs of Collision

Collision Type	Cost				
a) Fatality	$ 1,000,000	a)	(3a)(4a)	=	$ 0
b) Major Injury	$ 150,000	b)	(3b)(4b)	=	$159,000
c) Minor Injury	$ 10,000	c)	(3c)(4c)	=	$ 45,400
d) Possible Injury	$ 2,500	d)	(3d)(4d)	=	$ 12,000
e) Property Damage Only	$ 2,500	e)	(3e)(4e)	=	$ 34,000
			TOTAL, B	**=**	**$250,400**

6. Service life, n = 5 years 7. Salvage Value, T = $1,000 8. Interest rate, I = 4%

9. Present Worth of Costs, PWOC:

Present Worth Factor of a uniform series, SPWin = 4.4518
PWOC = I + K(SPWin) – T(PWin) = 67,786

10. Present Worth of Benefits, PWOB =B(SPWin) = 1,114,731

11. Benefit Cost Ratio, B/C = PWOB/PWOC = **16.45**

12. Net Benefit = PWOB – PWOC = **1,046,945**

Sensitivity Analysis based on a) accident savings of 80% (.2 resultant factor) and b) interest rate at 4% (A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements 1977, page 14). SPWin rates from Appendix A, page 118).

Deploying the Winter Maintenance Support System (MDSS) in Iowa

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ABSTRACT

Adverse weather conditions dramatically affect the nation's surface transportation system. Each year, 6,600 people die, 470,000 people are injured and 544 million hours of time are lost on the nation's highways because of adverse weather conditions, according to the Federal Highway Administration. The development of a prototype winter Maintenance Decision Support System (MDSS) is part of the FHWA's effort to produce a prototype tool for decision support to winter road maintenance managers to help make the highways safer for the traveling public. The MDSS is based on leading diagnostic and prognostic weather research capabilities and road condition algorithms, which are being developed at national research centers.

It is anticipated that components of the prototype MDSS system developed by this project will ultimately be deployed by road operating agencies, including state departments of transportation (DOTs), and generally supplied by private vendors.

In 2003, The Iowa Department of Transportation was chosen a field test bed for the continuing development of this important research program. The FHWA also selected five national research centers to participate in the development of the prototype MDSS. They were selected because of the applicability of their expertise to the MDSS task. The participating national labs include the Cold Regions Research and Engineering Laboratory (CRREL), National Center for Atmospheric Research (NCAR), Massachusetts Institute of Technology - Lincoln Laboratory (MIT/LL), National Severe Storms Laboratory (NSSL), and the Forecast Systems Laboratory (FSL).

Key words: decision support system—technology—winter maintenance

INTRODUCTION

The development of a prototype winter Maintenance Decision Support System (MDSS) is part of the Federal Highway Administration (FHWA) Road Weather Management Program. The objective of the MDSS effort is to produce a prototype tool for decision support to winter road maintenance managers. The MDSS is based on leading diagnostic and prognostic weather research capabilities and road condition algorithms, which are being developed at national research centers. It is anticipated that components of the prototype MDSS system developed by this project will ultimately be deployed by state departments of transportation (DOTs), and generally supplied by private vendors.

There are five national research centers that are participating in the development of the MDSS Functional Prototype (FP). The participating national labs include:

- Army Cold Regions Research and Engineering Laboratory (CRREL)
- National Center for Atmospheric Research (NCAR)
- Massachusetts Institute of Technology - Lincoln Laboratory (MIT/LL)
- NOAA National Severe Storms Laboratory (NSSL)
- NOAA Forecast Systems Laboratory (FSL)

The MDSS field demonstration evaluated the MDSS by operating the systems in a real time winter environment. This also allowed the users to work the system and verify the data. The following evaluations were performed in FY2003:

1. Weather prediction component
2. Treatment recommendations
3. Impact of supplemental mesoscale models
4. Potential benefit of operational system
5. Identify and evaluate current system limitations

The Iowa DOT provided a “test bed” for the MDSS prototype in the winter of 2003.

Field Demonstration Period

The MDSS field demonstration began on February 3, 2003 and continued through April 7, 2003, to capture all major snow events. The system operated 24-hours per day, 7-days per week during this period. Three DOT maintenance garages participated in the demonstration. They were:

- Ames Garage
- Des Moines - North

- Des Moines - West

Selected Winter Maintenance Routes for Field Demonstration

Iowa DOT representatives selected several winter road maintenance routes that were used in the MDSS field demonstration. A total of 15 routes, covering 400 miles were configured in the MDSS. The selected routes are described in Table 1 and a corresponding map of the routes is provided in Figure 1. Separate treatment plans were generated by the MDSS prototype for each of the routes shown in Figure 1.

TABLE 1. Iowa Maintenance Routes for the MDSS Field Demonstration

Garage	**Segment Number**	**Route**	**Start Mile Post**	**End Mile Post**	**ADT Range**	**Service Level**
Ames	1A	US 65	98.38	112.09	1500-2000	C
Ames	1B	US 30	164.93	172.30	5000-6000	B
Ames	2	US 65	112.09	132.59	1000-2000	C
Ames	3	I-35	111.60	128.46	20000-23000	A
Ames	4	I-35	96.60	111.60	23000-26000	A
Ames	5	US 30	142.88	172.30	6000-27000	B
Ames	6	IA 210	13.79	34.43	1000-3000	D
Des Moines North	7	I-35	93.20	96.60	26000-53000	A
Des Moines North	8	I-35	86.94	93.20	53000-59000	A
Des Moines North	9	I-80	137.82	142.10	50000-61000	A
Des Moines North	10	I-35/I-80	131.50	137.82	59000-63000	A
Des Moines West	11	I-35/I-80	123.53	131.50	32000-72000	A
Des Moines West	12	I-35	67.89	72.70	22000-33000	A
Des Moines West	13	I-235	0.00	8.80	42000-125000	A
Des Moines North	14	I-235	8.80	14.26	46000-125000	A
Des Moines North	15	IA 415	0.00	21.93	1000-21000	B-D

Service Levels:

A	Interstates	C	2,500 – 5,000 vehicles per day
B	5,000+ vehicles per day	D	less than 2,500 vehicles per day

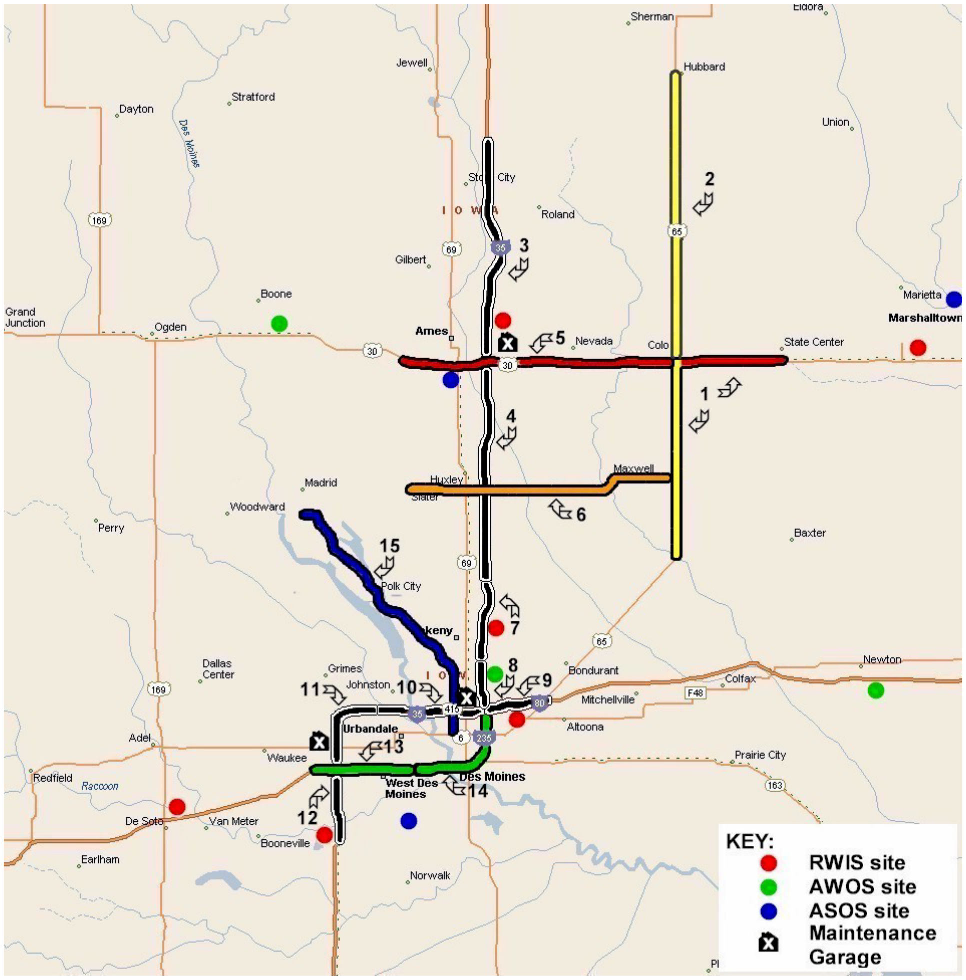


FIGURE 1. Map Of Routes to be Supported by the MDSS Prototype During the Winter of 2003 Iowa Field Demonstration

The MDSS Iowa Weather Display

The MDSS Iowa Weather Display page was configured to provide weather alerts when the weather conditions deteriorated according to the criteria in the MDSS Technical Description. The alerts graphically referenced based on the Iowa weather forecast zones illustrated in Figure 2. These forecast zones are consistent with the weather forecast zones used by Meridian Environmental Technology, which is the operational road weather forecast provider for Iowa during the winter of 2002-2003.

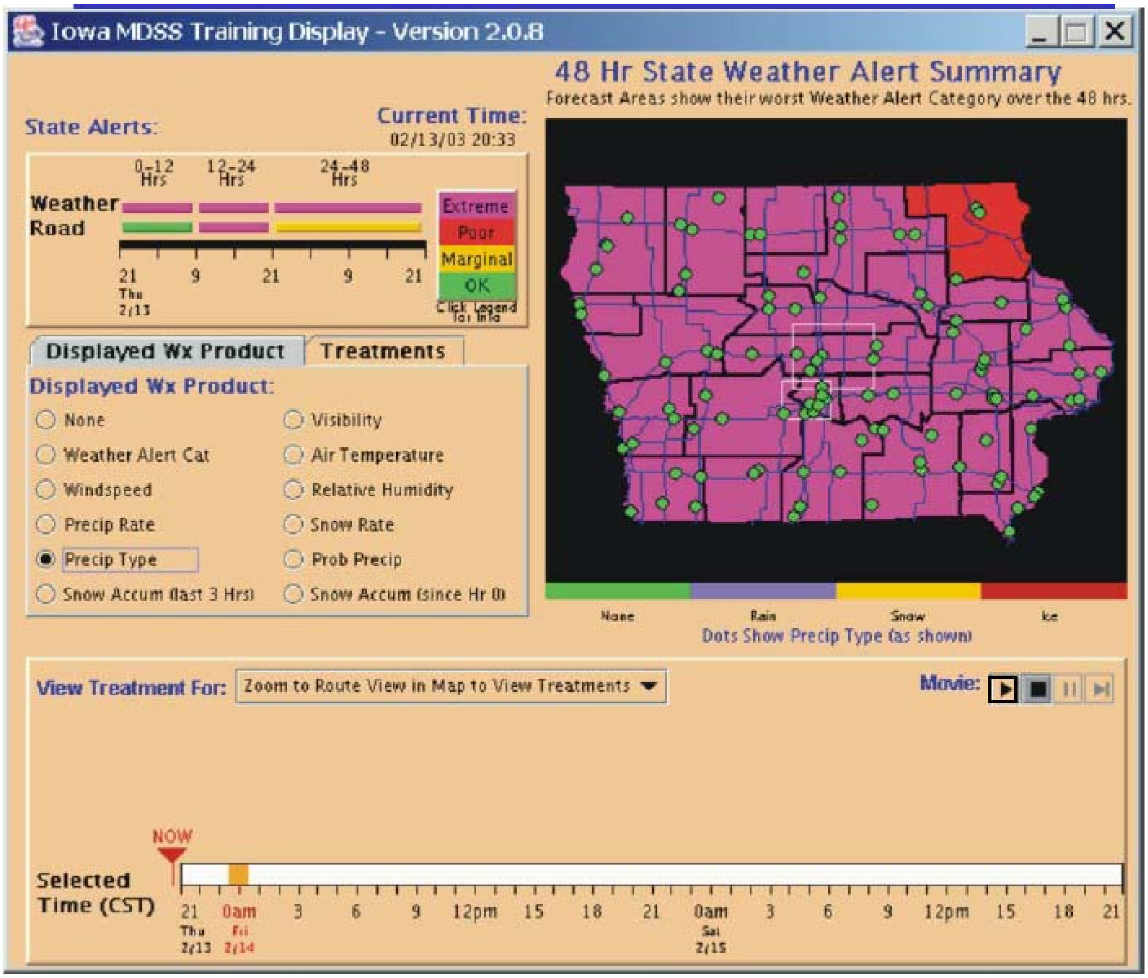


FIGURE 2. MDSS Iowa Weather Display Page

MDSS System Configuration for Iowa

The MDSS core components (e.g., Road Weather Forecast System, Road Condition and Treatment Module and data server) operated centrally at NCAR in Boulder, Colorado. A server at NCAR communicates (via the Internet) with local PCs running the display application at the Iowa DOT maintenance garages. Supplemental weather forecast models run at FSL in Boulder and the data are forwarded to NCAR for inclusion in the Road Weather Forecast System (RWFS). Iowa DOT RWIS data were also provided to NCAR via FSL as part of the MADIS project (*1*).

The MDSS displays are located in the three maintenance garages. Each garage had the MDSS display running at the supervisor's desk and an additional display application at the shift supervisor's desk. Data were obtained over the Internet (client-server approach). A simplified illustration of the system configuration is provided in Figure 3.

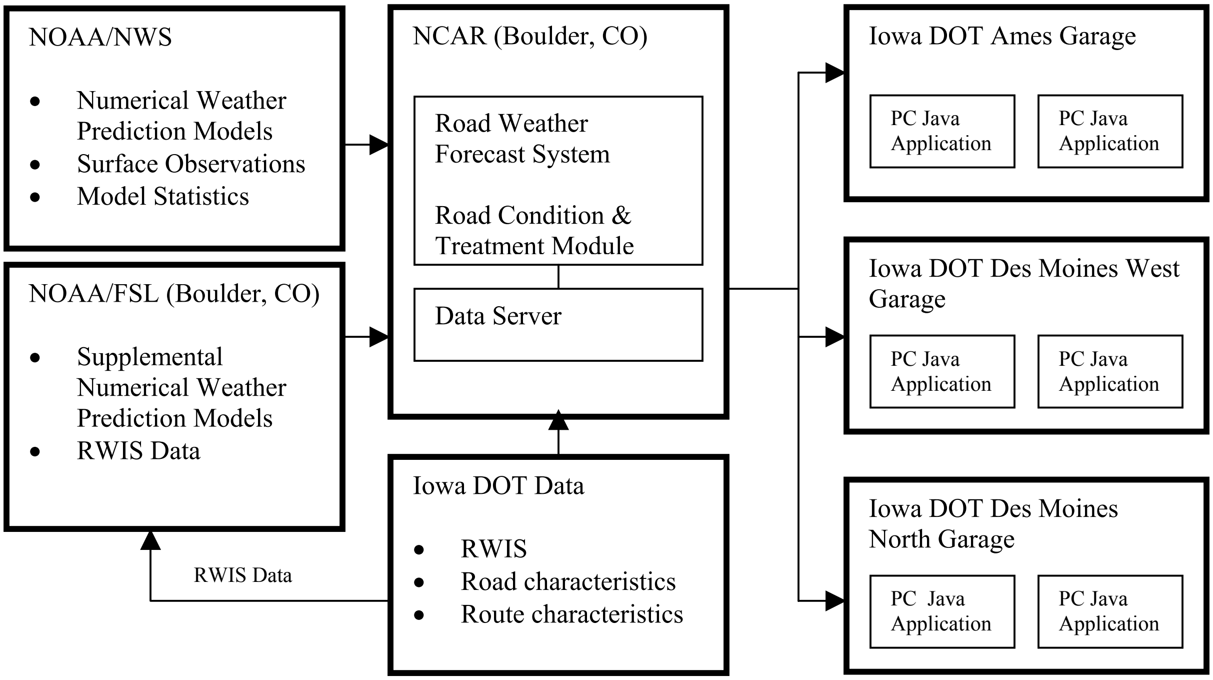


FIGURE 3. Depiction of the MDSS Prototype Configuration for the Iowa Field Demonstration. All Network MDSS Connections to the Sites will be Via the Internet (Figure Courtesy of NCAR).

FIELD–TEST PERIOD

The national labs were responsible for preparing a technical performance assessment by performing data analyses that seek to answer questions related to the technical performance of the MDSS system. Iowa DOT worked with the labs to identify critical ground truth data sets.

From the period of February 3-April 7, 2003 the MDSS was field–tested. There were a total of eight weather events that tested the system, in various stages of intensity.

System Tests

Total Weather incidents:	8
Light Snow Events	5
Heavy Snow Events	3
Mix: Snow/Rain/Ice	1

Data Sources

Iowa DOT also provided field weather and operational data from the garages to verify the model. Data were obtained from several sources. Where available, data were obtained and archived in real time. Archived data were used if not available in real time. The following data were collected for verification:

a) Iowa RWIS (weather and road condition data)
b) NWS METAR (aviation observations)
c) Local observer surface data (where available)
d) Weather satellite
e) Weather radar
f) NWS storm summaries
g) Iowa DOT observations (where available)
h) Iowa DOT Maintenance Concept Vehicle data
    i. Air temperature
    ii. Pavement temperature
    iii. Material distribution setting
    iv. Freezing point detection
    v. Treatment type
    vi. Treatment rate
    vii. Plow position

Data Collection Forms

In order to fully estimate the road conditions and determine the actual treatments performed during each event, it was necessary for Iowa DOT personnel to fill out data forms following each shift that required winter road treatments.

Numerous iterations of the format and content for these forms were determined through discussions between Iowa DOT, the Labs and the FHWA.

The winter maintenance data collection forms captured the following information:

- Date
- Shift time
- Route ID
- Equipment type
- Treatment performed
    - Treatment start and stop times
    - Chemicals used ($NaCl$, $CaCl_2$, etc.)
    - Chemical amount (tonnage)
    - Plowing performed
- Estimated road condition per route
    - Wet, dry, icy, snow packed, blowing snow, snow depth, slush, rain, freezing rain, frost, etc.
    - Road temperature (where available from equipment)
- Any other pertinent observations such as chemical dispersion rate, condition of road before and after treatment, precipitation start and stop times.

MDSS Supplemental Daily Log: AMES GARAGE

Driver Name:		Truck ID:
Shift Start Date:	Shift Start Time: (AM/PM)	
Shift End Date:	Shift End Time: (AM/PM)	
Comments:		

Segment ID	Route	Start MP	End MP
1A	U.S. 65	98.38	112.09
1B	U.S. 30	164.93	172.30
2	U.S 65	112.09	132.59
3	I-35	111.60	128.46
4	I-35	96.60	111.60
5	U.S. 30	142.88	164.93
6	IA 210	13.79	34.43

Directions: Use 1 Column per Treatment. Use a New Column if one of the following applies:
1). you are starting a new segment
2). you make a significant change on the same segment (such as a change of application rate)
3). you begin a second treatment on the same segment
Mark up the map on the reverse side with any additional details that makes the report of your treatment more complete.

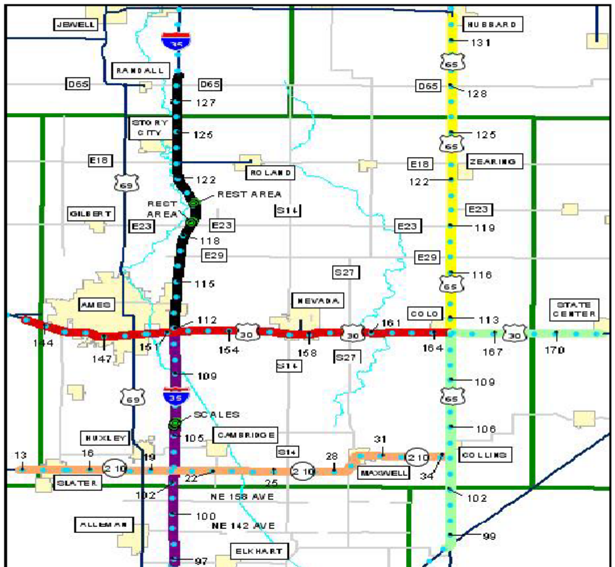
	Treatment 1	Treatment 2	Treatment 3	Treatment 4
Segment ID Covered (circle one)	1A 1B 2 3 4 5 6	1A 1B 2 3 4 5 6	1A 1B 2 3 4 5 6	1A 1B 2 3 4 5 6
Treatment Start Time	AM/PM	AM/PM	AM/PM	AM/PM
Treatment End Time	AM/PM	AM/PM	AM/PM	AM/PM
Did You Plow?	YES NO	YES NO	YES NO	YES NO
Lane Materials Applied On	Driving Center Passing	Driving Center Passing	Driving Center Passing	Driving Center Passing
Pre-Storm Chemical Application Note amount	YES NO Brine gal/lane-mile	YES NO Brine gal/lane-mile	YES NO Brine gal/lane-mile	YES NO Brine gal/lane-mile
During Storm Chemicals applied? (Circle type) Note amount	NONE Salt Calcium Iceban Iceslicer brine ____gal or lb/lane-mile	NONE Salt Calcium Iceban Iceslicer brine ____gal or lb/lane-mile	NONE Salt Calcium Iceban Iceslicer brine ____gal or lb/lane-mile	NONE Salt Calcium Iceban Iceslicer brine ____gal or lb/lane-mile
Abrasives applied? (amount)	______lb/lane-mile	______lb/lane-mile	______lb/lane-mile	______lb/lane-mile
Road condition before treatment (Circle all that apply)	dry wet slushy icy snow-packed road frost drifting snow	dry wet slushy icy snow-packed road frost drifting snow	dry wet slushy icy snow-packed road frost drifting snow	dry wet slushy icy snow-packed road frost drifting snow
Road condition after treatment (Circle all that apply)	dry wet slushy icy snow-packed drifted snow unknown	dry wet slushy icy snow-packed drifted snow unknown	dry wet slushy icy snow-packed drifted snow unknown	dry wet slushy icy snow-packed drifted snow unknown
Estimate Traffic Volume (circle one)	low normal high	low normal high	low normal high	low normal high
Traffic speed (circle one)	stopped slow normal	stopped slow normal	stopped slow normal	stopped slow normal
Treatment Specific Comments				

FIGURE 3. Data Collection Form for Ames Garage

Mark up this map with any information that adds detail to the treatment information from the front of this form (examples: show the treatment route, highlight areas of drifts, and indicate accident locations). Use the boxes provided to organize the details. Draw a line from the box to the highlighted region.

Treatment 1 Details

Treatment 2 Details



Segment ID	Color	Segment ID	Color
1A	Green	1B	Green
2	Yellow	3	Black
4	Purple	5	Red
6	Orange		

Treatment 3 Details

Treatment 4 Details

Ames_100A

FIGURE 4: Second Page of Data Collection Form

The data collected from the field will be incorporated into the MDSS improve the model.

CONCLUSIONS AND RECOMMENDATIONS

Following the field demonstration, a MDSS Stakeholders' meeting was held in Des Moines on June 17 – 18, 2003 to discuss the outcome of the project. Interested parties from across the USA, and other countries including Great Britain, Canada, were on hand to provide input to the MDSS project.

The following recommendations were put forth following this past winters' field demonstration:

The MDSS weather predictions need improvement. The MDSS failed to pick up light snow events. While these events do not produce a lot in precipitation, operationally, the garages still need to deploy personnel and equipment to clear the roadways. The garages also reported that the start time for events were not as accurate as they would have liked. To effectively deploy the pre–treatments, it is critical for the field supervisors to know the start time of precipitation events.

During the field–tests, the garage users also requested refinements in the display portion of the MDSS interface. The display on the screen uses dots to indicate weather conditions. The users of the systems asked that the weather data be shown, along with wind direction and velocity to more readily obtain the weather conditions.

An important aspect of the field test was to collect weather data from the garages and equipment operators. To collect the data we used paper data collection forms that the equipment operators and supervisors completed. This proved to be a cumbersome and time–consuming process. The field staff strongly recommends that further data collection be automated to allow the equipment operators to focus on the task at hand. Data, such as plow position, location, spreader rates, pavement temperature, etc. can be collected with the existing GPS and AVL units that Iowa DOT has. Other data, such as weather and traffic conditions will still have to be collected manually.

A further recommendation is to tie the MDSS into the chemical inventory system to track chemical usage and assist in the chemical inventory at optimal levels.

The MDSS shows real promise in assisting winter maintenance managers in fighting winter storms. If fully deployed, the MDSS could assist winter maintenance managers statewide with prompt, accurate, tactical information to alleviate the effects of winter weather on the roadways. By providing as much probabilistic weather forecasts and treatment recommendations as possible to the field–level supervisors, they can use those data to make better–informed decisions on clearing the roadways in a cost effective manner.

For more complete information on the Maintenance Decision Support System (MDSS) program, please refer to the following website:

http://www.rap.ucar.edu/projects/rdwx_mdss/index.html

REFERENCES

1. Tutorial on the Maintenance Decision Support System Field Demonstration Software. February 4, 2003. http://www.rap.ucar.edu/projects/rdwx_mdss/iowa.html

Cost-Effectiveness of Abrasives, Salt Brine, and LCS for Winter Operations in Nebraska

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ABSTRACT

The primary objective of winter maintenance operations is to improve traffic safety and efficiency during winter storm periods. Abrasives and salt brines have been successfully applied to increase traction and prevent snow and ice from bonding to road surface. However, because of some undesired side effects, such as corrosion and damage to the environment, salt and abrasives may need to be supplemented by other substances in some areas. Powerful non-corrosive acetate-based chemicals have been considered by several agencies, but their high price has limited their use. Recent research has focused on the use of some new, less corrosive, and highly effective chemicals, such as liquid corn salt (LCS). This research evaluates and compares the cost-effectiveness of using salt brine, and LCS on two highway sections in Nebraska. Field studies were conducted during the winter of 2002-2003. Field data included weather information, chemical use, time to achieve bare pavement, and other information available from maintenance logs. A benefit-cost analysis was performed to determine the cost-effectiveness of each treatment alternative. The operational benefits were the savings in road user costs resulting from reduction in travel time and delay. They were determined from field study data. The safety benefits related to accident reduction due to improved road surface conditions were estimated based on findings of previous research. The costs, including material and operational costs, were obtained from maintenance logs and a review of the relevant literature. The cost-effectiveness of salt brine and LCS were compared based on their benefit-cost ratios calculated over a range of ADTs and truck percentages. Guidelines were developed for the most appropriate use of these chemicals under various weather and traffic conditions.

Key words: abrasives—liquid corn salt—winter maintenance operations—salt brine

Note: Preparation of this paper was still in progress at the time of publication; final results will be presented at the symposium.

Planning the Mississippi River Trail in Iowa Using Geographic Information Systems

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ABSTRACT

Bicycle transportation in the U.S. has been increasingly supported through cooperation for new bicycle facilities among all levels of government. A prime example is Millennium Trails, a federal program devoted to trail development to recognize America's history and future. The Mississippi River Trail (MRT) is a Millennium Trail, envisioned to begin at the Mississippi headwaters in Minnesota and to end at the Gulf of Mexico. Iowa's rural MRT portion is being planned to provide economic and recreation opportunities, but also presents potential planning issues. The MRT includes urban and regional trails over varied terrain, includes off-road bicycle trails and on-road bicycle lanes, currently planned or programmed bicycle facilities, and must be aligned to maximize use of Iowa's riverside amenities while providing safe cycling conditions. Geographic information systems (GIS) were used to effectively analyze the trail planning factors of the Iowa MRT. GIS allow many levels of information to be simultaneously analyzed, and may process large datasets required for the bicycle level of service analysis used to determine safe trail routings. The bicycle level of service analysis outlined safe MRT on-road bicycle lanes, and also directed where off-road trails should be used instead. Verification of results with public officials and the public is key to validate the trail planning effectiveness with GIS. This process to develop the MRT has proven effective for Iowa and may be applicable to analyze trail development feasibility in other states.

Key words: bicycle facilities—bicycle level of service analysis—bicycle trails planning—geographic information systems—multimodal, alternative transportation

Proceedings of the 2003 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2003.

INTRODUCTION

Bicycling has grown in recent years as both a rewarding recreational activity and a viable mode of personal transportation alternative to the automobile. The United States is working to develop and maintain bicycle policies to increase both bicycle facilities and the level of bicycle use in the nation. One notable step towards welcoming alternative modes of transportation, such as the bicycle, onto U.S. transportation systems was the introduction of the Intermodal Surface Transportation Efficiency Act (ISTEA), which required states and metropolitan planning organizations (MPOs) to integrate bicycle and pedestrian planning into their long-range plans (*1*). This federal requirement urged cities and states to expand their bicycle and pedestrian facilities in a move to promote modes of transportation alternative to the automobile in urban and state plans.

In these transportation planning efforts, bicycle facility needs for all levels of cyclists have been taken under consideration. Generally, there are three types of cyclists: Level A, or advanced, Level B, or basic, and Level C, children (*2*). While there are advanced cyclists are accustomed to sharing facilities with automobiles, there are many more basic cyclists and children that are not comfortable with sharing roadways with bicycles. Because there are more basic cyclists and children than advanced cyclists, separately designated bicycle facilities are needed to better serve these populations (*2*). In this, the designation of public funds, time, and efforts towards adding designated bicycle facilities such as bicycle trails or lanes to our current transportation system will allow the majority of cyclists more access to more places in the United States.

The Millennium Trails represent an idea formed by the Clinton administration in the 1990s to honor U.S. history through trails and natural areas to “honor the past and imagine the future” of the United States (*3*). There are many planned or existing Millennium Trail projects in the United States at this time, but the Mississippi River Trail (MRT) represents a unique opportunity to connect the communities along the approximately 2000 miles of the Mississippi River by bicycle. From the Mississippi headwaters in Lake Itasca in Minnesota to its convergence with the Gulf of Mexico, the MRT can serve short-distance as well as long-distance cyclists, connecting people and places by this alternative mode of transportation. In addition, the trail will be composed of both bicycle trails and lanes; the different facilities along the route will cater to different levels of cyclists, from advanced and basic cyclists on the on-road bicycle lanes, to children on the off-road bicycle trails.

The rural Iowa portion of the Mississippi River Trail will provide connections by bicycle between communities along the river, through both existing bicycle facilities and facilities that will be designed and constructed expressly for the MRT. The urban sections of the MRT in Iowa are being planned and constructed by local groups using a combination of on-street and off-street bicycle facilities. To ensure a well-rounded trail plan, the Iowa Mississippi River Trail Advisory Committee was formed to brainstorm as well as to review potential plans for the rural segment of the trail. The advisory committee provided support from numerous organizations, clubs, and state, regional, and local agencies as well as give input on trail design from local cyclist perspectives. The advisory committee contributed to the geographic information systems (GIS) trail development effort by providing insights on future bicycle facilities to incorporate, amenities that could interest cyclists, and other information pertinent to the development process.

This paper will document the use of GIS to develop a recommended alignment for the rural portion of the Mississippi River Trail in the state of Iowa. The use of GIS in this process allows for simultaneous analysis of physical land attributes such as topography and hydrology, roadway attributes such as average daily traffic counts and lane widths, political attributes such as county and city borders, and trail amenities and potential development problems.

A current problem in bicycle facility planning is that no standardized methods of bicycle trail or lane planning in GIS exist. Because no standardized methods of bicycle facility planning in GIS currently exist, undertaking this task at any jurisdictional level can be daunting. While documenting the process used to develop the MRT in Iowa, this paper will introduce a method to develop trails with GIS that can be replicated by state, regional, and local agencies to better develop bicycle facilities. The use of GIS for trail development will be compared to previous methods of trail development used by statewide agencies. In addition, the bicycle level of service (BLOS) and bicycle compatibility index (BCI) will be introduced as methods to create safer bicycle facilities. Then, the BLOS study will be combined with area amenities, trail development concerns, existing or planned bicycle facilities, as well as other factors, to determine the best routing of the MRT.

The MRT plan created through this process is for the rural segments of the MRT between communities along the Mississippi River. The Mississippi River Trail Advisory Committee wished to create a trail plan for rural areas that also provides a precedent and recommendations for cities to follow when developing their own segments of the MRT. Therefore, the resulting MRT plan for Iowa only shows rural trails that end at cities' political boundaries. By knowing where the rural trail is recommended to terminate at their borders, cities are better equipped to create their own bicycle facility programs.

BACKGROUND

There are five basic types of bicycle facilities used in the United States today: shared lanes, wide outside lanes, bicycle lanes, shoulders, and separate bicycle trails (*2*). Out of these facilities, bicycle lanes and bicycle trails are specifically designated as bicycle facilities separate from automobile facilities. Because these facilities do not require cyclists to share the same facilities as automobiles, they are more suitable for B level cyclists than shared-roadway bicycle facilities. Because of this, the MRT in Iowa will use both bicycle lanes and trails; because the MRT will be a dynamically developed facility, a move from initially building bicycle lanes to the development of off-road bicycle trails is encouraged. C level cyclists, children, are not recommended to use MRT bicycle lanes even though they are designated separate lanes from those of automobiles; children may not have full control of their bicycles at all times, and could swerve into automobile traffic. However, children are encouraged to instead use the bicycle trail sections of the MRT. Because the MRT is recommended to be both bicycle lanes and trails, a method to build proper facilities along the routing is essential.

Previous methods of bicycle facility planning that have not used GIS technologies could be user-based or project-based. User-based methods analyze which levels of cyclist the facility would be planned for, and recommend appropriate facilities based on its users. In addition to this, bicycle planners decided where facilities should go, what type of facilities should be used, and how their jurisdictions could pay for the improvements. To better determine how user needs compared to bicycle facility type, cost, and feasibility, many bicycle planners used the BLOS or BCI methods to analyze bicycle suitability in their regions (*4*). These models include roadway variables that affect cyclist comfort and safety, including: average annual daily traffic counts, traffic speed, width of the right-hand motor vehicle lane, percentage of heavy vehicles, on-street parking types, and condition of the pavement surface (*4*). The BLOS model measures cyclists' perceived comfort and safety while riding, and is rated much like the roadway level of service method, on an A–F scale.

While these previous methods of bicycle facility planning have produced reliable, well-matched bicycle facilities to satisfy user needs and project feasibility, methods like BLOS are more difficult to apply on a regional or statewide level. BLOS requires calculations involving detailed roadway data on individual road segments, and the sheer volume of road segments that could be involved in a regional or statewide bicycle facility make the manual application of BLOS a daunting task. An answer to this problem is found in GIS technologies, which, as outlined in the next section, were used to produce reliable and fast results for the MRT BLOS analysis.

METHODOLOGY

The MRT in Iowa is planned to be a combination of both bicycle lanes and trails to better service all levels of cyclists along the routing. Because cyclist safety and efficient use of public resources are of utmost importance in creating the MRT, the methodology used to create a trail routing must reflect these factors. The BLOS was used to ultimately create a trail routing that reflected these factors, and this analysis was conducted in a GIS for reliable, visual, and fast results.

Route Evaluation and Mapping Using GIS

GIS were used to compile all information about the MRT to create the interim route. First, inventories of project area maps were made using base geographic information for the ten counties. After this, attribute maps of trail amenities and trail concern areas were created. Then, the BLOS analysis was performed within GIS to graphically display the BLOS rankings of individual corridors to make safe decisions on trail placement. To further analyze the safety of the potential routing, a shoulder improvements analysis was performed to determine each corridor's feasibility to carry a bicycle lane after adding paved shoulders. The interim route was determined by comparing results for the above listed analyses.

The ten Iowa counties along the Mississippi River represent a range of topographic, social, cultural, and physical differences that needed to be accounted for during MRT planning. First, an inventory of these counties included a collection of their amenities, potential trail development problems, existing bicycle facilities, programmed or planned bicycle facilities, and paved roads within 10 miles of the river for potential MRT bicycle lanes. This inventory of trail amenities or possible trail development concerns along the river was essential to identify areas where trail building is feasible and would highlight amenities. An example of an inventory map is seen in Figure 1, a trail amenities map for Scott County.

After the inventory of existing, programmed, and planned bicycle facilities along the Mississippi River in Iowa, gaps where no existing, programmed, or planned bicycle facilities were apparent. These gaps are where bicycle lanes or trails must be constructed in order to complete the MRT in Iowa. However, because existing facilities were not available in these gaps in the MRT, two issues made bicycle lane or trail planning complex here. First, cyclist safety was of utmost importance to the MRT, so all recommended trail routings must be located either on off-road trails or on roads deemed to be safe for cyclists through the BLOS scale. Second, while off-road trails separated cyclists from almost all instance of conflicts with motor vehicles, the cost of building all off-road trails for the MRT all at once would be much more money than the state, counties, or cities along the route could afford. In order to get the MRT started, the majority of the MRT needed to be placed on less-expensive six-foot wide bicycle lanes, or on-road designated bicycle facilities. However, the recommended bicycle lanes did not have to be included in the permanent routing of the trail; in recommending the MRT first be routed on bicycle lanes, MRT planners also recommended that off-road bicycle trails be built to replace the bicycle lanes as funding becomes available. In addition to be being less expensive per mile to construct, paving roadway shoulders for bicycle lanes also provides an important safety benefit for motorized vehicles that continues even if the bicycle lanes are abandoned for off-road trails in the future.

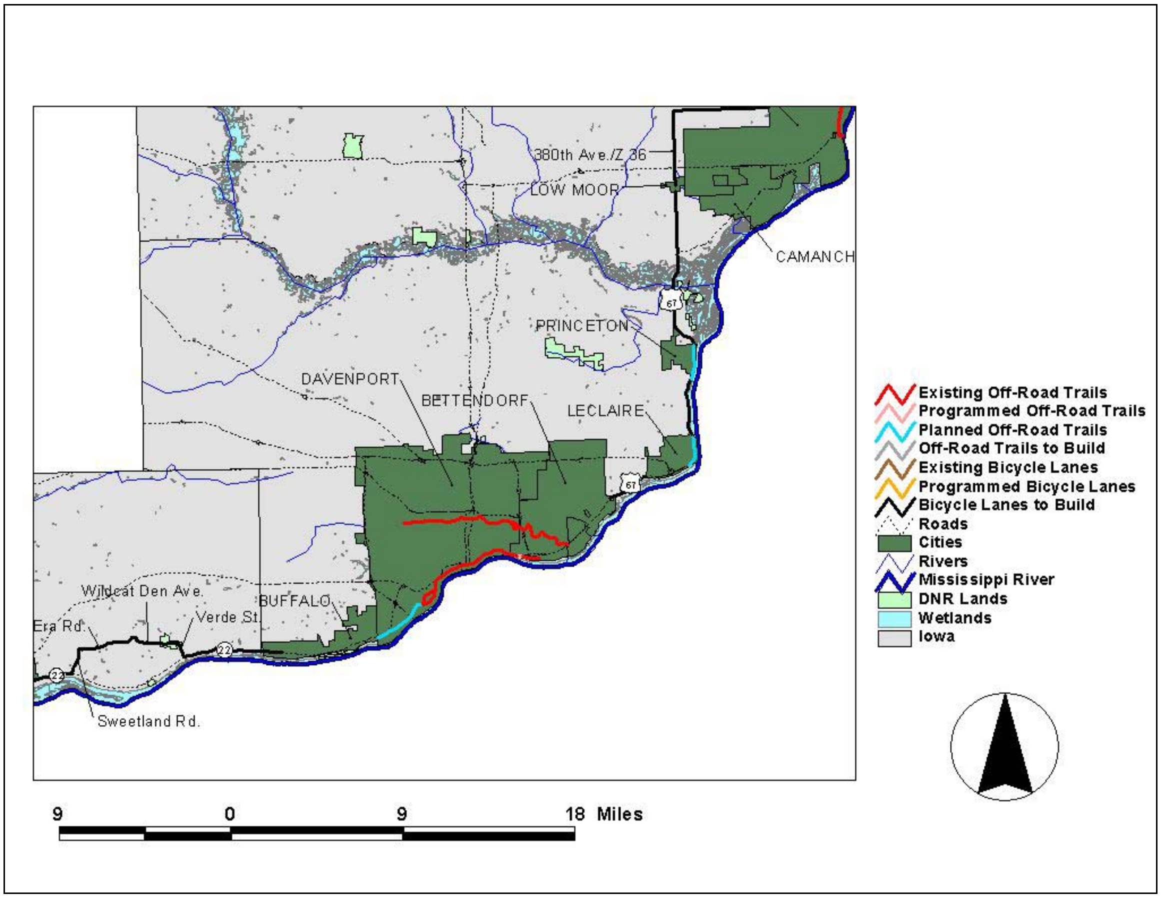

FIGURE 1. Scott County Trail Amenities

The development of bicycle lanes for the MRT will extend benefits to motorists as well as cyclists. Providing bicycle lanes for cyclists takes cyclists off the same travel path as automobiles and trucks. Also, the paved shoulders required for bicycle lanes provide safety benefits to motorists. A study to measure motorist safety benefits of paved shoulders by the Iowa Department of Transportation and the Center for Transportation Research and Education concluded that paved shoulders of at least 3 feet have been nationally shown to reduce associated motor vehicle crashes (*5*). In addition, the study recommends 6-foot wide shoulders for bicycle use, which is consistent with the recommendations of the Iowa Mississippi River Trail Advisory Committee and the bicycle level of service analysis used for the Iowa MRT.

In the context of the MRT in Iowa, BLOS represented a data-driven effort to design the Iowa portion of the MRT with the concept of bicyclist comfort and safety. While both provide a good measure for bicycle lanes, the MRT planners and advisors chose to use Bicycle Level of Service rather than the Bicycle Compatibility Index because BLOS contained variables that were relevant for rural roads. The League of Illinois Bicyclists and the Chicagoland Bicyclist Association derived the BLOS used for MRT planning (*6*). BLOS is used to estimate the safety and comfort of the cyclist. The BLOS scale ranges from A (extremely high compatibility) to F (extremely low compatibility); however, MRT trail planners and advisors decided the lowest acceptable BLOS for the MRT could be a level of C.

BLOS uses roadway data to determine if a paved corridor is suitable for an on-road bicycle lane. Important roadway data used in the BLOS calculation includes number of lanes, lane width, paved shoulder width (where the bicycle lane would be placed), annual average daily traffic counts, percentage of heavy vehicles, and speed limit.

On-road bicycle lanes were preferred for the MRT in Iowa over the more expensive off-road trails, so each corridor was analyzed for its suitability for a bicycle lane. This was done through three methods: Bicycle Level of Service analysis, a shoulder improvements study, and field studies with public input. To fully assess each roadway's current potential for bicycle lanes, an initial BLOS analysis was performed in GIS using current roadway conditions for all roads in the ten counties within ten miles of the Mississippi River. The result of this analysis showed roads that would be adequate for bicycle lanes in their current states; no additional shoulder paving would be required to create bicycle lanes. However, the majority of roads did require additional shoulder paving in order to create safer bicycle lanes. To assess the remaining roads' viability to be converted into bicycle lanes after additional paved shoulder width is constructed, another BLOS analysis, the shoulder improvements study, was performed. This study used the current roadway traffic conditions in addition to a hypothetical shoulder width of 6 feet, the shoulder width that would be used if there were a bicycle lane on the road. The result of the shoulder improvements analysis showed the roads within the project area that could have bicycle lanes safely after additional widths of paved shoulders were constructed.

The two BLOS analyses resulted in a disjointed MRT routing of existing, programmed, and planned bicycle facilities and roadway segments that scored at least a C on either BLOS analysis. There were different routing alternatives to choose from, and there were still areas along the recommended route that did not yet connect to make a complete trail. At this point, the recommended routing was chosen by comparing route alternatives to the previously made maps of trail amenities and trail development concerns to the two BLOS analyses. The trail routing was chosen from the alternatives by determining which routing could best serve cyclists by the highest levels of amenities, aesthetics, and safety. An example of a shoulder improvements analysis map is seen in Figure 2, the shoulder improvements analysis map for Scott County.

After the trail alternatives were chosen, however, there remained three small gaps on the trail network between bicycle facilities where the nearest roadway was not found to be adequate to safely have bicycle lanes. At these points, the construction of off-road trails were recommended to finish the complete recommended MRT routing in Iowa.

To verify the accuracy of the trail development procedure in GIS, and in the interest of time for this process, planners went on site visits and drove the recommended routes for the MRT. The observed routes were considered adequate for the MRT, and with the construction of needed road improvements or off-road trails the MRT planning process in GIS seemed a success. Although the trail development process in GIS was deemed reliable through site visits and driving the recommended routes, public input was necessary to gain local opinions and insights on trail development and its impacts. Public meetings were held at three points along the recommended route: Lansing, Davenport, and Fort Madison, Iowa. In addition to public meetings, the draft MRT plan was posted on the Internet for public comment. These two methods of public comment generated new ideas about the MRT through Iowa. Some communities along the MRT recommended routing did not approve of the interim routing on bicycle lanes, for some of the on-road facilities were routed away from certain community amenities and river views for safety reasons. To solve this problem, these communities have decided to skip the interim routing of the MRT on bicycle lanes and have instead committed to building off-road bicycle trails to take advantage of amenities each community had to offer the MRT cyclists.

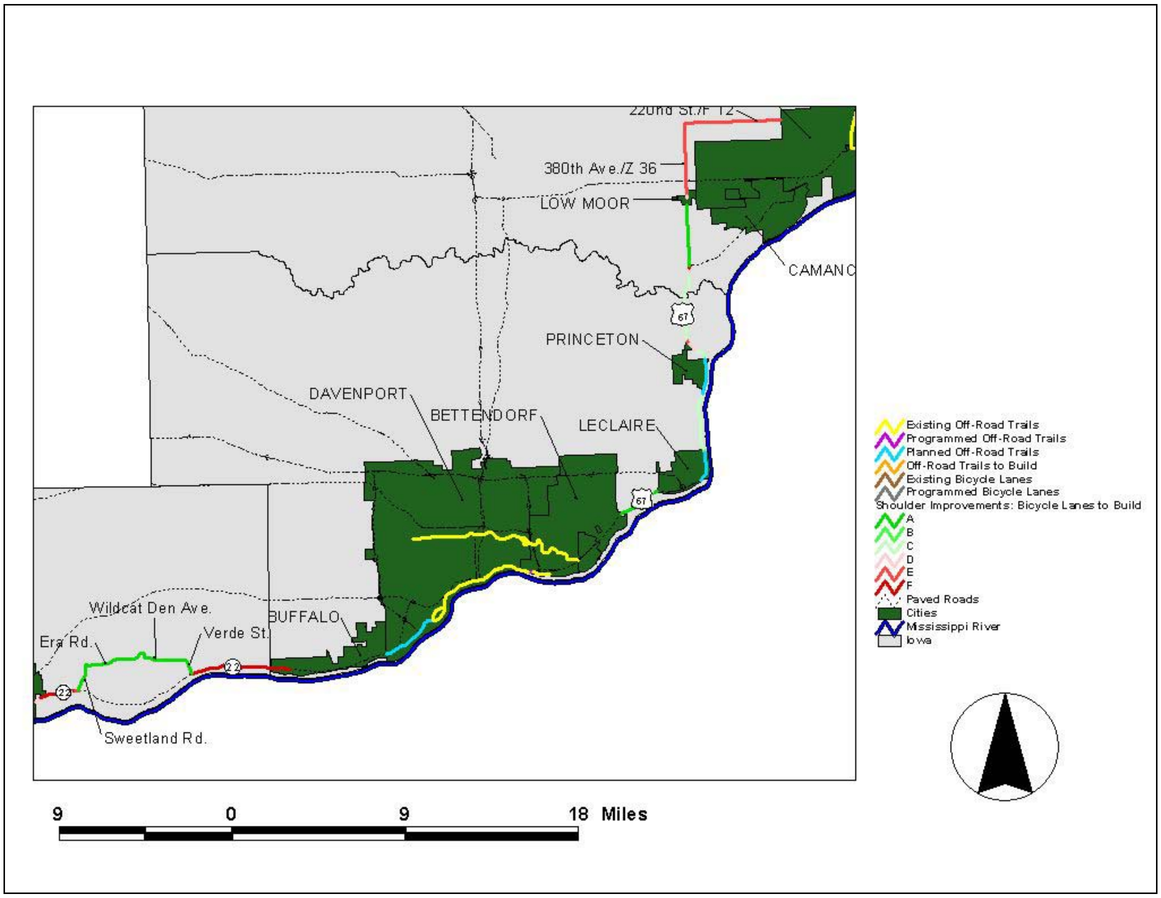

FIGURE 2. Scott County Shoulder Improvements Analysis

Cost Estimates

The costs of creating the Mississippi River Trail in Iowa are dependent upon construction projects required for each bicycle lane or trail to meet MRT standards. Bicycle lanes for the MRT will be created on roads by constructing six-foot asphalt shoulders at a cost of approximately $107,000 per mile (*7*). Non-motorized asphalt off-road bicycle trails 10 feet wide may be constructed at a cost of approximately $85,344 per mile. In addition to these costs, structural improvements may be necessary for the roadway to accommodate bicycle trails. Narrow structures on roadways may need to be redecked to include bicycle lanes at a cost of $50 per square foot, assuming a six-foot wide bicycle lane.

Because either county or state government will complete portions of the rural MRT, both county and state agencies need to be aware of their jurisdiction's unique responsibilities in MRT development. The state of Iowa will be responsible for an estimated 99 miles of state roadway to be improved for bicycle lanes, while individual counties will be responsible for approximately 125 miles of the Iowa MRT on county roads. There are also county jurisdiction off-road trails that will need to be funded. In addition, individual municipalities have jurisdiction over approximately 54 miles of the Iowa MRT, and each is responsible for developing their own MRT routing. Altogether, these mileages add up to approximately 278 miles of bicycle facilities in both rural and urban areas that will be created by the Iowa Mississippi River Trail. In this recommended rural trail mileage, 15.19 miles will be off-road bicycle trails, while 205.45 miles will be on-road bicycle lanes. The approximate cost, without the major expenses of land acquisition, trail design, and other costs, to create the off-road trails is approximately $740,000, while the cost to build the bicycle lanes, without striping and other costs, is approximately $22 million.

CONCLUSIONS

The plan for the MRT in Iowa outlines many individual bicycle facility projects that make up the complete trail. The trail plan should direct state and county bicycle facility programs, but should be used as a guideline for city bicycle facility programs. Trail planners, advisors, and others have deemed this routing safe through the BLOS analysis, shoulder improvements analysis, field reviews, and public input. However, because the trail routing has been recommended and analyzed for safety, there are no guarantees that it will be developed as outlined in the MRT plan. Some cities along the MRT route are concerned that bicycle lanes rather than trails are being recommended; some of these areas are taking the initiative to plan for off-road trails immediately, rather than first building bicycle lanes. Although the trail routing is not always agreed upon, the idea of the MRT has been positively received in all levels of government in the project area.

Planning for the Iowa portion of the Mississippi River Trail in geographic information systems not only allowed for data to be spatially displayed with ease, but also enabled planners to quickly analyze existing conditions for bicycle facilities in the large project area. GIS was especially beneficial to use for the MRT because it is a long distance rural trail; many road segments, existing facilities, and other data were incorporated into analysis, which would have been very time consuming if calculated manually. The results of all analyses for the MRT were spatially displayed easily, which allowed for better decision-making based on all levels of analysis. Another important benefit of using GIS in bicycle facility planning is that the process used to plan for the MRT can be easily replicated for other areas. The trail amenities, trail development concerns, bicycle level of service, and shoulder improvement analysis helped to develop the trail digitally to be a safe facility, but the accompanying field reviews and public input verified the accuracy of the GIS analysis. A combination of GIS analysis with field reviews and public input for verification can be easily replicated in other areas for bicycle facility planning.

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Impact of Modern Roundabouts on Vehicular Emissions

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ABSTRACT

Vehicular emissions have increased considerably over the years with the increase in traffic. Modern roundabouts can improve traffic flow as well as cut down vehicular emissions and fuel consumption by reducing the vehicle idle time at intersections and thereby creating a positive impact on the environment.

The primary focus of this research is to study the impact of modern roundabouts in cutting down vehicular emissions. Six sites with different traffic volume ranges, where a modern roundabout has replaced Stop controlled intersection, have been chosen for the study. The operation of the roadways at the intersection was videotaped and the traffic flow data was extracted from these tapes and analyzed using Signalized and Unsignalized Intersection Design and Research Aid (SIDRA) software. The version used is a.a. SIDRA 2.0. The software produces many measures of effectiveness (MOEs) of which five were chosen for analyzing the environmental impact of roundabouts. The chosen five outputs give rate of emission of HC, CO, NO_X, and CO_2 in (kg/hr).

All the MOEs were statistically compared to determine which intersection control performed better. After comparing all the MOEs at all locations for the before and after traffic volumes, it was found that the modern roundabout performed better than the existing intersection control (i.e., stop signs) in cutting down vehicular emissions thereby creating a positive impact on the environment. The research concludes that a modern roundabout can be used, as a viable alternative to cut down vehicular emissions and thereby making intersections more environment friendly.

Key words: modern roundabouts—reducing air pollution—vehicular emissions

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INTRODUCTION

With the increase in traffic over the years, one of the major threats to clean air in many of the developed countries like the U.S. is vehicular emissions. Problems posed by the environmental impact of traffic are growing and are a challenge for traffic engineers. Vehicular emissions are dependent on the total amount of traffic, intersection control type, driving patterns and vehicular characteristics.

Vehicular emissions contain a wide variety of pollutants, principally carbon monoxide (CO), carbon dioxide (CO_2), oxides of nitrogen (NO_X), particulate matter (PM_{10}) and hydrocarbons (HC) or volatile organic compounds (VOCs) which have a major long term impact on air quality. These emissions vary with the engine design, the air-to-fuel ratio, and vehicle operating characteristics. With increasing vehicle speed there is an increase in NO_X emissions and decrease in CO, PM_{10} and HC or VOC emissions. The emissions of (CO_2) and oxides of sulfur (SO_x) vary directly with fuel consumption and for any given vehicle and fuel combination, aggregate emission levels vary according to the distance traveled and the driving patterns (*1*).

Road and street intersections force vehicular traffic to slow down and stop in varying patterns of interruption of ideal, constant traffic flow at an ideal speed. The longer the stops, the more fuel that is consumed and vehicular emissions increase. With the vehicular emissions problems worsening, it has become prudent to choose effective traffic control devices (TCDs) that can improve traffic flow on the roads and reduce emissions per vehicle kilometer traveled while enhancing mobility.

Modern roundabouts in the U.S., which are functioning as one of the safest and most efficient forms of intersection control (*4, 14, 15*) and improving traffic flow at intersections, have the additional advantage of cutting down vehicular emissions and fuel consumption by reducing vehicle idling time at the intersections and thereby having a positive affect on the environment.

The primary focus of this research is to study the impact of modern roundabouts in cutting down vehicular emissions at intersections. This research focuses on six sites with different traffic volume ranges where a modern roundabout has replaced stop-controlled intersection. The emissions at the intersections were compared for the before (stop controlled) and after (modern roundabout) conditions to assess the impact of a modern roundabout at these intersections.

LITERATURE REVIEW

Vehicle exhaust fumes played a major role in the deterioration of air quality in urban areas since 1950s and as a result the Clean Air Act (CAA) was passed in 1970. The CAA gives the Environmental Protection Agency (EPA) the authority to set limits on emission standards. The EPA estimates that over 5,000 tons of VOCs from transportation sources were emitted in 1999 and that approximately 62 million people living in areas that do not meet health based standards. EPA also estimated that in 1999 the transportation sector, including on-road and non-road vehicles, contributed to 47 percent of hydrocarbon (HC) emissions, 55 percent of nitrogen oxides (NO_X) emissions, 77 percent of carbon monoxide (CO) emissions, and 25 percent of particulate matter (PM) emissions (*2*).

Roundabouts are being implemented throughout the U.S. in a variety of situations. Many states and cities are considering roundabouts as a viable alternative to other TCD's, and, in some cases, complex freeway interchanges. Modern roundabouts are becoming popular in the US for more than just safety reasons. As stated in an article by the Insurance Institute for Highway Safety (IIHS) they reduce fuel consumption and vehicular emissions by reducing stopping at intersections, and also reduce noise levels by making the traffic flow orderly. Modern roundabouts can enhance the aesthetics of the place and create visual gateways to communities or neighborhoods. In commercial areas they can improve access to adjacent properties (*3*).

Vehicles stopping at traffic signals and stop signs emit more carbon dioxide (CO_2) when compared to roundabouts as the delay and queuing are greater. Even if the delays are similar to that of roundabout, traffic signals always queue traffic at a red light and hence emissions are usually greater. The average delays at roundabouts have to be significantly larger than at traffic signals for the emissions to be equal. When traffic volumes are low, traffic rarely stops at a roundabout and the emissions are very small (*5, 6*).

When roundabouts become very congested with large queues, the emissions equal those at traffic signals. During off-peak hours roundabouts do not experience long queues and delays and the emissions are low. Traffic signals and stop signs stop vehicles even during off-peak hours and thereby experience higher delays and emissions. United Kingdom (UK) engineers believe that traffic signals have lower emissions only in exceptional cases (*5, 6*).

As stated by Barry Crown, a roundabout expert from the UK: "When vehicles are idle in a queue they emit about 7 times as much carbon monoxide (CO) as vehicles traveling at 10 mph. The emissions from a stopped vehicle are about 4.5 times greater than a vehicle moving at 5 MPH" (*5*).

The Bärenkreuzung/Zollikofen project undertaken in Bern, Switzerland, replaced two important signalized intersections by roundabouts and the result was a reduction of emissions and fuel savings by about 17 percent. The roundabouts also steadied the driving patterns (*7*).

On a microscale there have been studies conducted on the effect that different traffic flows have on emissions at an intersection. Of the studies that reported quantitative results, roundabouts reduced vehicle emissions for hydrocarbons (HC) in 5 studies by an average of 33 percent, carbon monoxide (CO) in 6 studies by an average of 36 percent, and nitric oxides (NOx) in 6 studies by an average of 21 percent. The regional scale air quality benefits of roundabouts would depend on their percent contribution to regional mobile source emissions (*8, 9*).

In a study conducted by Mustafa et.al (1993), the authors concluded that there exists a direct relationship between vehicle emissions and traffic volumes at urban intersections regardless of traffic control. Their simulation results showed that traffic signals generate more emissions (almost 50 percent higher) than a roundabout. In case of higher traffic volumes the HC generated by traffic signals is twice as high as that generated at roundabouts (*10*).

In another study conducted by Varhelyi in Sweden, he found that replacing a signalized intersection with a roundabout resulted in an average decrease in CO emissions by 29 percent and NOx emissions by 21 percent and fuel consumption by 28 percent per car within the influence of the junction (*11*).

Results of a study conducted by Jarkko Niittymaki show fuel consumption reductions of 30 percent in an intersection designed as a roundabout instead of using traffic signals and environmentally optimized traffic control systems have proved an energy saving potential of 10 percent to 20 percent in different cases (*12*).

METHODOLOGY

Description of Study Sites

Six study sites were selected for this research. Five of the sites are in Kansas and one in Nevada. Of the sites studied in Kansas two were in Olathe, one in Lawrence, one in Hutchinson, and one in Paola. Data from these sites was available from previous roundabout studies at Kansas State University (KSU) (*4, 14, 15*).

The sites in Olathe are (1) the intersection of the Ridgeview Road and Sheridan Avenue and (2) the intersection of Rogers Road and Sheridan Avenue. Sheridan Avenue runs in the East-West direction while the Ridgeview and Rogers Roads run in the North-South direction, roughly parallel to Interstate 35 (I-35).

The site in Lawrence is the T-intersection of the Harvard Road and Monterey Way. Harvard Road runs in the east-west direction while and ends at Monterey Way, which runs in the north-south direction.

The site in Hutchinson is the intersection of 23rd Street and Severance Avenue. Severance Avenue runs in the north-south direction and 23rd Street runs in the east-west direction.

The site in Paola is the intersection of the Old KC Road, State Route K68, and Hedge Lane. The Old KC Road runs in the north-south direction. The K68 runs in the east-west direction. Hedge Lane runs in Southeast- northwest direction, and intersects K68 just east of the K-68 and Old KC Road intersection.

The site in Nevada is the intersection of the Wedekind Road and ClearAcre Lane. Wedekind Road runs in the east-west direction while ClearAcre Lane runs in the north-south direction.

All the sites except Hutchinson and Nevada (which had a two-way stop control, TWSC) were controlled by stop signs on all approaches (all-way stop control, AWSC) prior to the installation of the modern roundabout. The major drawback of AWSC is that the presence of vehicles on all the approaches of the intersection will result in longer departure headways and longer driver decision times that reduce the capacity of the intersection. The major drawback of TWSC is that congestion on the minor street caused by a demand that exceeds capacity, and queues that form on the major street because of inadequate capacity for left turning vehicles yielding to opposing traffic. In the after condition a single-lane modern roundabout was built at all sites. The Paola roundabout is different from the others because it has five legs, and is an intersection on the state highway (*4*). See Table 1 for the intersection hourly traffic volume ranges and the percentage of left turn for the intersections studied.

TABLE 1. Intersection Hourly Traffic Volume Ranges and Percentages of Left Turns

PAOLA DATA			
AM (AWSC)	**AM (Roundabout)**	**PM (AWSC)**	**PM (Roundabout)**
257-594 (veh/hr) 28% left turns	235-559 (veh/hr) 29% left turns	192-690 (veh/hr) 38% left turns	156-663 (veh/hr) 40% left turns
LAWRENCE DATA			
AM (AWSC)	**AM (Roundabout)**	**PM (AWSC)**	**PM (Roundabout)**
227-536 (veh/hr) 30% left turns	263-447 (veh/hr) 17% left turns	412-733 (veh/hr) 26% left turns	442-692 (veh/hr) 21% left turns
OLATHE: ROGERS/SHERIDAN DATA			
AM (AWSC)	**AM (Roundabout)**	**PM (AWSC)**	**PM (Roundabout)**
926-1625 (veh/hr) 28% left turns	931-1738 (veh/hr) 28% left turns	1220-1994 (veh/hr) 21% left turns	1244-2024 (veh/hr) 22% left turns
OLATHE: RIDGEVIEW/SHERIDAN DATA			
AM (AWSC)	**AM (Roundabout)**	**PM (AWSC)**	**PM (Roundabout)**
708-1110 (veh/hr) 33% left turns	776-1124 (veh/hr) 33% left turns	1140-1626 (veh/hr) 35% left turns	1119-1784 (veh/hr) 38% left turns
HUTCHINSON DATA			
AM (TWSC)	**AM (Roundabout)**	**PM (TWSC)**	**PM (Roundabout)**
449-983 (veh/hr) 13% left turns	415-864 (veh/hr) 12% left turns	514-1204 (veh/hr) 13% left turns	501-1110 (veh/hr) 15% left turns
NEVADA DATA			
AM (TWSC)	**AM (Roundabout)**	**PM (TWSC)**	**PM (Roundabout)**
423-718 (veh/hr) 31% left turns	372-691 (veh/hr) 32% left turns	619-893 (veh/hr) 28% left turns	547-881 (veh/hr) 27% left turns

Data Collection

The available data had been collected in two phases. The first phase was videotaping intersection traffic movements with a video camera and the second phase was obtaining traffic counts visually from the videotapes (*4, 14, 15*).

Phase 1: Video Data Collection

The benefit of using this method for data collection is that all the data is recorded on videotapes and can be accessed and retrieved at a later time. In this method, all the information recorded on the tapes can be accessed for evaluation at any time and serves as a permanent record for re- verification of data, or reuse for other purposes. A specially designed 360-degree omni directional, video camera and videocassette recorder were used for data collection at each location. The camera was designed to provide a full 360 degrees view when mounted above the intersection. The camera was placed near the intersection to see the traffic flow coming toward and leaving the intersection. The camera was installed on existing poles and mounted perpendicular to the ground. The perpendicular mounting allowed the video image to be relatively distortion free to the horizon in all directions. The camera was mounted approximately 6 meters (20 feet) above the ground. This mounting height provides a focal plane of approximately 40.5 meters by 54.0 meters (133 feet by 177 feet). The camera feed went in to a TV/VCR unit placed in a recycled traffic signal controller cabinet. All the equipment was mounted on a single pole. The video images were recorded on standard VHS videotapes. See Figure 1 for details (*13, 14*). The traffic counts from the intersection were video taped for two six-hour sessions from 7:00 AM to 1:00 PM and from 1:00 PM to 7:00 PM on normal week days for the before and after conditions. A normal day in this study refers to a day with no adverse environmental/weather or any external factor(s), such as special events in the nearby locality of the study intersection that would impact the flow of traffic through the study intersection.

FIGURE 1. Camera and TV/VCR Units Used in Data Collection

Phase 2: Visual Data Collection

In this phase the data was visually collected from the videotapes. All the videotapes were studied visually to extract the traffic volumes and turning movements for the analysis. Various graduate student research assistants in the Department of Civil Engineering at KSU did the data extraction from the videotapes. Every vehicle coming from all the approaches for a period of 15 minutes was recorded on pre-prepared data collection sheets. Hourly counts were used as input data for analysis using the computer program Signalized and Unsignalized Intersection Design and Research Aid (aaSIDRA) (*15*).

Software Selection

The software used for data analysis is aaSIDRA, Version 2.0. The Australian Road Research Board (ARRB), Transport Research Ltd., originally developed the SIDRA package as an aid for design and evaluation of intersections such as signalized intersections; roundabouts, two-way stop control, and yield-sign control intersections. SIDRA was taken over by a private company that now supports the software. aaSIDRA 2.0 is the latest version.

> In evaluating and computing the performance of intersection controls there are some advantages that the SIDRA model has over any other software model. The SIDRA method emphasizes the consistency of capacity and performance analysis methods for roundabouts, sign-controlled, and signalized intersections through the use of an integrated modeling framework. Another strength of SIDRA is that it is based on the *U.S. Highway Capacity Manual* as well as Australian Road Research Board research results. (*16*)

The input to the software includes the road geometry, traffic counts, turning movements, and speed of the vehicles. The SIDRA software analyzes the data and the output provides measures of effectiveness from which the performance of the roadway can be determined. There are 19 measures of effectiveness given in SIDRA output but only four of them were considered relevant to the project. The four measures of effectiveness (MOEs) used in evaluating the performance are as follows:

- Carbon Monoxide (CO)
- Carbon Dioxide (CO_2)
- Nitrogen Oxides (NOx)
- Hydrocarbons (HC) or VOCs

SIDRA uses a four-mode elemental model for estimating fuel consumption, operating cost and pollutant emissions for all types of traffic facilities. This helps with estimation of air quality, energy and cost implications of alternative intersection design. For this purpose, a unique vehicle drive-cycle model (acceleration, deceleration, idling, cruise) is used. See Figure 2 for details (*17*).

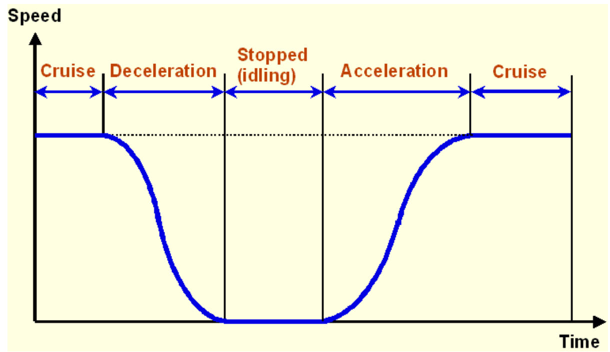

FIGURE 2. Graphical Representation of Drive-Cycle Model Used by SIDRA

Fuel consumption and emission rates are calculated from a set of equations which use such vehicle parameters as mass and fuel emission efficiency rates, as well as road grade and relevant speeds (cruise, initial, final).

Data Analysis

The data collected from videotapes for the AM and PM periods was recorded manually in 15-minute periods, and hourly data was then input to the SIDRA software for analysis. All the MOEs were statistically compared using standard statistical procedures. Minitab 13 was the software used to perform the statistical tests. The data analysis was done separately for the AM and PM hourly volumes but the procedure followed was the same for both sets of data. This was done to see whether the results differed due to the differences in before and after traffic volumes for both AM and PM traffic counts, as there was more traffic during the PM period than during the AM period.

RESULTS

The statistical analysis of the MOEs helps determine if and how the Stop controlled Intersections and the Roundabout controlled Intersections differed in cutting down vehicular emissions. The analysis provides information to assess characteristics of the Stop Controls and the Roundabout. The statistical testing was done separately for the AM and PM periods for all the locations in order to evaluate the operation of the intersection during these separate periods. The results obtained for each site are then averaged and the overall results are given in Table 2.

TABLE 2. Overall Emissions Results

EMISSIONS RESULTS				
AM Results				
Measures Of Effectiveness	**SC**	**R.A**	**% Diff.**	**Statistically Different**
Carbon Monoxide (CO) Kg/Hr	9.77	7.67	-21%	Yes
Carbon Dioxide (CO2) Kg/Hr	138.91	117.18	-16%	Yes
Oxides Of Nitrogen (NOX) Kg/Hr	0.31	0.25	-20%	Yes
HydroCarbons (HC) Kg/Hr	0.23	0.19	-18%	Yes
PM Results				
Measures Of Effectiveness	**SC**	**R.A**	**% Diff.**	**Statistically Different**
Carbon Monoxide (CO) Kg/Hr	11.8225	6.855	-42%	Yes
Carbon Dioxide (CO2) Kg/Hr	335.7	138	-59%	Yes
Oxides Of Nitrogen (NOX) Kg/Hr	0.3875	0.2015	-48%	Yes
HydroCarbons (HC) Kg/Hr	0.662375	0.23	-65%	Yes

Note: SC: AWSC or TWSC, RA: Roundabout

- The average Carbon Monoxide (CO) emissions (Kg/hr) for the intersection locations studied are 21 percent and 42 percent less for the AM period and PM periods respectively for the case of a modern roundabout. Statistical tests showed that the decrease in CO emissions after a roundabout was installed is statistically different from the emissions that occurred in case of AWSC for both AM and PM conditions.
- The average Carbon Dioxide (CO_2) emissions (Kg/hr) for the intersection locations studied are 16 percent and 59 percent less for the AM period and PM periods respectively for the case of a modern roundabout. Statistical tests showed that the decrease in CO_2 emissions after a roundabout was installed is statistically different from the emissions that occurred in case of AWSC for both AM and PM conditions.
- The average Oxides of Nitrogen (Nox) emissions (Kg/hr) for the intersection locations studied are 20 percent and 48 percent less for the AM period and PM periods respectively for the case of a modern roundabout. Statistical tests showed that the decrease in NOx emissions after a roundabout was installed is statistically different from the emissions that occurred in case of AWSC for both AM and PM conditions.
- The average Hydrocarbons (HC) emissions (Kg/hr) for the intersection locations studied are 18 percent and 65 percent less for the AM period and PM periods respectively for the case of a modern roundabout. Statistical tests showed that the decrease in HC emissions after a roundabout was installed is statistically different from the emissions that occurred in case of AWSC for both AM and PM conditions.
- The results from SIDRA analysis also showed that there was a statistically significant decrease in delay, queuing and stopping after the modern roundabout was installed when compared to the before (AWSC/TWSC) because, as previous studies have concluded, the modern roundabouts have less delay, queuing and stopping than an AWSC/TWSC. This is reflected in the decrease in vehicular emissions shown above.

CONCLUSIONS

- The modern roundabouts in Kansas operated more effectively than the before intersection control (AWSC/TWSC) in reducing vehicular emissions at all locations studied.
- There was a (21 percent to 42 percent) decrease in the Carbon Monoxide (CO) emissions (Kg/hr) for the AM and PM periods after the installation of modern roundabout. The decrease was observed to be statistically significant for both periods.
- There was a (16 percent to 59 percent) decrease in the Carbon Dioxide (CO_2) emissions (Kg/hr) for the AM and PM periods after the installation of modern roundabout. The decrease was observed to be statistically significant for both periods.
- There was a (20 percent to 48 percent) decrease in the Oxides of Nitrogen (NOx) emissions (Kg/hr) for the AM and PM periods after the installation of modern roundabout. The decrease was observed to be statistically significant for both periods.
- There was a (18 percent to 65 percent) decrease in the Hydrocarbons (HC) emissions (Kg/hr) for the AM and PM periods after the installation of modern roundabout. The decrease was observed to be statistically significant for both periods.
- Reduction in delays, queues and proportion of vehicle stopped at the intersection in the case of roundabouts suggest that roundabouts enhanced the operational performance of the intersections and account for the reduction in vehicular emissions.
- Since all the locations had a range of different traffic conditions, it is reasonable to suggest that a modern roundabout may be the best intersection alternative to reduce vehicular emissions for several other locations in Kansas with similar ranges of traffic volumes.

Overall Conclusion

Considering the above summary, it is concluded that at the intersections studied the modern roundabouts studied significantly reduced the vehicular emissions of the intersections studied by making the traffic flow orderly.

Further Study

Further studies should be conducted in other locations in United States with different traffic conditions, particularly those where volumes are high enough that a multi-lane roundabout is required, in order to get a much clearer picture. Also, field studies should be conducted using emissions detection equipment to further verify the results obtained from SIDRA.

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Collaboration, Automation, and Cognitive Aids Technologies to Solve Complex Transportation Problems

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ABSTRACT

It has always been the endeavor of scientists and engineers to develop systems that could mimic the human brain and adapt themselves to complex situations in order to arrive at a solution to the problem at hand in an efficient and effective manner. These systems would not only be capable of action but also of thought. Heuristic and algorithmic technologies are needed for the automation of cognitive activities. There is a need to develop tools, which are capable of assessing a situation at hand, alerting the human part of the system to various concerns and providing advice on possible responses, and possibly proceed towards solving the problem. This paper deals with the development of a human-machine optimal automated transportation system. In this paper it is proposed to develop specific algorithms to predict and avoid traffic congestion and collision. Two such tools that can be used with each other to produce optimal results are the Global Positioning System (GPS) and the Kalman filter. The Kalman filter is an algorithm, which can be used to predict the future state of a system based on the past information. This could be useful when we are dealing with typically, the problem of collision. A Kalman filter could be designed to give an accurate estimate of the future state of a highway or an intersection based on the data collected for that particular highway or intersection by GPS. The goal is to successfully use data fusion algorithms specifically like GPS and Kalman filters to improve the safety and efficiency of transportation systems particularly with respect to collision avoidance.

Key words: automated transportation system—global positioning system—Kalman filter

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INTRODUCTION

It has always been the endeavor of scientists and engineers to develop systems that could mimic the human brain and adapt themselves to complex situations in order to arrive at a solution to the problem at hand in an efficient and effective manner. These systems would not only be capable of action but also of thought. Heuristic and algorithmic technologies are needed for the automation of cognitive activities. There is a need to develop tools, which are capable of assessing a situation at hand, alerting the human part of the system to various concerns and providing advice on possible responses, and possibly proceed towards solving the problem.

One of the most complex problems facing all the big cities of the world alike is that of traffic congestion and unwanted incidents of collision. There is an ever-growing demand for new highway systems as a result of the exponentially increasing vehicle population. This growing problem has to be taken into account seriously, not only by governments, but also by private sector. A solution to the problem here lies not in creating more and more assets but in the smart management of existing assets.

Hence, design of smart vehicles and highway systems and efficient management of automated systems can definitely alleviate the existing problem of traffic congestion and help in collision avoidance.

The objective of this proposal is to develop a human-machine automation system and identify the level of automation that would optimize the performance levels of both, the human and the machine. It is also intended through this project to delve into the prospect of the successful use of data fusion algorithms specifically like GPS and Kalman filters to improve the safety and efficiency of transportation systems particularly with respect to collision avoidance.

LITERATURE REVIEW

A thorough study of all the concepts dealing with automation and smart human-machine technologies will be undertaken for this research. All the previous work will be analyzed and improvisations, if any will be suggested. The literature review carried out till date is summarized below.

Intelligent Vehicle/Highway System

The Intelligent Vehicle/Highway system is an intelligent transportation system, in which vehicles and highways will exchange information through a two-way communication system. The automated highways will have a set of lanes on which vehicles with specialized sensors and wireless communication systems could travel under computer control at closely spaced intervals.

This type of arrangement is called Platoons. The vehicles could continuously exchange information with other vehicles and traffic-control centers about speed, acceleration, braking, obstacles, road conditions, etc. Sensor data can be processed and sent back to each vehicle guaranteeing a continuous exchange of information.

The highway system will know the destinations and planned routes of individual vehicles. In that way the system can coordinate traffic flow more efficiently, reduce speed fluctuations, monitor unsafe vehicle operation and traffic shock waves, maximize highway capacity and minimize avoidable traffic congestion. In addition, the system will respond rapidly to changing highway conditions.

The vehicles might use several types of devices to sense its environment, such as magnetometers, visual sensors, infrared sensors, laser sensors, accelerometers, etc. Each vehicle has to have a powerful computer to process sensory data and the information that come from the traffic-control centers.

Devices and systems such as Global Positioning System (GPS), Automatic Vehicle Monitoring (AVM), Incident Management System (IMS), Intelligent Transportation System (ITS), Vehicle Navigation System (VNS), and Driver Assistance System (DAS) can all be lumped under Intelligent Vehicle/Highway System (IVHS).

Collision Avoidance

Collision Avoidance System (CAS) denotes the first step towards accomplishing fully automated highways. However, the development and near-term implementation of CAS is driven by the system's role in preventing rear-end vehicle collisions.

The development of CAS represents a change of focus in dealing with the consequences of accidents. Traditionally the emphasis has been on injury mitigation for those involved in a collision, for example by providing stronger vehicle frames, seat belts, and airbags. Starting with the development of antilock brakes, the focus has now shifted to collision avoidance. Most CASs are currently in experimental stage; before they can be implemented on a wide-scale basis; several technical and political questions need to be answered.

A collision avoidance system operates, generally, through a sensor installed at the front end of a vehicle, which constantly scans the road ahead for vehicles or obstacles. When found, the system determines whether the vehicle is in imminent danger of crashing, and if so, a collision avoidance maneuver is undertaken. Most CASs are non-cooperative, that is, detection is independent of whether other vehicles on the road are equipped with collision avoidance devices.

An alternative technology relies on vehicle-to-vehicle communications to exchange information on vehicles' presence, location, lane of travel, and speed among other factors. In addition to the front-end sensor, vehicles require a rear-end transponder as well, since communication, and therefore detection, only occurs among equipped vehicles.

Criteria to Activate the Collision Avoidance System

Time-to-collision criterion: the system decides whether a collision is likely to occur at prevailing speeds and distances, within a certain time interval. In a car-following scenario, the time-to-collision is the time taken for the two vehicles to collide if they maintain their present speed and heading.

Worst-case scenario: the system infers that the vehicle preceding the CAS equipped vehicle could brake at full braking power at any time. Basically, it operates on a "critical headway distance" that is, the minimum distance necessary for the CAS-equipped vehicle to come to a stop in the event of the leading car abruptly brakes.

Collision Avoidance Maneuvers

Headway distance control: the system alerts the drivers when their cars are following the leading car too closely. Some systems include automatic speed control, in such a way; the car could automatically reduce its speed in order to maintain a safe headway with the leading vehicle.

Hazard warning: the system alerts the driver of an object (moving or stationary) within its projected path, so that the driver has enough time to avoid collision.

Automatic vehicle control: the system controls the vehicle's brakes and steering wheel, and applies them automatically when it determines it is necessary.

Warning Devices

Visual head-up displays: warnings are displayed on the windshield in the driver's fields of view, so that their content can be assimilated in conjunction with the driving scene ahead. These displays are intended to minimize distraction from driving tasks, in addition to ensuring that warning does not go undetected.

Audio/Voice signals: in comparison to visual signals, auditory signals appear to be less intrusive on driving tasks. They are also insensitive to external conditions such as poor light, bad weather, or a dirty windshield. Two different auditory warnings have been developed: speech (synthesized voice) or non-speech (buzzer) displays.

Haptic devices: they provide redundant information via alternative sensory modalities, given that the primary visual or auditory channel may be degraded or overburdened. Research suggests that one possibility is to increase the force needed to push the gas pedal.

Kalman filters: First reported in ASME's Journal of Basic Engineering by R.E.Kalman (1960), this positional estimation algorithm has been widely used for a variety of optimization tasks. Transportation systems employing Kalman filtering use discrete-time algorithms to remove noise from sensor signals in order to better determine the present and future positions of a target.

Kalman filtering produces fused data that estimate the smoothed values of position, velocity, and acceleration at a series of points in a trajectory.

Although no set of sensors can pinpoint a target with complete accuracy, the tolerance of each sensor's positional fix accuracy can be known and assigned. So Kalman filtering can be used to define a region of space within which an object is located. The narrower these spatial limits are kept, the better the estimation algorithm can perform.

The Kalman filter estimates a process by using a form of feedback control. The filter estimates the process state at some time and then obtains feedback in the form of (noisy) measurements. As such, the equations for the Kalman filter fall into two groups: time update equations and measurement update equations. The time update equations are responsible for projecting forward (in time) the current state and error covariance estimates to obtain the "*a priori*" estimates for the next time step. The measurement update equations are responsible for the feedback—i.e. for incorporating a new measurement into the "*a priori*" estimate to obtain an improved "*a posteriori*" estimate.

The time update equations can also be thought of as predictor equations, while the measurement update equations can be thought of as corrector equations. Indeed the final estimation algorithm resembles that of a predictor-corrector algorithm for solving numerical problems.

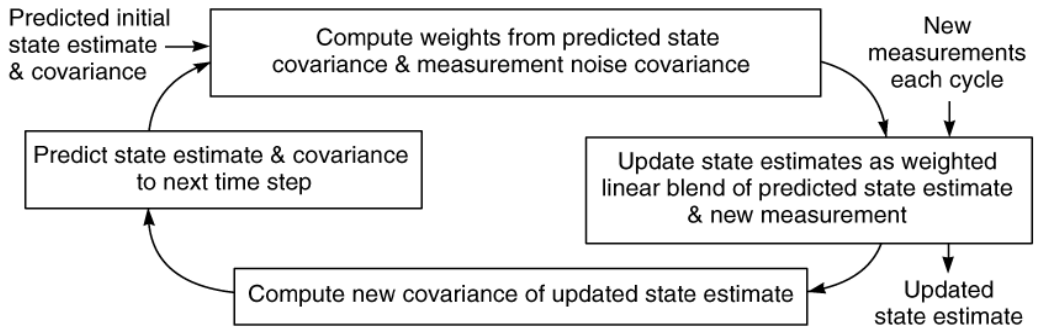

FIGURE 1. Kalman Filter Cycle

The Kalman filter is a recursive, linear filter. At each cycle, the state estimate is updated by combining new measurements with the predicted state estimate from previous measurements.

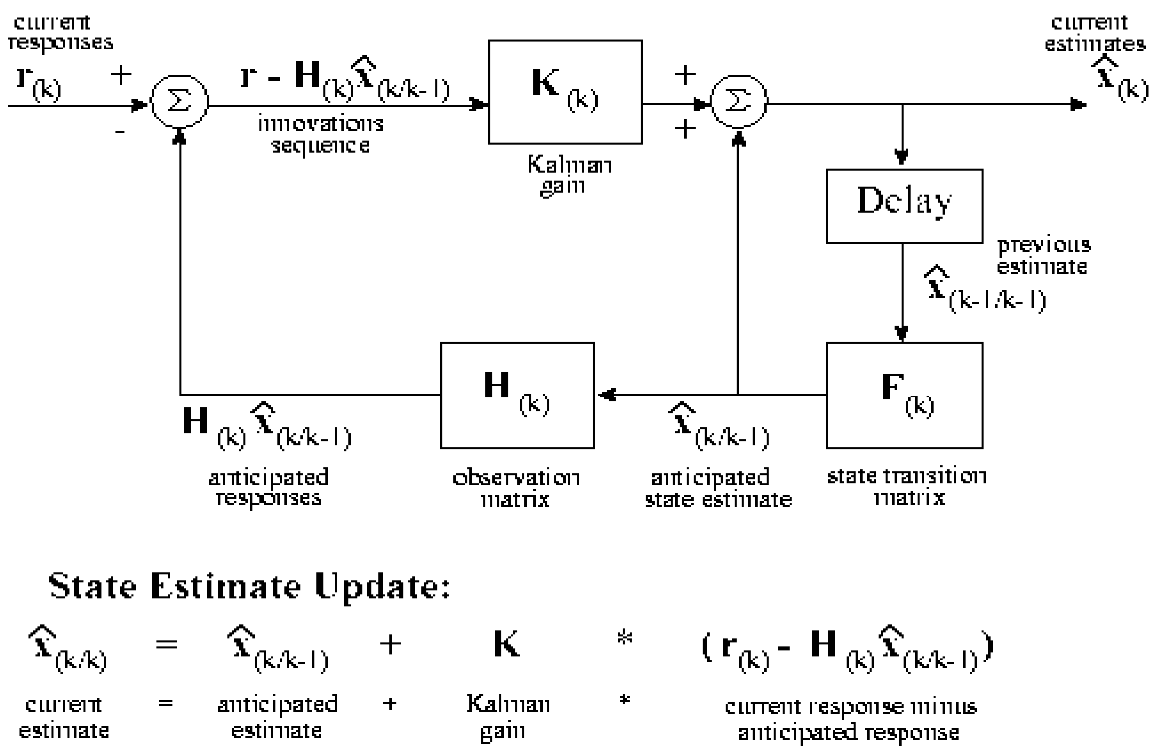

FIGURE 2. Kalman Filter Operation (*1*)

Global positioning system (GPS): A global positioning system employs a network of Earth-orbiting satellites to calculate a subject's position and then transmit that information to the subject's GPS receiver. The GPS has 3 parts: the space segment consists of 24 satellites, each in its own orbit 11,000 miles above the Earth. The user segment consists of the receivers. The control segment consists of ground stations (five of them located around the world) that make sure the satellites are working properly.

Each GPS satellite transmits data that indicates its location and the current time. All GPS satellites synchronize operations so that these repeating signals are transmitted at the same instant. The signals, moving at the speed of light, arrive at a GPS receiver at slightly different times because some satellites are further away than others. The distance to the GPS satellites can be determined by estimating the amount of time it takes for their signals to reach the receiver. When the receiver estimates the distance to at least four GPS satellites, it can calculate its position in three dimensions. Ground stations are used to precisely track each satellite's orbit. A GPS receiver knows the location of the satellites, because that information is included in satellite transmissions. By estimating how far away a satellite is, the receiver also knows it is located somewhere on the surface of an imaginary sphere centered at the satellite. It then determines the sizes of several spheres, one for each satellite. The receiver is located where these spheres intersect.

GPS accuracy: The accuracy of a position determined with GPS depends on the type of receiver. Most hand-held GPS units have about 10-20-meter accuracy. Other types of receivers use a method called Differential GPS (DGPS) to obtain much higher accuracy. DGPS requires an additional receiver fixed at a known location nearby. Observations made by the stationary receiver are used to correct positions recorded by the roving units, producing an accuracy of greater than 1 meter.

Data fusion process: The data generated by a GPS could be passed through a Kalman filter which would forecast future traffic condition. A Pattern matching software could be used to match the current traffic situation with the historical situations. A Knowledge–Based Expert System (KBES) could then be used to identify abnormal traffic conditions, and if possible qualitatively suggest corrections early enough to avoid any incidents.

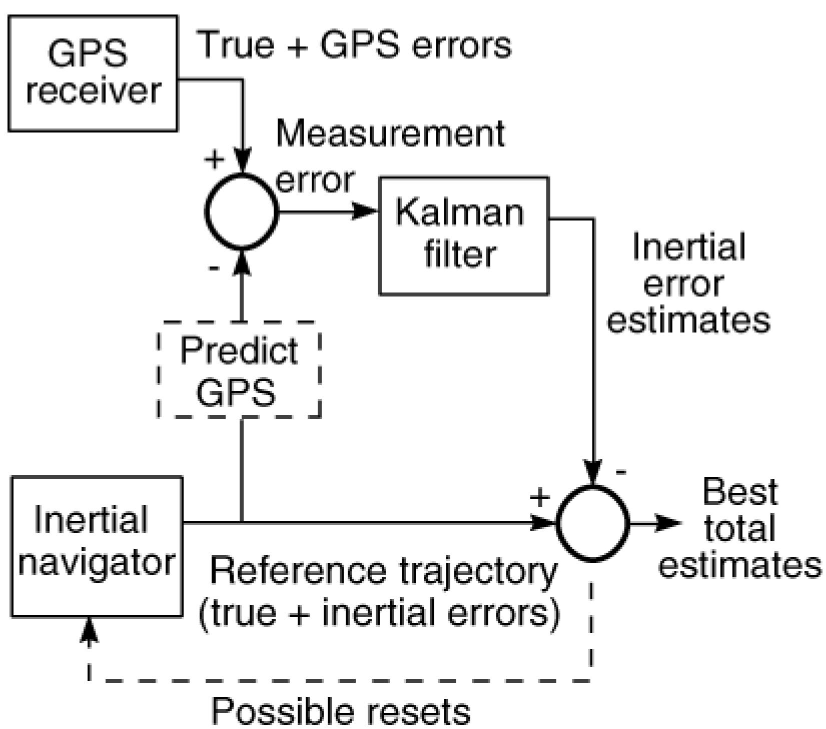

FIGURE 3. GPS Flowchart

An integrated GPS receiver and inertial navigator use a Kalman filter to improve overall navigation performance

RESEARCH APPROACH

Developmental Stages

Stage I: to equip vehicles and highways with GPS receivers and transmitters to allow exchange of data between them as well as with the Traffic Information Centers (TIC).

Stage II: To design a Kalman filter specifically to process the data generated by the Global Positioning System and accurately predict the future traffic condition. The most challenging aspect of this step would be to define the initial state i.e., to determine the state equations that effectively represent the traffic model, to be able to extract the data that will be needed to predict the future states.

Stage III: Developing software capable of identifying similar situations in past appearing in the present data. The software developed could be trained in such a way that, it in collaboration with a database has the capability of identifying events and categorizing them.

Stage IV: Development of specific expert system to recognize anomalous traffic conditions. This system should not only identify the abnormal traffic conditions but it should also give a qualitative account of the condition.

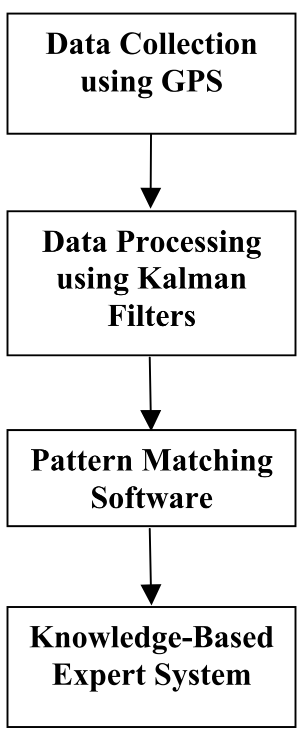

FIGURE 4. Diagrammatic Representation of the Process

SUMMARY

The challenge is to ensure that new information, safety, and automation technologies are integrated to create human-centered intelligent vehicles that can advance safety, surface transportation efficiency, and economic competitiveness. It is my opinion that by developing human-centered smart systems the demands on present-day transportation syatems can be met effectively. It is through human-centered automation, the most efficient method of data-analysis and decision-making can be achieved.

Technological change, however, is constant. Even as one set of technologies are envisioned, the envelope of possibilities expands. The latest innovations in information and computer technology, for example, are focusing on the development of neural links that will allow commands to be given with muscle impulses, eye movements, and brain waves, creating an almost symbiotic relationship between humans and machines. The vision of a human-centered intelligent vehicle, therefore, is not fixed but will continuously evolve in the wake of continuing technological breakthroughs. However, for transporation systems, putting people at the center of technology, will remain paramount.

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GASB Statement 34—the On-Ramp to Transportation Asset Management or a Detour Leading to Business as Usual?

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ABSTRACT

When the Government Accounting Standards Board (GASB) adopted Statement 34, it changed the generally accepted accounting standards for state and local governments. GASB 34 required that agencies place the value of the assets they manage in their annual financial reports.

GASB 34 allowed agencies to track the current value of an asset by using depreciation or a modified approach that focuses on the preservation (rather than depreciation) of assets. The modified approach requires that the condition in which assets are managed meets or exceeds self-imposed minimum conditions, thus requiring the use of an asset management system. The infrastructure management community assumed that this would encourage the adoption of asset management systems.

The authors interviewed several large cities and found that they did not adopt the modified approach and instead used the depreciation approach. The message inferred through our findings is that asset management systems should be adopted because they represent good practice and not because of a change in accounting standards.

Key words: asset management—depreciation—Government Accounting Standards Board Statement 34—infrastructure management—preservation

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INTRODUCTION

The Government Accounting Standards Board (GASB) produced Statement 34 in 1999. GASB produces the standards used to conduct financial accounting for governmental agencies. GASB statement 34 (known as GASB 34) is thought to be the most significant advent in government accounting practice since generally accepted accounting principals (GAAP) for governmental agency were established in 1934 (when the accounting standards were first created) (*1*). GASB 34 modified the GAAP such that public agencies would be required to show their infrastructure assets on their comprehensive annual financial reports (CAFR). This meant that public agencies' annual financial reports would contain many billions of dollars of infrastructure assets that had not previously appeared in their financial reports.

By accounting for the historical value of infrastructure assets and placing their value in comprehensive financial reports, advocates of better stewardship of infrastructure assets assume that once the large value of these assets is known, agencies would recognize the enormity of their responsibility. The significance of the responsibility placed in the hands of current infrastructure stewards would lead to budgeting and management to preserve or enhance the value of infrastructure assets left to the next generation (*1*).

Many public asset managers thought that GASB 34 would facilitate the wide spread adoption of infrastructure management systems for individual asset categories (e.g., pavements management systems, bridge management systems, etc.). This was so much the case that papers and reports written on infrastructure management systems would commonly include GASB 34 as part of the lexicon of issues pressuring asset managers toward embracing modern infrastructure asset management (*2*).

Although public asset mangers had high expectations for changes in management practices resulting from GASB 34, it should be remembered that GASB 34 is an accounting standard. GASB 34 may create conditions that make it attractive to implement management systems and the implementation of management systems may be a side benefit of GASB 34. However, the purpose of accounting standards is to provide consistency between financial reports, making it easier for creditors and the public to judge the financial performance, credit worthiness, and solvency of an agency.

To comply with GASB 34, a public organization must estimate the current value of assets based on historical costs. If an agency chooses to follow the conventional approach to estimating the current value of an asset, the value is the original cost of the asset minus the cumulative depreciation while the asset has been in use. This conventional approach depreciates the value of assets through time.

GASB 34 also allows for an alternative to the depreciation methodology, known as the "modified method." The modified method assumes that the agency will preserve the asset through time, thus allowing the agency to consistently perform its tasks at the same level of performance (condition). Because the level of condition remains consistent (or relatively stable), the value of the asset stays constant through time. If an agency decides to use the modified method to comply with GASB 34, the agency must adopt a management system to ensure that the asset's condition is preserved. The conventional approach views infrastructure as an asset which depreciates and the modified approach views infrastructure as an asset to preserve.

Many in the public works community saw the modified approach as the preferred method to complying with GASB 34 because it helped to recognize the stewardship responsibility of public agencies. For example, the American Public Works Association (APWA) (*3*) adopted a policy to encourage its members to adopt the modified approach. The American Association of State Highway and Transportation Officials (AASHTO) did not adopt this policy but, unofficially, encouraged its members to follow the modified approach. In addition, it appears that over half of the State Transportation Agencies produced financial reports based on the modified approach (*2*).

GASB 34 phases in the new requirements over time and the largest agencies are required to comply first. Agencies with annual revenues of more than $100 million were to start preparing annual financial reports following GASB 34 standards in the fiscal year beginning July 2001 and ending June 2002. Agencies with annual revenue between $10 million and $100 million would start in July 2002 and agencies with less than $10 million in annual revenue were to start in July 2003. Therefore, agencies with the largest cash flow (over $100 million) produced their first GASB 34 compliant annual financial reports in the summer of 2002.

To determine the approach that local governments were taking to respond to GASB 34, we interviewed financial managers of several large midwestern cities in Iowa, Minnesota, Nebraska, and South Dakota (cities with more than $100 million in revenue) during the winter of 2002-2003. These cities had recently completed their first GASB 34 compliant financial report. Although our interviews were rather open-ended and covered several issues, our main intent was to determine which method was used to include infrastructure in their financial reports (the depreciation method or the modified method).

None of the cities we interviewed chose the modified approach. The overwhelming response to our open-ended questions was that they (city officials) knew in advance that the value of the infrastructure was going to be enormous and now they have a more exact estimate of the value. However, most believed that having this knowledge would change very little. A couple of cities believed that the attention paid to infrastructure as a result of GASB 34 would or has helped them to implement systems to manage assets but they doubted that these systems would ever be used to support their financial reporting.

WHAT IS ASSET MANAGEMENT?

For at least the last 40 years there have been systems developed to manage individual categories for public sector assets. For example, pavement management has been practiced since the 1960s and 1970s. Similar systems have been developed for bridges, underground utilities, buildings, etc. Partially, the abundance of systems to manage assets is a result of the decline in the cost and availability of computing resources. All systems at a minimum involve the following:

- a physical inventory of the assets and their locations
- a measurement of the assets' condition
- rules that allocate resources (in the form of a treatment) to the asset when its condition declines to minimum level
- a budget developmental tool which allocates resources to individual assets
- a model that determines the improvement to an asset when the asset receives a treatment
- a report that identifies the actions applied to assets

Beyond these minimum requirements, systems can vary dramatically. There are systems that focus on planning level decisions or project specific level decisions, can provide single year or multi-year resource allocations, and can allocate resources based on decision-making rules or optimization. GASB 34's modified approach requires that any classification of infrastructure must have an accompanying management system to ensure that the condition of the asset is preserved. However, GASB 34 places very few requirements on the mechanics of the management system.

Although what GASB 34 encourages are systems to manage individual categories of infrastructure, it helps to promote the science of asset management. Asset management looks across all asset categories and takes a holistic view of assets rather than managing one asset category at a time and rather then managing each asset category to incomparable performance standards such as pavement roughness and bridge health.

The private sector more easily takes a holistic view of its assets. The assets necessary for the production and marketing of a good or service are organized so that return on investment is maximized. In other words, maintenance, preservation, and renewal of all assets are either organized so that rate of return on investment is maximized or cost is minimized. The private sector, however, has the advantage of making all decisions based on the objective of maximizing overall return on investment and hence providing a common measure of return (revenue) for all of its assets (investments). In the public sector, the objective is similarly to maximize return on investment but return in the public case are the benefits to the users, which are less easily measured and quantified.

Because return on investment is more easily measured in the private sector, private sector asset management is more easily defined and practiced. Because the measures of return are not as easily defined, the public sector definitions of asset management are generally not as direct or as specific.

In the 1980s as pressure was placed on governments in the UK, Australia, New Zealand, and Canada to privatize public services, public agencies began to adopt a perspective similar to the private sector regarding asset management. In the mid 1990s, the concept of asset management began to enter into the public sector vocabulary in the U.S.

At the federal level, in the 1990s Congress and the executive branch took a number of initiatives to improve capital asset decision-making. These included "enacting the Government Performance and Results Act of 1993, the Federal Acquisition Streamlining Act of 1994, the Clinger-Cohen Act of 1996 and a series of federal financial accounting standards developing the capital programming guide (1997), and appointing a President's commission to study capital budgeting (1997)" (*4*). At the state level, AASHTO and the Federal Highway Administration (FHWA) have held a series of workshops starting in 1996 which has led to the creation of an AASHTO Task Force on Transportation Asset Management and an National Cooperative Highway Research Program guide on asset management (*5*). At the local level, the APWA formed an asset management task force that published a final report in 1998, recommending a number of steps for further development of asset management. Then in 2002 AWPA published its own manual on asset management (*6*). In the late 1990s FHWA established an Office of Asset Management and a number of universities have developed university programs focusing on infrastructure asset management. Lastly in 2000, the Transportation Research Board established a Task Force on Transportation Asset Management.

With all the interest at the federal, state, and local levels in asset management, a great deal of enthusiasm was generated in the anticipation of agencies having to comply with GASB 34. The infrastructure management community expected that those responsible for implementing GASB 34 standards into financial reports would see the attraction of implementing the modified method and GASB 34 would be the vehicle for widespread adoption of infrastructure management systems. This would then lead to agencies embracing asset management principals, techniques, and systems.

The assumptions that GASB 34 and asset management are tied together and have, if not the same, at least parallel objectives, can be substantiated many times throughout literature where they are discussed. For example, in a recent paper where the authors describe the development and need for a regional geographic information system database of transportation assets, one of the database uses is to support asset management application/GASB 34 (*7*).

At least among the large cities that we interviewed, we found that in practice, the accountants preparing annual financial reports compliant with GASB 34 (an accounting standard) and infrastructure managers each have their own objectives. From the perspective of the government accountants we interviewed, it is their feeling that asset managers should adopt infrastructure management systems and asset management principals because they support and reinforce good stewardship of infrastructure assets. The accountants we talked to generally saw the purpose of accounting standard to create comparability of financial reporting to help creditors and the public judge the financial performance, solvency, and creditworthiness of agency.

INTERVIEWS

To provide consistency between interviews, our first step was to develop a standard list of questions. Since initially we believed that many of the largest and most sophisticated cities in the region would naturally choose to adopt the modified method, our questions most closely focused on issues that were most relevant to the application of the modified approach. The objective of the questionnaire was to lead the discussion and not restrict the interview to only the questions, thus allowing the respondent to expound on the issues. The interview was divided into three categories with questions under each. The categories and questions included the following:

1. Organizational issues, approach, and responsibility for preparation of infrastructure inventory and valuation—
    a. What individual or office within your organization was responsible for implementing the GASB 34 compliant methods for reporting transportation infrastructure?
    b. What systems did your agency use before GASB 34 to inventory and track assets?
    c. Did your organization use the depreciation method or the modified approach?
2. Methodological issues—
    a. What methods were used to determine the historical value of assets?
    b. If depreciation was used what method was used to calculate it?
    c. If the modified approach was used, how was the asset management system established?
3. Impact of GASB 34—
    a. As a result of GASB 34, has your agency adopted any new asset management systems?
    b. Has your agency's policy making board or citizens shown any additional interest in the agency's financial reports as a result of changed practices?
    c. Does your agency intend to educate the public or public policy makers in the interpretation of the asset valuation in the annual financial report?
    d. Do you expect that the information contained in the annual financial report will be used to justify utility rates, capital program, or decisions regarding bonding for infrastructure improvements?
    e. Do you believe that any benefits have been achieved as a result of complying with GASB 34?

The interviews were first conducted by contacting the city by telephone, searching for a person in the city who felt qualified to answer the questions, emailing them the list of questions, and then interviewing the individual over the telephone. A total of nine cities were interviewed. Given the $100 million per year revenue threshold, we interviewed the majority of the cities above the threshold in four midwestern states (Iowa, Minnesota, Nebraska, and South Dakota).

Interview Responses

Organizational Issues, Approach, and Responsibility

In all cases, the infrastructure portion of the financial report was led by the financial office, which also determined the strategy for valuing the infrastructure assets. The public works department often supported the financial office with field data. The public works department was always called on to help determine the lives of various assets. Informally, some of the respondents noted that public works officials were sometimes concerned that the city should follow the modified approach but in all cases the financial manager's perspective prevailed and infrastructure was added to the financial report using the depreciation approach.

Although some of the cities interviewed already have asset management systems (usually pavement and/or bridge) the location, construction costs (or approximate construction costs), and extent of the infrastructure were largely derived from financial records, capital improvement plans, and construction records. In some cases the city planned to make use of existing asset databases to verify the information included in financial records. Since the depreciation method was used in all cases, no asset condition data were needed.

Methodological Issues

All but two cities used financial records to determine the historical costs of assets when they were constructed. Two cities estimated historical costs. They both determined the replacement cost of typical infrastructure designs (e.g., a standard roadway cross section) and then applied a deflation factor to estimate the cost of assets when they were constructed.

All of the cities used straight-line depreciation. The expected lives of infrastructure assets were provided by the public works officials of each city.

Impact of GASB 34

When asked about the impact of GASB 34, none of the organizations believed that moving infrastructure assets on to their financial report would have any impact immediately. However, a few cities felt that this will help policy makers to understand the scale of the assets for which they are responsible and may help in budgeting in the future. For example, one respondent stated, "It was a great exercise in getting every one (city management staff and the mayor's office) thinking about the city's financial statement. It got them to think about financial reports and what the implications are of raising and programming capital. It was a nice exercise and the practice will evolve in the future." However, just over half the cities interviewed openly stated that GASB 34 will change very little and placing infrastructure assets in their financial reports implied a lot of work for very little benefit.

On the positive side, three of the cities saw the interest in GASB 34 resulting in new infrastructure management systems. The largest city in our sample stated that they had been planning to develop a comprehensive infrastructure management system and the interest in infrastructure management caused by GASB 34 has caused them to act on their plans. They are currently moving forward with a comprehensive system. However, even after the asset management system has been developed, the respondent was doubtful that the city would adopt the modified approach to reporting infrastructure assets.

CONCLUSIONS

The authors were surprised that none of the cities in our sample adopted the modified approach. In retrospect, however, we can see that the belief that cities would embrace management systems for their infrastructure due to GASB 34 may have been naïve and an exercise in wishful thinking. GASB 34 is an accounting standard and is intended to make the financial position and performance of governmental agencies more easily interpreted from their annual financial report.

Asset managers who were looking for an outside force to encourage agencies to invest in modern systems to manage assets, at least in our sample cities, did not find their champion in GASB 34. It is only speculation, but we believe that the modified approach was more readily adopted by State Transportation Agencies because their management is more engineering-oriented than municipalities and, therefore, more likely to promote the use of systems to manage infrastructure.

Having seen what happened when our sample of cities complied with GASB 34, we can observe that it did not meet our expectations and probably the expectations of much of the infrastructure asset management community. However, GASB 34 did cause all the cities we interviewed to think about how they manage their infrastructure assets and in a couple cases may have even resulted in the adoption of new systems to manage them. From an asset management perspective, we can see that GASB 34 did have some success in promoting the use of systems to manage infrastructure assets, although not as much as we expected.

As public agencies move toward the use of asset management and take a more holistic approach toward the management of infrastructure, GASB 34 may not have been the on-ramp to asset management we would have liked but it certainly was not a detour. If it got top agency management to start thinking about how cities manage, fund, renew, and replace infrastructure, then the cause of better infrastructure management has certainly been moved forward.

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Benefit-Cost Assessment of Automatic Vehicle Location (AVL) in Highway Maintenance

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ABSTRACT

AVL has been used extensively in public transit, law enforcement, and EMS applications (among others), and is garnering more and more interest with the highway maintenance community. Sponsored by the Kansas DOT, The University of Kansas conducted a study of the use of AVL for highway maintenance activities, especially snow removal. State DOTs and other transportation agencies were surveyed with respect to their use or potential use of AVL. At the time of the survey, only 8 states had deployed AVL, in addition to several municipalities and one Canadian province. None of the surveyed agencies had conducted quantitative assessments of the benefits of system deployment.

Qualitative and perceived benefits taken from the aggregated survey data were used to develop estimates of benefits likely to be realized from an AVL deployment. Savings from improved fleet management, paperwork reductions, and reductions in snow-related crashes were compared with the system investment and maintenance costs. Costs for system wide deployment were estimated to be about $9,000,000 with about $800,000 needed for maintenance annually. Benefit/cost ratios were calculated for three deployment schedules based on conservative assumptions and then based on moderate assumptions. The analysis estimated B/C ratios would be at least 2.6 and probably closer to 25.

This paper elaborates on the results of the survey and details the methodology used in the analysis.

Key words: AVL — benefits—ITS—maintenance—snow removal

INTRODUCTION

Automatic Vehicle Location (AVL) systems are a fleet management tool that integrates several technologies to allow a fleet manager or dispatcher to see the location of their vehicles at any given time. Many systems can also indicate the status of each vehicle. For example, a law enforcement dispatcher can be informed whether a vehicle is in service, on break, or in hot pursuit. A snowplow can automatically report whether the plow is up or down and when it is spreading sand or salt. Putting this kind of information at the hands of managers enables them to make more efficient use of their resources. Both public and private agencies are taking advantage of AVL to enhance both the efficiency and the effectiveness of their operations. AVL has been widely used in the trucking industry for several years. Many transit and emergency services agencies have also implemented AVL. Use among highway departments has been sparse, possibly because the benefits in that context are less obvious. Even so, several state Departments of Transportation and municipal Public Works departments have implemented AVL and found it to be a valuable tool for maintenance and operations activities.

HISTORICAL PERSPECTIVE

The AVL services industry was born when Qualcomm launched its OmniTRACS service in 1988. Providing vehicle tracking and data messaging, the service primarily targeted the long-haul trucking industry. Although OmniTRACS continues to hold a commanding share of the AVL market, especially in the trucking industry, a number of rivals emerged, including Highway Master, InTouch Communication, American Mobile Satellite Corporation (AMSC) and Orbcomm. (*1*)

Most of these companies rely on the same technology, Global Positioning System (GPS), for their geolocation function. In contrast, a variety of wireless technologies are used for communications. OmniTRACS, for example, leases capacity on geo-stationary satellites. HighwayMaster and InTouch use analog cellular telecommunications. AMSC relies on a hybrid system that operates over both its geo-stationary satellite and the ARDIS terrestrial data network. In November 1998, Orbcomm began offering commercial vehicle tracking services that use its constellation of 28 Low-Earth Orbit (LEO) satellites for communications. Teletrac uses a dedicated radiolocation network. (*1*)

HIGHWAY MAINTENANCE APPLICATION OF AVL/MDT

In 1998, the Virginia Department of Transportation (VDOT) implemented an AVL system that allowed snowplow managers to better manage snow removal in four counties. (*2*) The Smart Plows utilize ITS technologies to monitor vehicle location, road surface condition, and apply the appropriate amount of chemicals or sand to treat the roadway. The goal of the system was to facilitate the following functions.

- Continuous location of snowplow fleet operations
- Ability to identify vehicles with abnormal behavior
- Increase safety for the vehicle operator
- Ability to detect and minimize waste and fraud

- Ability to capture statistical data
- Improved communications efficiency.

These benefits are typical of those expected by other highway departments who have deployed AVL. In general, the consensus seems to be that AVL can deliver all of these benefits, although quantitative data is not yet available.

AVL/MDT COST-BENEFIT ANALYSES

Without formal evaluation studies, it can be difficult to judge whether or not the benefits outweigh the costs of implementation and operation. AVL and MDT technologies have been deployed around the world by various types of agencies, but predominantly by transit agencies and commercial trucking companies. All too frequently, once an agency obtains approval to implement AVL, justifying the costs becomes a low priority. Evaluation plans are often sacrificed at the first signs of a budget shortfall. While that is understandable, it is often the result of a near-sighted perspective and is very unfortunate for the transportation community at large, particularly in the longer term as agencies have difficulty justifying investments in AVL because of the lack of documentation of benefits.

Many transit operators experience strong pressure to upgrade their fleet management systems with the latest AVL technologies. The many benefits of such systems have been very well publicized in numerous U. S. Department of Transportation (USDOT) publications. With a 45% annual return on investment reported as the cost effectiveness, these systems quickly end up paying for themselves. In that light, it is not surprising that there is a strong push for system procurement (*3*). But it is noteworthy that this return of investment applies for transit fleet management systems, but not necessarily for highway maintenance and operation purposes.

There are, however, some general principles that are worth considering in developing a strategy for assessing the potential benefits of AVL in a highway maintenance context. Schweiger C. L., and Marks J. B. (*4*) have outlined four critical parameters that should be considered. Their particular concern was the use of AVL in public transit, but the principles have broad application across contexts. They recommended the following four components be included in cost-benefit analysis.

- Determination of Life-Cycle Costs
- Methodological Approach to Cost/Benefit Analysis
- Quantification of Risks
- Assignment of Dollar Values to Intangible Benefits

If these components can be effectively applied to AVL in a highway maintenance context, valuable information can be gleaned for transportation agencies considering AVL. Toward that end, in Spring 2000, the Kansas Department of Transportation contracted with The University of Kansas to conduct a study of the costs and benefits associated with implementing AVL in their maintenance and operations.

APPROACH

An initial survey was conducted to determine the level of involvement of highway agencies in AVL deployment. Those agencies identified by the survey as having deployed an AVL system were contacted with a more extensive survey related to their experiences. The results of the survey were used to establish parameters for the benefit-cost analysis.

Two risk perspectives (low risk and very low risk) were examined to provide insight into the likely magnitude of the benefits. Costs were constant across scenarios. Benefits in the very low risk perspective were calculated using all conservative assumptions. The results of this analysis represent the minimum benefits that can reasonably be expected. In other words, the actual benefits are highly likely to be equal to or greater than the calculated benefits, and likely to be much greater. Benefits in the low risk perspective were calculated using more moderate (though still somewhat conservative) assumptions. The resulting figures more closely approximate what could be expected. For these scenarios, the actual benefits are likely to be close to but slightly greater than the calculated benefits. For both perspectives, the results were used to calculate a cost-benefit ratio based on three different implementation scenarios, conservative, moderate, and aggressive, with the full implementation occurring in 20, 10, and 6 years, respectively.

SURVEYS

The literature discusses a number of AVL/MDT deployments among highway maintenance agencies, but no formal cost-benefit assessment is available. Costs vary widely from implementation to implementation, and benefits are largely anecdotal and undocumented. To exacerbate the problem, most deployments in the highway maintenance arena are so recent that even the anecdotal reports of benefits are speculative. It is very likely that some agencies have implemented AVL without having published information about the systems, though it is unlikely that such an implementation would include a formal evaluation.

Based on results of the literature review, a survey was developed to help identify agencies that have had experience with AVL. Initially, all 50 state DOTs, Canadian provinces, and 6 municipal public works departments were contacted regarding their deployment of AVL/MDT technologies for tracking maintenance vehicles. Several of the municipal applications were identified by contacting technology integrators (i.e., consultants) that are prominent in the field. The survey was also made available over the Internet and various organizations were asked to bring it to the attention of their constituents. Organizations contacted included ITS America, ITS Australia, ERTICO, and VERTIS. Regretfully, no responses were received as a result of involving the ITS organizations.

The preliminary survey revealed that fifteen agencies had already deployed AVL/MDT technologies for tracking their highway maintenance vehicles, eight of which were State DOTs. A detailed questionnaire was developed to further explore the experiences of these agencies. This questionnaire included questions regarding the technology being used, costs and benefits experienced, and obstacles encountered. All surveys—both preliminary and detailed—were followed up via Email and telephone until all responses were received.

Results Summary

Figure 1 shows a time line of the AVL deployments that were identified in the preliminary survey. Among the deployments, the number of vehicles outfitted with *in-vehicle units* (IVUs) ranged from 4 to 150, with an average of 36. Half of the deployments involved 20 vehicles or more. 12 of the 15 agencies cited snow removal as their primary application. The most frequently cited benefit was improved snow removal. The agencies were asked to cite the most significant obstacle encountered during the deployment process. The most common issue seemed to be funding, although several other obstacles were mentioned. The majority of the deployments (10) utilized CDPD for transmitting data. Two used satellite, two used analog cellular (one of these two also used satellite), and two used LMR. When asked whether they thought the system was cost-beneficial overall, two agencies said their systems were cost-beneficial, and the rest indicated in some way that they could not yet tell. These responses were not surprising given that, at the time of the survey, none of the systems had been in operation for more than four years, and most were much more recent.

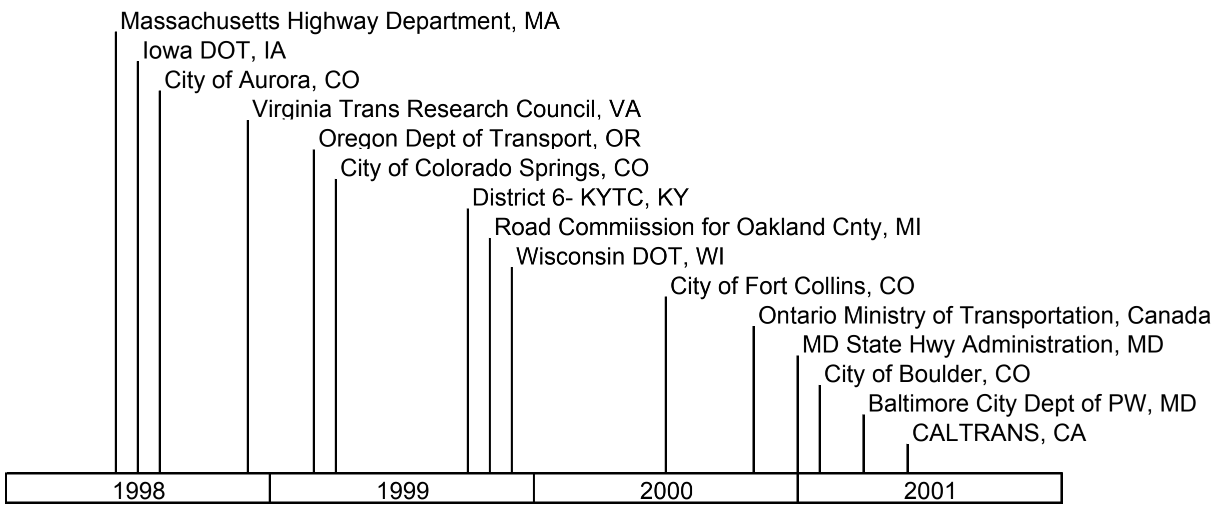

FIGURE 1. Time Line of AVL Deployments

COST ASSESMENT

Two categories of system costs were considered, investment costs and operation and maintenance costs. Costs were estimated for a small scale pilot project and for statewide deployment.

Investment Costs

It was assumed that KDOT's existing 800 MHz radio system would be used, and a dedicated channel would be added for data transmissions. The implementation cost for the dedicated data channel was approximately $750,000 for a pilot project and $6,000,000 for a statewide deployment. These estimates were provided by the KDOT Bureau of Maintenance and Construction based on current equipment costs. Consistent with other deployments, the communications costs are by far the most significant portion of the total system cost. Cost estimates for other components of the pilot test and subsequent statewide implementation were drawn from the data collected through the survey of transportation departments. Various considerations were used to determine each number, including the consistency of the costs for that component across implementations, how recent the costs were (i.e., when the implementation began), and the similarities and differences between the characteristics of the implementations described in the survey and the expected characteristics of an application to KDOT needs. The scope of the pilot project and the statewide deployment in terms of administrative areas and vehicle counts is shown in Table 1.

TABLE 1. Pilot Project and Statewide Deployment Scope

	Pilot	Statewide
Areas	1	26
Subareas	6	112
Maintenance Vehicles	23	585
Paint Trucks	1	6

In the vehicle, three different types of expenditures were considered. An In-Vehicle Unit (IVU), comprising a GPS receiver, a data modem, and a Mobile Data Terminal (MDT), was estimated to cost approximately $3,500, including installation. A total of 24 units were considered for the pilot project—23 maintenance vehicles and one paint truck. Road and air temperature sensors were estimated to cost $600 per vehicle. However, it was assumed in this analysis that the existing sensors would be used. Considering the communications, in-vehicle, other costs, the total cost for a pilot deployment was estimated to be $957,000. The costs associated with a statewide deployment are shown in Table 2.

TABLE 2. Statewide Implementation Costs

Items	Unit Rate ($)	No. of units	Amount ($)
Base station hardware	7,000	26 (1/area)	184,000
Software (licensing)	25,000 for the first computer 5,000 per additional	26 (1/area)	150,000
Sensors and software integration	15,000 (software)		15,000
In-Vehicle Units	3,500/unit	585 units	2,047,500
Training (3 days on site)	3,000/area	26 areas	78,000
Repair and Maintenance	4,000/year/area	26 areas	104,000
System integration	15,000/area	26 areas	390,000
Add data channel to radio system[1]			6,000,000
Total initial expenditure			8,968,500

[1] Includes $750,000 expended during pilot project.

Operation and Maintenance Costs

The operating costs generally involve the monthly fees for the CDPD connection, if a CDPD based communication system is used. For an implementation of AVL using KDOT's radio system, operation and maintenance costs are comprised primarily of maintenance and repair for the radio system's dedicated data channel, the in-vehicle units, and the base station equipment.

Annual maintenance costs were estimated to be the purchase price of the equipment divided by the typical service life. Only equipment unique to the AVL system was considered. That is, the cost of maintaining the 800 MHz radio system is a cost that would be incurred regardless of whether or not an AVL system were implemented. Consequently, the implementation of AVL adds no incremental cost to the maintenance of the existing radio system. The cost of the in-vehicle units is estimated to be $3,500 each. Assuming the statewide implementation would involve 585 KDOT vehicles, and assuming a 7-year service life based on Schweiger, et al. (1999), the average annual maintenance cost of the in-vehicle units would be $292,500. Assuming one base station at each area office with an initial cost of $7,000, also with a service life of 7 years, the annual maintenance cost of the base stations would be $26,000.

The incremental maintenance costs incurred by the addition of a dedicated data channel were estimated based on the KDOT Replacement Life Cycle of 12 years, assuming that an average of 1/12 of the equipment will be replaced each year. Under this assumption, each year's maintenance would be equal to the cost of the entire system times the percentage of the system deployed divided by 12. The total annual maintenance cost of the system, once fully deployed, would be $818,500.

EXPECTED BENEFITS

The nature of the expected benefits can be drawn from the experience of other agencies combined with the operational characteristics of KDOT maintenance crews. Benefits that can be expected to include the following.

- More timely response to emergencies.
- Improved resource management by analyzing past activities to improve efficiency.
- Reduced snow-related accidents due to reductions in snow removal times.
- Increased security for drivers.
- Reduced legal costs from tort claims allegedly involving KDOT maintenance vehicles.
- Reduced material costs with more efficient application strategies.
- Reduced time associated with routine paperwork.
- More timely pavement condition information.
- Enhanced locational accuracy of various inventories and map segments.
- Increased completeness of various inventories (e.g., PMS).
- Automatic and continuous updates of pavement conditions for KDOT maintenance.
- Potential feed of near real-time information to KDOT Advanced Traveler Information Systems (ATIS) (e.g., web site, dial in, 511 (eventually)).
- Improved efficiency and effectiveness of roadside maintenance.
- Reduced fleet maintenance costs due to improved fleet management

To develop an objective comparison of the costs and benefits associated with the implementation of AVL, a subset of the benefits listed above were quantified and compared with the maintenance costs over a 20-year period. The benefits considered in the cost-benefit analysis are shown in bold type in the list above. The other benefits listed were omitted from the analysis either because they are difficult to express in monetary terms or because the information gleaned from the surveys was insufficient to estimate their magnitude.

Benefits Survey

In the detailed survey, 7 of the 15 agencies contacted provided estimates of their efficiency savings in snow removal due to the implementation of AVL. The values cited ranged from 5% to 50% improvement, with an average of just over 20%. Only 3 agencies provided estimates of savings due to reduced paperwork. The values were 10% and 20%, with one agency citing an increase of 15% in paperwork to facilitate analysis. The minimum and average values were used for the cost/benefit analyses discussed in the following sections (the increase in paperwork to facilitate analysis was ignored).

Two sets of estimates of benefit magnitudes are presented in each of the sections that follow. Conservative estimates are detailed, representing the least magnitudes cited in the follow-up survey. The probable benefits are represented by moderate estimates, using the average of the values cited in the follow-up survey.

It should be noted that because only one agency provided a value for savings in materials, this factor was not included in the costs/benefit analysis. The efficiency improvement cited was 25-50 percent, which would represent a substantial monetary savings in addition to those included in the analysis.

Savings in Paperwork

The savings resulting from reduced paperwork were calculated on the basis of the daily and weekly reports filed by maintenance crews and paint crews. Based on estimated averages provided by KDOT, the calculations assumed that the reports require 25 minutes daily for each maintenance vehicle and 20 minutes weekly for each sub-area supervisor. An hourly wage of $10 was used, and the AVL system was assumed to reduce the time required to fill out the paperwork by 10%, which was the smallest of the values reported in the benefits survey. The conservative estimate for savings in paperwork was $67,908 per year. A moderate estimate of 15% time savings—the average of the values reported in the benefits survey—would result in an annual savings of $101,862.

Savings from More Efficient Fleet Management

By recording and analyzing fleet activities, vehicles can be used more efficiently, reducing the overall mileage incurred and the associated costs. For maintenance vehicles, annual mileage was estimated by dividing the service life in miles by the service life in years. 200,000 miles / 11 years = 18,182 miles/yr. An operational cost of $0.47/mi was assumed. The conservative and moderate estimates of savings were taken as the minimum and average values, respectively, that were reported in the benefits survey. The conservative estimate of 5% savings would result in an annual savings of $398,864, while the moderate estimate of 20% savings would result in an annual savings of $1,595,455.

For paint trucks, a similar calculation was performed using the same percent savings estimates (none of the agencies surveyed were using AVL in paint trucks). Using the vehicle service life, an annual average was calculated for the hours of use. 10,000 hours / 15 years = 667 hrs/yr. Assuming an operational cost of $71.88/hr (from a KDOT estimated average) and a 5% improvement in efficiency due to the implementation of AVL, an annual savings of $17,336 would be realized. The moderate estimate of annual savings was $69,344.

Savings from Reduced Accidents

Accident costs were calculated based on USDOT recommendations. (USDOT, 1994) Costs per accident were adjusted to 2002 dollars using the implicit outlay deflators recommended by the US Bureau of Economic Analysis. (2001) The accident counts used were statewide counts taken from the Kansas Accident Records System (KARS) for the year 2000. (KDOT, 2001) To ensure a conservative estimate, the number of accidents was used, rather than the number of parties involved. If more than one party was injured in any given accident, only the maximum injury as indicated by KARS was considered. The data was filtered to extract only accidents in which the reported pavement condition was snow covered, snow packed, or ice covered. It should be noted that while winter maintenance operations planning is done in terms of winter seasons which span two consecutive years, both weather records and accident records (as well as economic parameters) are archived based on the calendar year. Consequently, snow-related accidents for the year 2000 will involve accidents that occurred in two different winter seasons. The calculations performed are valid, however, so long as the time frame used is consistent.

To estimate the effects of AVL implementation on accident costs, it was assumed that efficiency in snow removal would be improved by 5% for the conservative estimate, with a corresponding 5% reduction in snow-related accidents in each severity category. For each of those accidents affected, it was assumed that injury accidents would have fallen into the next less severe category (property damage only accidents would be averted). For example, for every 100 accidents resulting in a debilitating injury to at least one occupant, it was assumed that implementing AVL would result in five of those accidents (5%) being classified as non-debilitating injury instead of debilitating injury. Downgrading accident severity was used rather than simply reducing the number of accidents in each category in order to ensure a conservative estimate. Based on these assumptions, the total annual statewide savings in accident costs due to AVL implementation is conservatively estimated to be $5,865,296, as shown in Table 3.

TABLE 3. Statewide Savings in Accident Costs (Conservative Estimate) (*5, 6, 7*)

KARS severity code	F	D	I	P	N	
Description	Fat	Debilitating	Non-Debilitating	Possible	PDO	Notes
Cost per event (1994 dollars)	$ 2,600,000	$ 180,000	$ 36,000	$ 19,000	$ 2,000	1
Cost per event (2002 dollars)	$ 2,981,557	$ 206,415	$ 41,283	$ 21,788	$ 2,294	2
Accidents	24	100	516	600	5421	3
Involved Parties	43	134	794	967	8185	4
Total Weather Costs	$ 71,557,369	$ 20,641,549	$ 21,302,078	$ 13,072,981	$ 12,433,093	
Accidents Affected	5%	5%	5%	5%	5%	5
Savings	$ 3,330,170	$ 825,662	$ 502,966	$ 584,844	$ 621,655	6
				Total Accident-Related Savings	**$ 5,865,296**	

1) Data Source: FHWA, 1994 Technical Advisory
2) Adjusted for 2002 using deflators from the Bureau of Economic Analysis
3) Source: 2000 KARS Data
4) Shown for comparison only. To ensure a conservative estimate, accidents are used in calculations.
5) Percentage of accidents affected was arbitrarily set as a conservative estimate.
6) Savings result from 1% of accidents in each category being downgraded to the next lesser severity.

Because the amount of snowfall can vary widely from year to year and from one location in the state to another, historical records were examined to determine what relationship the snowfall in the year 2000 has to the average annual snowfall. Data was obtained from the High Plains Regional Climate Center at the University of Nebraska, Lincoln. (HPRCC, 2002) The data shows that the winter of 2000 was a relatively mild winter compared to historical averages for Kansas, at least with respect to snowfall. Eleven locations spread across the state were used as representative samples of the snowfall in their respective regions. Of the eleven locations considered, nine experienced snowfalls below normal in 2000, five of them by more than 6 inches. Wichita and Yates Center, the two locations that experienced snowfall above normal for the year, exceeded the annual averages by just 4.1 and 3.7 inches, respectively. Based on this data, the use of snow-related accident data from the year 2000 is likely to be a conservative estimate of what can be expected annually.

To generate a moderate estimate of accident savings, a percent reduction of 20% was used, the average value reported in the benefits survey. Additionally, the number of parties involved in accidents was used in place of the number of accidents. For example, an accident involving two cars in which 4 people were injured would count as 1 in the conservative estimate and 2 in the moderate estimate. Some adjustment for snowfall was merited, though the data available is not sufficiently detailed to provide any statistically based adjustment. The average percent difference between the average annual snowfall and the year 2000 snowfall for the locations considered was a little more than a 27% decrease. Because one of the more heavily populated areas, Wichita, had above normal snowfall, a conservative adjustment was warranted. For the moderate estimate of cost savings due to AVL implementation, it was assumed that the typical year would experience 13.6% more snowfall than occurred in 2000 (half of the average across the locations), and a corresponding increase in snow-related accidents would occur. The resulting calculations are tabulated in Table 4.

TABLE 4. Statewide Accident Savings (Moderate Estimate) (*5, 6, 7,)*

KARS severity code	F	D	I	P	N	
Description	Fat	Debilitating	Non-Debilitating	Possible	PDO	Notes
Cost per event (1994 dollars)	$ 2,600,000	$ 180,000	$ 36,000	$ 19,000	$ 2,000	1
Cost per event (2002 dollars)	$ 2,981,557	$ 206,415	$ 41,283	$ 21,788	$ 2,294	2
Accidents	24	100	516	600	5421	3
Involved Parties	43	134	794	967	8185	4
(Adjusted for avg snowfall)	49	152	902	1099	9298	5
Total Weather Costs	$ 145,643,098	$ 31,421,391	$ 37,236,693	$ 23,934,711	$ 21,325,380	
Accidents Affected	20%	20%	20%	20%	20%	6
Savings	$ 27,112,023	$ 5,027,423	$ 3,516,799	$ 4,283,053	$ 4,265,076	7

Total Accident-Related Savings $ 44,204,374

1) Data Source: FHWA, 1994 Technical Advisory
2) Adjusted for 2002 using deflators from the Bureau of Economic Analysis
3) Source: 2000 KARS Data
4) Shown for comparison only. To ensure a conservative estimate, accidents are used in calculations.
5) Increased by 13.6% to account for below average snowfall in 2000, the year from which accident data was used.
6) Percentage of accidents affected was arbitrarily set as a conservative estimate.
7) Savings result from 1% of accidents in each category being downgraded to the next lesser severity.

Based on the moderate assumptions, the total annual savings in snow-related accident costs resulting from a statewide AVL implementation would be $44,204,374.

COST-BENEFIT COMPARISON

Three implementation scenarios were considered. After the pilot test completion in 2004, the aggressive implementation assumes one district is added to the system each year until the system is complete. The moderate implementation assumes full implementation occurs over 10 years, and the conservative implementation assumes the full implementation occurs over 20 years. Table 5 shows the net annual savings, net present value (NPV), and benefit-cost ratio (B/C) for each implementation scenario based on moderate assumptions. The investment cost figures shown in the tables are based on the total implementation cost shown in Table 2, converted to 2002 dollars to account for the differing implementation schedules.

It should be emphasized that this analysis considered only those benefits that could be foreseen and could be quantified with reasonable confidence. Some benefits, such as reductions in response time for emergency situations, cannot be reliably expressed in monetary figures, but are nonetheless real benefits. They should be considered in addition to those represented in the quantitative cost-benefit analysis. Additionally, there will almost certainly be benefits that cannot be foreseen. AVL is a mature technology, but its development has occurred mostly in the commercial vehicle, transit, and emergency services communities. The application of AVL to highway maintenance is relatively recent, and the spectrum of potential benefits is still being explored.

TABLE 5. Moderate Estimate of NPV for Statewide Implementation

Disc. Rt. 5%	Aggressive Implementation		Moderate Implementation		Conservative Implementation	
Year	Pct Complete	Net Savings (2002 Dollars)	Pct Complete	Net Savings (2002 Dollars)	Pct Complete	Net Savings (2002 Dollars)
2003	0%	$ -	0%	$ -	0%	$ -
2004	17%	$ 6,825,780	10%	$ 4,095,468	5%	$ 2,047,734
2005	33%	$ 13,001,485	20%	$ 7,800,891	10%	$ 3,900,446
2006	50%	$ 18,573,551	30%	$ 11,144,130	15%	$ 5,572,065
2007	67%	$ 23,585,461	40%	$ 14,151,277	20%	$ 7,075,638
2008	83%	$ 28,077,930	50%	$ 16,846,758	25%	$ 8,423,379
2009	100%	$ 32,089,063	60%	$ 19,253,438	30%	$ 9,626,719
2010	100%	$ 30,561,012	70%	$ 21,392,709	35%	$ 10,696,354
2011	100%	$ 29,105,726	80%	$ 23,284,581	40%	$ 11,642,290
2012	100%	$ 27,719,739	90%	$ 24,947,765	45%	$ 12,473,883
2013	100%	$ 26,399,751	100%	$ 26,399,751	50%	$ 13,199,876
2014	100%	$ 25,142,620	100%	$ 25,142,620	55%	$ 13,828,441
2015	100%	$ 23,945,353	100%	$ 23,945,353	60%	$ 14,367,212
2016	100%	$ 22,805,098	100%	$ 22,805,098	65%	$ 14,823,314
2017	100%	$ 21,719,141	100%	$ 21,719,141	70%	$ 15,203,399
2018	100%	$ 20,684,896	100%	$ 20,684,896	75%	$ 15,513,672
2019	100%	$ 19,699,901	100%	$ 19,699,901	80%	$ 15,759,921
2020	100%	$ 18,761,810	100%	$ 18,761,810	85%	$ 15,947,539
2021	100%	$ 17,868,391	100%	$ 17,868,391	90%	$ 16,081,552
2022	100%	$ 17,017,515	100%	$ 17,017,515	95%	$ 16,166,639
2023	100%	$ 16,207,157	100%	$ 16,207,157	100%	$ 16,207,157
Total		$ 439,791,381		$ 373,168,650		$ 238,557,229
Inv. Cost		$ 7,225,610		$ 6,595,465		$ 5,322,254
NPV		**$ 432,565,771**		**$ 366,573,185**		**$ 233,234,975**
B/C		**28.4**		**27.4**		**24.3**

Based on the moderate assumptions, the agency savings in reduced paperwork and improved fleet management total $1,766,660/year. Annual maintenance costs are estimated to be $818,500/year. According to these numbers, the system would more than pay for its operational costs in efficiency savings.

The Benefit-Cost Ratio would be at least 2.6 (based on the conservative assumptions) and would probably be 24 or higher (based on the moderate assumptions). Depending on the implementation schedule, AVL would likely result in a net benefit of between $233 million and $433 million over 20 years (Net Present Value, 2002 dollars).

CONCLUSIONS

Because the application of AVL to highway maintenance is a relatively recent phenomenon, quantitative data that defines the benefits are not available. In the FHWA's most recent report on ITS benefits, when AVL and other operations and maintenance applications are discussed, the authors summarize the state of the practice, "As implementation of these systems expands, quantified benefits of their use will become apparent. However, there are no benefits data available at this time." (*8*) This lack of data emphasizes the need for including the evaluation from the outset of the implementation planning. As an afterthought, evaluation seldom gains the momentum necessary to generate funding.

In spite of the lack of quantitative studies, the evidence seems clear that there are real benefits and that the likely magnitudes of those benefits are large enough to justify deployment from a cost-effectiveness perspective. The literature and the results of the survey conducted during this study suggest that AVL can provide a significant benefit to highway maintenance operations. The cost-benefit ratio is almost certainly greater than 1, and probably greater than 20. A moderate estimate of the net present value of statewide implementation ranges from $233 million to over $433 million over 20 years, depending on the implementation schedule. The annual efficiency savings for the Department are estimated to be nearly twice the annual maintenance cost of the system. So, in addition to paying for itself, the system will provide excess fiscal benefit that can be used to improve other aspects of maintenance.

The potential for AVL to improve the efficiency and effectiveness of highway maintenance operations appears to be significant. Because the technology is well established and there is some precedent among transportation agencies from which to learn, AVL implementation can be done cost-effectively and with a high level of confidence that the system will prove beneficial. The agency and user cost savings afforded by AVL make the technology a very appealing tool for highway maintenance activities, and the state of the practice is ready to support reliable deployment. With proper attention to planning and evaluation, AVL can help KDOT and other transportation agencies further improve the quality of highway transportation.

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Using Reidentification to Evaluate the SCOUT Traffic Management System (TMS)

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ABSTRACT

The Kansas City SCOUT Traffic Management System (TMS) is a cooperative effort funded by the State Departments of Transportation of Kansas and Missouri and involving the DOTs, local and state law enforcement, Mid-America Regional Council (MARC—KC MPO), local municipalities and townships, and others. The system integrates dynamic message signs, video surveillance, traffic detection, computer networking and analysis, and a Traffic Operations Center.

In the evaluation of SCOUT, incident-related delay was chosen to be the primary measure of effectiveness, because that is where the greatest benefits of the system are expected to occur. To measure this delay, an approach was developed that utilizes video observation and post-processing to obtain travel times throughout the peak periods. The travel times were obtained by employing characteristic-based reidentification, aided by a custom computer application developed at The University of Kansas, *ReID*. Using *ReID*, temporal profiles across the peak periods can be generated of the average travel time for each corridor under study. Profiles are generated for baseline (i.e., non-incident) conditions and for each period during which an incident occurred on the study segment while it was under observation. The baseline profile is subtracted from the incident profiles to obtain the delay caused by the incident. Comparing the delay caused by incidents before the implementation of SCOUT with those after the implementation of SCOUT will show any improvements caused by the SCOUT system. Data was collected over a total of approximately seven months during 2002 and 2003.

Key words: evaluation— delay—incident —ITS

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INTRODUCTION

Evaluation of Intelligent Transportation Systems (ITS) is a critical issue to sustained progress, providing decision makers the evidence of benefit they need to justify system updates, upgrades, and expansions. Complex ITS deployments, such as urban Traffic Management Systems (TMS) require that significant resources be devoted to evaluation if the evaluation is to be thorough and defensible. While some aspects of ITS evaluation are straight forward (e.g., changes in speeds, throughput, or travel times), others are more difficult to quantify, such as public acceptance, safety benefits, and other benefits related to incidents. Sometimes, the needed historical data does not exist, or the amount of data needed makes the data collection prohibitive.

BACKGROUND

The first phase of deployment of the Kansas City SCOUT Traffic Management System (TMS) is scheduled to be completed in late 2003. Phase I of the deployment includes four primary corridors, the southern leg of I-435, I-35, and I-70 east of the CBD. The system comprises 45 variable message signs (VMS), 75 video surveillance cameras and 1200 inductive loops, all of which are connected to a Traffic Operations Center (TOC) via a region-wide fiber-optic network. SCOUT is jointly funded by the State Departments of Transportation of Kansas and Missouri. In 2001, the DOTs asked the University of Missouri and The University of Kansas to work cooperatively to evaluate SCOUT Phase I.

Between the early stages of planning the system and the beginning of the deployment, various factors led to the postponement of the implementation of ramp metering until a later phase. When ramp metering was deleted from the initial deployment, the focus of the system shifted from recurring congestion to incident-related congestion. While the system could still provide some benefits related to recurring congestion, the greatest benefits are likely to be incident-related. The system is expected to improve incident detection and reduce response times. The VMS can be used to inform drivers, reducing flow to the bottleneck by allowing drivers to choose alternate routes and reducing secondary accidents by warning drivers of the congestion ahead.

APPROACH

The incident-orientation factored heavily in the planning of the evaluation. Many of the measures of effectiveness commonly used in evaluations of other TMS are much less relevant because they pertain specifically to recurring congestion, such as travel time reliability or spot speed characteristics. Measures of effectiveness that relate most directly to incidents include detection, response, and clearance times, secondary accidents (i.e., accidents *caused by* incident related congestion), and various measures of congestion. Common measures such as travel time reliability, throughput, and level of service will all be considered, but it is the incident-related measures that are expected to most clearly show the benefits of SCOUT.

For detection, response, and, sometimes, clearance times, accident reports and motorist assist logs often provide the only available data. The precision of the times must be carefully considered in interpreting the results, but if a change is significant enough to be statistically credible, these times are a very good basis on which to compare *before* and *after* conditions. It is difficult to translate this data into a monetary figure that could be used in a benefit-cost assessment, but they may lend themselves to a cost-effectiveness comparison or some other relative evaluation tool. This kind of data is being pursued for use in the SCOUT evaluation.

Reductions in secondary accidents are a very important kind of benefit of urban ITS deployments. Reliable data, however, is virtually nonexistent. Accident records reports frequently have no place to indicate that an accident was a secondary accident except in a field for general comments. As a result, some records may show no indication that an accident was caused by backup from a previous accident rather than some other cause. With respect to the SCOUT evaluation, the identification would almost necessitate examining location and time of each accident and performing some manner of temporal-spatial analysis to identify likely secondary accidents as those occurring near upstream of a known incident within some time frame, which would have to be dependent to some degree upon the characteristics of the incident. Consideration is still being given to the identification of secondary accidents, but to date no means of reliably distinguishing secondary accidents from primary accidents has been identified.

The measure of effectiveness that is expected to most clearly show a positive change after the implementation of SCOUT is incident-related delay. When SCOUT is fully operational on Phase I corridors, it is expected that all of the incident-related measures already discussed represent areas of real benefits. For each of them the difficulty lies in obtaining the data necessary to quantify (or verify) those benefits. The same difficulty exists for incident-related delay, but the difficulty has been overcome with careful planning and significant effort.

Calculating Incident-Related Delay

Delay can be simply defined as an increase in travel time. Using delay, t_{delay}, as a measure implies that some baseline travel time over a given segment exists for which $t_{delay} = 0$. This baseline mean travel time, *f(t)*, is the travel time when traffic flow is not being affected by any incidents.

If *g(t)* is the mean travel time during or following an incident on the study segment, then t_{delay} can be expressed as shown in Equation 1.

$$t_{delay} = \int_{t=T_0}^{T_f} \left[g(t) v_t - f(t) v_{b,t} \right] dt \qquad (1)$$

where T_0 is any time before the incident occurs, T_f is any time after traffic flow returns to baseline levels (i.e., after the effects of the incident on traffic have completely dissipated), v_t is the traffic volume at time t during an accident, and $v_{b,t}$ is the traffic volume at time t under baseline conditions.

Since no known function *f(t)* or *g(t)* exists, the data must be aggregated temporally to approximate the continuous profile. Samples are extracted from the data to provide an approximation of the temporal profile of travel time across the segment. FIGURE 1 shows a plot of sample data. In the figure, the accident data is actual. However, because the data processing is not yet complete, a hypothetical baseline data set is shown.

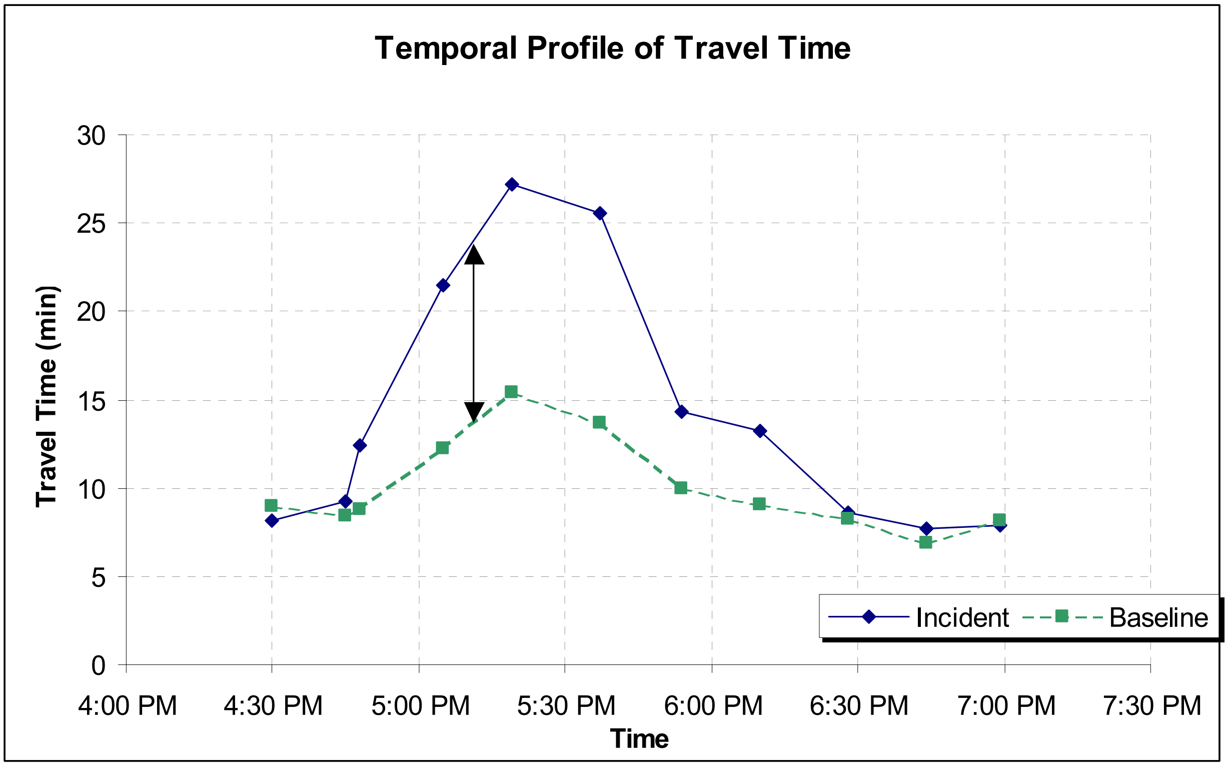

FIGURE 1. Plot of Accident and Baseline Temporal Profiles

The total travel time is the area under the plot, and the delay is the difference between the area under the baseline plot and the area under the accident plot.

Once aggregated, t_{delay} can be calculated as follows.

$$t_{delay} = \sum_{m=0}^{M-1}\left(\frac{tt_m + tt_{m+1}}{2} v_m t_m\right) - \sum_{n=0}^{N-1}\left(\frac{tt_{b,n} + tt_{b,n+1}}{2} v_{b,n} t_n\right) \qquad (2)$$

where tt_m is the average travel time after m time periods, v_m is the traffic volume during time period m, and t_m is the elapsed time between m and $m+1$. Subscript b indicates the parameter is for baseline conditions, otherwise the parameter represents accident conditions.

Measuring Travel Time

The values of tt_m and $tt_{b,n}$ in Equation 3 must be derived from a set of individual travel times. To obtain travel times of individual vehicles, a video surveillance technique was employed. Video cameras were set up to tape the traffic in the through lanes during the peak periods. For each corridor, four video observation sites were used, one for each direction of traffic at each end of the study segment. The distance between the cameras varied between 8 km (5 mi) and 18 km (11 mi), depending on the segment. The video was later catalogued with respect to individual vehicles and their characteristics (e.g., type, color, special markings), along with an observation time stamp. Once the video data was catalogued for the two ends of a segment, the catalogues were compared to find matching records, vehicles observed at both locations. When a match was identified, the difference in the time stamps was taken as the travel time across the segment for that vehicle.

To facilitate the cataloguing and matching of vehicles between the two observation points, a computer package was developed to assist in post-processing the video footage. The package, *ReID*, comprises two modules, the *ReID Cataloguer* module and the *ReID Matcher* module. Initial estimates suggest the software speeds the cataloguing and matching process by a factor of 3 to 5.

ReID Cataloguer

ReID Cataloguer is used to build a catalogue of vehicles that occur in each of the videotapes during a given time period. The user steps through the video, selecting each vehicle and entering its basic characteristics. For each vehicle, the area designated by the user is saved as a bitmap, and the entire video frame is saved as a second bitmap. The characteristics that can be entered include the vehicle type (or class), color, flag indicating a special characteristic such as racing stripes, and the lane the vehicle was in. In the screen shot of *Cataloguer* shown in FIGURE , these characteristics appear in the upper half of the left side of the window. *Cataloguer* can also be used to catalogue license plates, although multiple cameras must be used at each location if more than two lanes are to be catalogued using license plates. With three or more lanes, the resolution of digital video cameras is not sufficient for the plates to be legible. While data is being entered, *Cataloguer* announces each data item audibly. This provides confirmation to the user that the correct key was pressed.

FIGURE 2. Screen Shot of ReID Cataloguer

ReID Matcher

Once the video from both observation points has been catalogued for a given time period, *Matcher* can be used to help identify vehicles in the upstream data that match vehicles in the downstream data. A screen shot of *Matcher* is shown in FIGURE 2. The selected downstream vehicle is highlighted in the list in the bottom left, and the associated bitmap is shown in the top left. *Matcher* determines the most likely matches from the upstream catalogue and displays them as thumbnails. If the user can identify a match, that record is selected and the bitmap is displayed in the top right, side by side with the bitmap from the downstream data. If confirmed, the match is stored, and the speed statistics and distribution shown in the bottom right are updated.

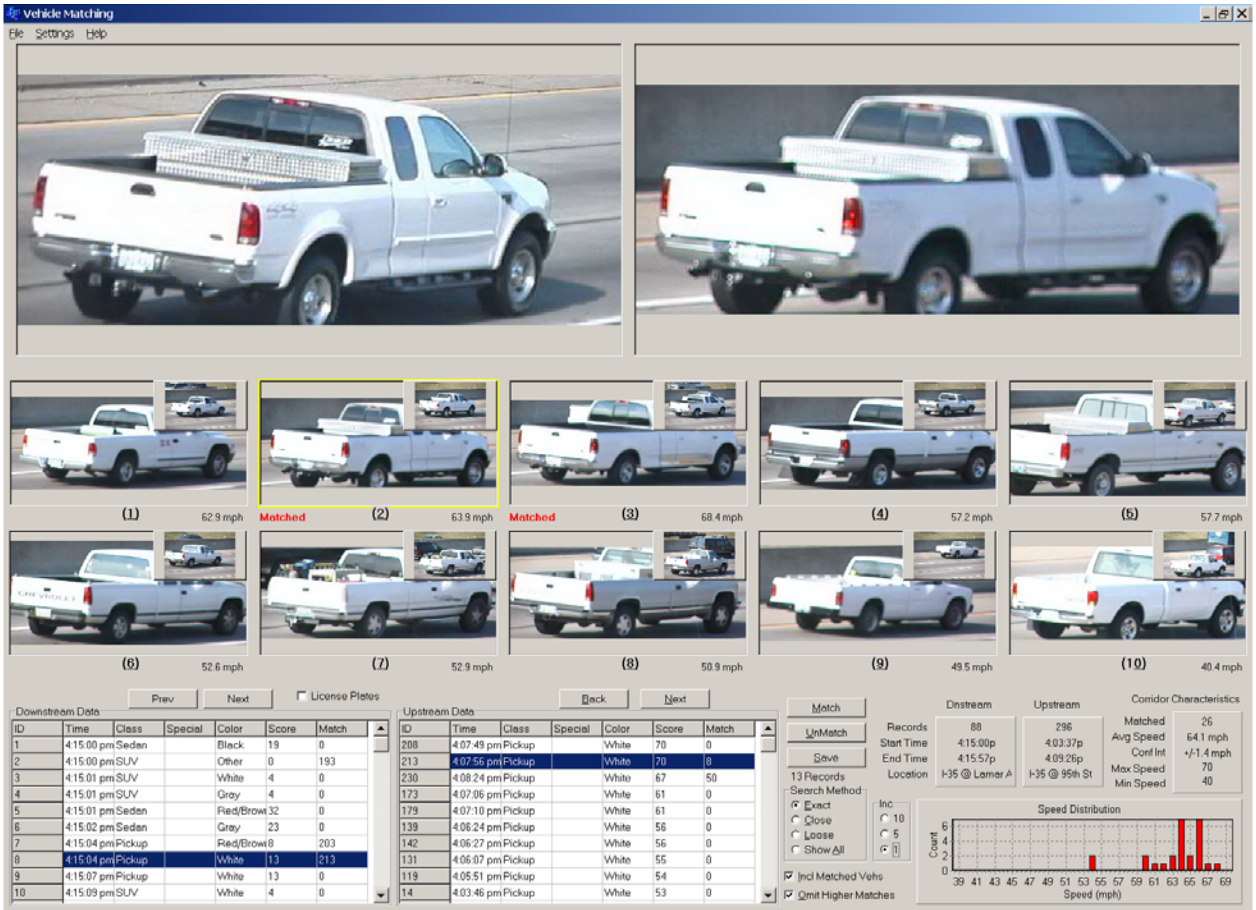

FIGURE 2. Screen Shot of ReID Matcher.

DATA COLLECTION

Digital camcorders were used for the video data collection because their image fidelity is significantly better than that of analog camcorders, and there is no fidelity loss in transferring the video to the computer such as occurs with the analog video capture process. The storage space required for digital video is significant (approximately 3.7 MB/sec), but once a video excerpt is catalogued, the video is no longer needed, because matching is done using the bitmaps extracted from the video by *ReID Cataloguer*.

Video data was collected Monday through Thursday during June through September of 2002. Further *before* data is being collected during Summer 2003, and *after* data will be collected during the summer of 2004 and, if necessary, 2005. Data collection for the morning peak period occurs between 6:30 AM and 8:30 AM. Data collection for the evening peak period occurs between 3:30 PM and 6:30 PM, based on continuous count data obtained from the Kansas Department of Transportation.

Approximately 500 hours of video were collected during Summer 2002. Another 250 hours of video will be collected during Summer 2003. The processing of the video data, even with the aid of the ReID computer modules, is extremely time intensive, requiring an average of approximately 8 hours to process each minute of video data for the segments under study.

DATA ANALYSIS

Once travel times are extracted from the video and incident-related delay is determined for the individual incidents whose effects are captured in the data, some statistical modeling will be necessary to extrapolate the results to characterize delay on an annual basis, and then to compare the annual delay without SCOUT to the annual delay with SCOUT.

MODELING INCIDENT-RELATED DELAY

Statistical modeling may be done with either regression analysis or artificial neural networks to model incident-related delay on the basis of various input parameters. Candidate parameters include the following.

- Lane-minutes of closure
- Total number of lanes
- Density at the time of the incident
- Incident type or measure of severity
- Clearance time
- Total baseline vehicle count for incident time and duration

These parameters will be explored to determine what relationship, if any, each has with incident-related delay, and which combination of parameters best serves to characterize the delay function.

Once a statistical model has been developed for the data collected with and without SCOUT in operation, both models will be applied to a representative set of accidents to determine the difference in incident-related delay that can be attributed to SCOUT. This can then be scaled to provide an estimated annual benefit in monetary terms.

CONCLUSIONS

The challenges in evaluating an incident oriented system such as SCOUT are substantial. The randomness of accidents requires staking out the study segments for long periods of time, and the processing of the data is very time intensive. However, provided that accident rates during the data collection are consistent with recent annual averages, the methodology described in this paper should provide a new look into the potential incident-related benefits of urban ITS deployments.

The use of video-based reidentification is a powerful tool for assessing traffic conditions for a wide variety of applications. Traditional means of reidentifying vehicles from video have been excessively time consuming, severely limiting the application of this technique. The development of the ReID computer modules improves the processing time substantially, broadening the range of applications for which the technique may be feasible.

In addition to providing information about benefits specific to the SCOUT system, the statistical modeling of incident-related delay will provide a foundation for assessing other improvements that affect incident response and clearance time, not only in Kansas City, but across the country.

ACKNOWLEDGMENTS

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Web-Based Database for Highway Erosion and Sedimentation Control Measures

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ABSTRACT

An online database was designed and developed as an Iowa Highway Research Board project by the authors. Our database is intended for identifying and evaluating effective erosion and sedimentation control measures (ESCMs) for use in roadway construction throughout Iowa and elsewhere in the Midwest. The ESCM database is an efficient practical resource to aid state, county, and municipal engineers in the selection of the best management practices (BMPs) for erosion and sediment control. The overall project's goal is to provide a helpful, simplified electronic "office-field manual" readily available prior to, during, and after roadway construction.

The database is based on an extensive review of literature pertaining to ESCMs. The literature review included in-house manuals, information on state department of transportation (DOT) websites, publications from state and federal government levels, industry design manuals, and specialized computer programs. Further considerations was obtained by surveying state DOTs through questionnaires within the Great Plains, Upper Midwest Region and the Iowa County engineers. The critical review of the available resources and the conducted surveys led to the development of an expert system.

The web-based version of the expert system (ES) is a comprehensive "inference engine" that will assist various specialists (owners, contractors, inspectors, etc.) involved in the selection, design, construction, inspection, and maintenance of ESCMs for roadway construction. To the knowledge of the authors, the current database is the most robust electronic "office-field" guide that aids the user in the selection of an appropriate ESCM for specific site conditions. The present paper summarizes design and usage considerations for the developed Iowa Highway Research Board ES.

Key words: best management practices—erosion and sedimentation control measures—web-based manual

INTRODUCTION

Each year, large amounts of soil are eroded from highway sites, especially from highways under construction (*1*). As a whole, soil erosion in Iowa is a major problem with an estimated average annual soil displacement of 9.9 tons per acre per year (*2*). Soil erosion is usually difficult to control at highway construction sites because of the extent of disturbed soil and the difficulty of controlling water runoff. Measuring the overall rates of sediment transported to streams or water basins at these construction sites are limited. The erosional rates are typically 10 to 20 times the rates of agricultural regions; some reports even suggest erosional rates up to 100 times more than cultivated land use today (*1*). The eroded soil incurs severe economic costs (e.g., excavation or dredging, soil replacement, highway consolidation) and environmental impacts (e.g., deterioration of water quality in the watershed and streamside vegetation and removal of important topsoil constituents). Consequently, erosion prevention and sedimentation control are major factors in the design and construction and ultimately maintaining our highways.

Irrespective of project size and erosion-mitigation method, selection of the optimum erosion control measure for a specific situation must be facilitated using a comprehensive, yet straightforward plan. Besides being technically feasible, quick and economical, the current approach in implementing an erosion control project includes compliance requirements with federal, state, and local regulations. Protecting water quality is of paramount concern in this regard. The new Phase II (no relation to Phase II of this project) rules from the Environmental Protection Agency (EPA) concerning storm-water erosion and sediment control practices was passed in Spring 2002 (*3*). Moreover, state departments of transportation (DOTs) must be ready to demonstrate how current methods, as well as new and innovative methods, will meet the water quality standards mandated by the Phase II rules.

Efficient planning for erosion control requires a comprehensive consideration of site topography, drainage pattern, rainfall data, soil data, existing vegetation, off-site features (streams, lakes, buildings), as well as available types and operational characteristics of the erosion control methods. These varied and complex considerations, commonly limit the number of problems encountered in finding feasible and economic methods to minimize erosion. Several disciplines of science and engineering are required to address erosion problems. Highway designers, project engineers, and maintenance personnel often need the advice of hydrologists, hydraulic engineers, soil engineers, soil scientists, agronomists, landscape architects, and other specialists to minimize erosion problems.

The Iowa Highway Research Board (IHRB) project summarized in this paper entailed a comprehensive review of the literature on erosion and sedimentation control measures (ESCMs). The literature was collected through conventional means, internet search, and a survey to various DOTS. A cursory examination of the literature shows that there are numerous guidelines for erosion and sediment control methods used in highway applications. The lack of a centralized source of information has led to a variety of erosion-control designs and procedures now in use during highway construction. Therefore, engineers from state DOTs, counties, and municipalities are forced periodically to conduct extensive literature reviews of various ESCMs. These reviews can be labor intensive, requiring engineers to survey needs, and develop best management practices (BMPs) for erosion and sediment control, and to adapt methods to meet the climatic variations that prevail locally.

The questionnaire survey was sent out to the state DOTs in the Great Plains and Upper & Middle Mississippi Valley (*4*). The collected information from the survey showed that the DOTs use a variety of literature sources for addressing Erosion and Sedimentation Control problems. The common characteristic for all DOTs is that most of them have developed their own in-house compilations of manuals/guidelines, possibly to take into account specific issues related to local state conditions. This study revealed that all the surveyed DOTs rely on guidelines assembled in hardcopy manuals comprising hundreds of pages. Manual layouts commonly follow conventional arrangement of content, i.e., temporary and permanent measures, protection of soil surface, runoff control, and sediment removal. Most of the newer manuals include the provisions related to Storm Water Pollution Prevention Plans (SWPPP), although presented separately. Important considerations involved in the selection process, such as overall efficiency and suitability of ESCMs for particular conditions are not included. It is, therefore, often difficult to use the published literature for a quick identification, assessment, and efficiently selecting site-specific erosion control methods for temporary or permanent use. This finding directed the project to develop a contemporary, computer-based expert system (ES) for use by highway engineers seeking guidance on ESCMs.

Rather than compiling a new set of hardcopy manuals regarding ESCMs, the study initiated the development of an ES to aid ESCM selection. The bulk of the literature surveyed for this study now is enclosed in the expert system. Presented below are the main phases of the ES development.

OBJECTIVES

The objective of our project was to identify and evaluate ESCMs utilized in highway applications in Iowa (and thereby elsewhere in the Midwest) through a literature review. Then we assembled this information in an electronic format allowing for a readily available single reference. This compilation assures a specific BMP is selected for preventing soil erosion at each site, during and after construction. The compiled information, now in a computer software, will aid engineers in the decision-making and design to select and implement ESCMs for a desired highway project. Input from Iowa County Engineers and Iowa DOT personnel was sought in order to ensure accessibility and utility of our ES.

PHILOSOPHY OF DATABASE

Human experts solve problems by using a combination of factual knowledge and reasoning ability. The ES is a type of computer application program that aids decision making or solving problems in a particular field by using knowledge and analytical rules defined by experts in the field (Figure 1). In an ES, both the factual knowledge and reasoning ability are contained in two separate but related components, a knowledge base and an inference engine. The knowledge base provides specific facts and rules about the subject, and the inference engine provides the reasoning navigational tool enabling the ES to output conclusions (Figure 1).

During the ES design, an interdisciplinary team was assembled from geologists, research engineers and computer programmers to collaborate and ultimately translate the rules into terms that a computer can process. The ES designed in this project is a comprehensive “inference engine” aimed at assisting state, county, and municipal engineers in the selection, planning and implementation of ESCMs. It was developed to ensure that the selected ESCMs take into account type and duration of erosion and sediment control (Figure 2), the erosion control purpose (Figure 3), and Iowa’s Midwest environment. The ES reviews user inputs (Figure 4) and suggests potential ESCMs for a particular situation. As well as, providing both general and detailed information on the technical elements involved in ESCM design and construction, inspection, maintenance, removal, economical considerations, and efficiency (Figure 5). The role of the ES is to identify that the appropriate methods for erosion and sediment control are used at each site, during and after construction.

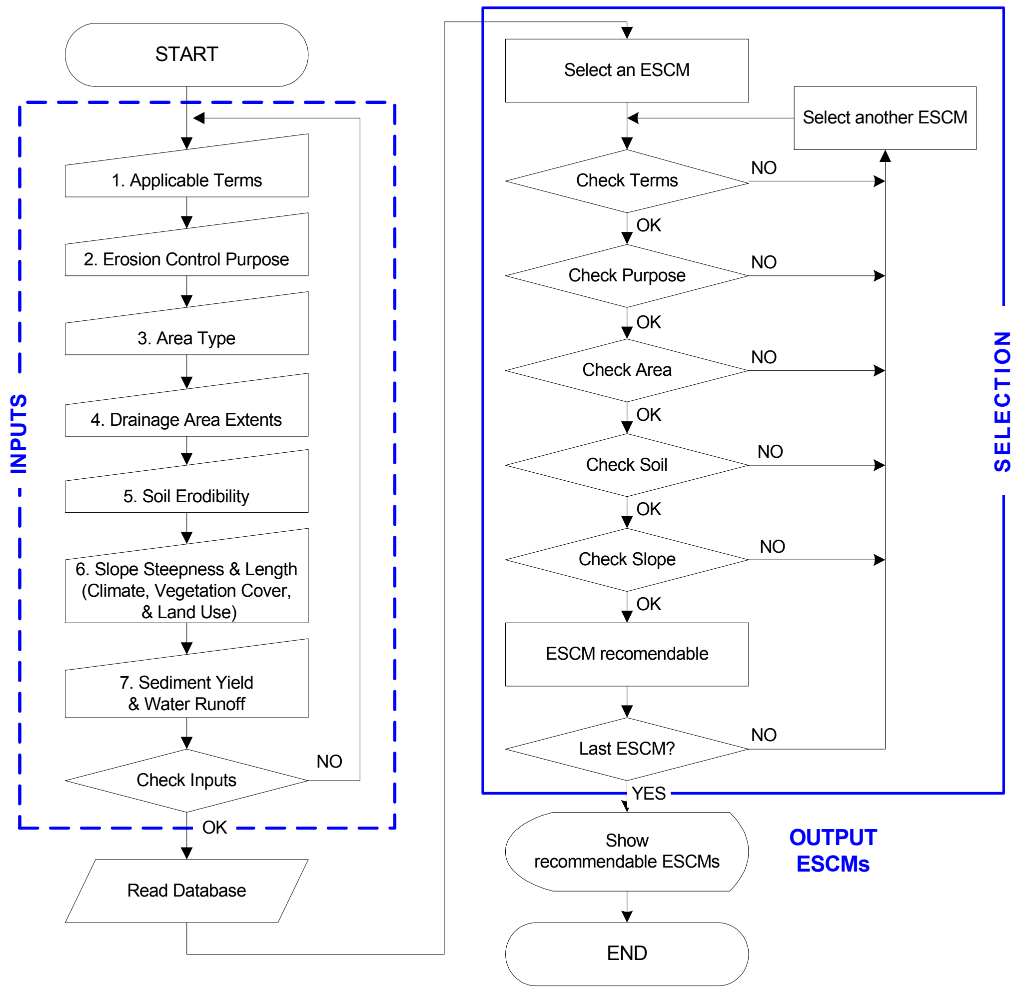

FIGURE 1. Database Input and Output Connections

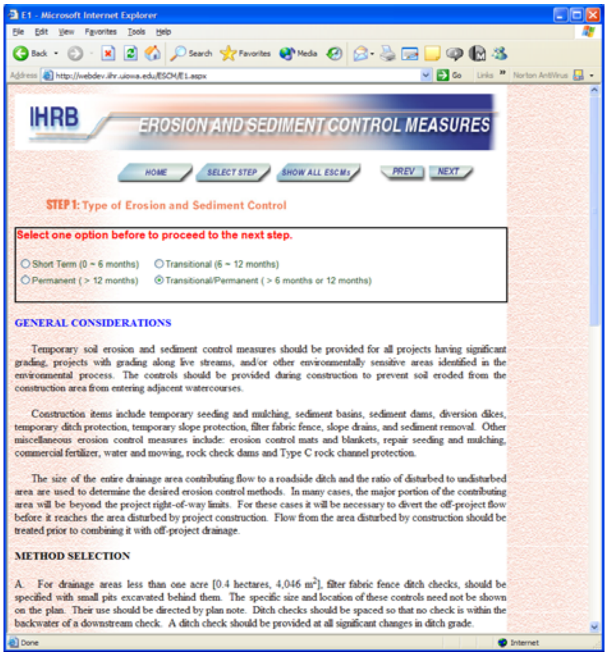

FIGURE 2. Introduction to the ES Navigation Rules

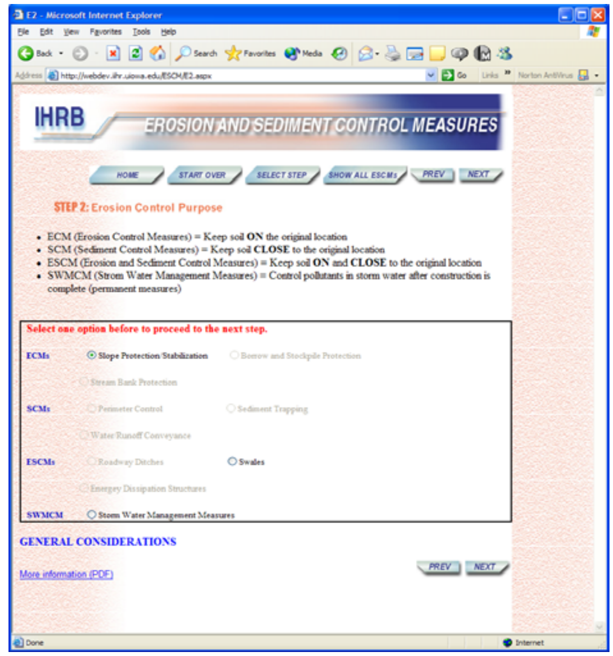

FIGURE 3. ES Interface for Input Data

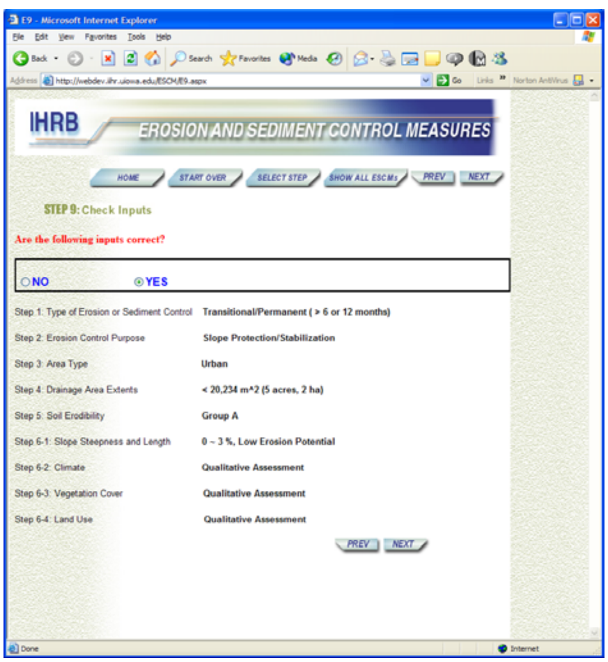

FIGURE 4. Summary of Users Inputs

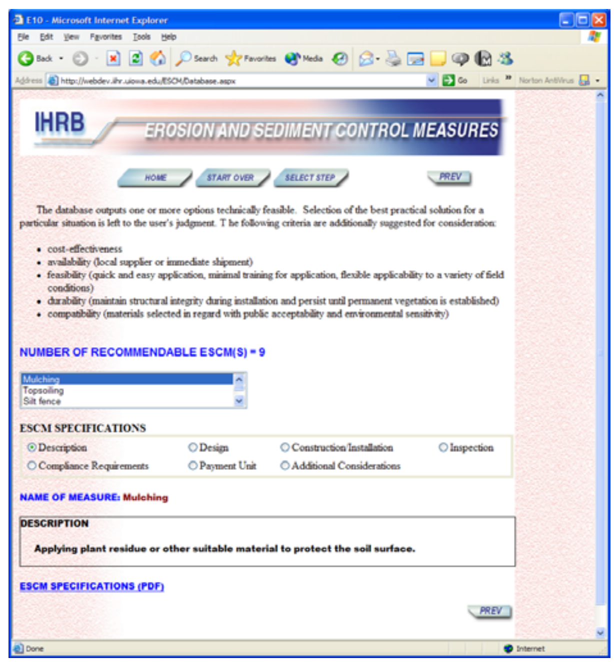

FIGURE 5. ES Outputs

ESCM DATABASE DEVELOPMENT

The ES was developed as a PC-based format first (Phase I), followed by a Web-based version (Phase II). The ES interfaces were designed using Borland C++ Builder, Version 5.0 (Borland Software Corporation). The ES database was organized using Paradox 7 (Corel Corporation). Phase II of the project was developed using ASP.Net using a source code elaborated with Visual Basic .Net (Microsoft Inc.). The following main features characterize the PC and World Wide Web based ES.

Phase I: PC based Version

1. *Comprehensive simulation of the ESCM decision-making process*: All technical elements involved in the selection of the control measures are incorporated in the ES (objectives, type, site evaluation, ESCM specifications). Permitting considerations relevant to selection of the ESCM are also included.
2. *Multi-layered information:* User interfaces for each requested input or output information are contained by two or more layers: the first layer addresses general information valid for classes of ESCM, while the subsequent layers address details pertaining to specific factors or selected ESCM. Given the fact that field engineers are only occasionally implementing ESCMs, the terms of the interfaces are explained in plain language to accommodate various technical backgrounds.
3. *Self-contained*: The various levels of information make the ES a comprehensive source of information that does not need additional references to guide in the selection of the appropriate ESCM for a particular situation. When needed, the user is directed to additional sources of information regarding data collection, data interpretation, and evaluation.
4. *Portability:* The PC-based design of the ES assumes user access to PC. Each of the steps involved in the decision making process can be printed as hardcopy containing exclusively the specifications related to the ESCM of interest.
5. *Compact format and efficient navigation:* Use of the multi-layer structure allows minimization of the number of ES interfaces. Navigation rules are simple and straightforward, thereby enabling users to form queries, provide information, and efficiently interact with the ES. The ES prompts the user when input data are incomplete or the functions are not yet implemented in the engine.
6. *Compact format and efficient navigation:* Use of the multi-layer structure allows minimization of the number of ES interfaces. Navigation rules are simple and straightforward, thereby enabling users to form queries, provide information, and efficiently interact with the ES. The ES prompts the user when input data are incomplete or the functions are not yet implemented in the engine.
7. *Flexibility:* The design of the ES allows unlimited further development and upgrading of the database with minimum changes to the core ES elements.
8. *Iowa specific:* Though the information assembled in the ES is collected from various state DOTs and Iowa counties, priority was given to include ESCMs evidently best suited for Iowa and to rely in principal on the literature resources available in the state.

Phase II: World Wide Web Version

Phase I of the project has led to a convenient, PC-based ES for identifying and evaluating potentially effective ESCMs for use in roadway construction throughout Iowa and elsewhere in the Midwest. Given that this first version of the ES is PC-based and addresses limited user categories, we anticipated that the ES can be considerably enhanced by substantiating the level of detail in the knowledge database and by transitioning the developed application to a web-based platform. The objectives of a Phase II of the project have been accomplished through the following tasks:

1. *Review the ES PC database*: ensuring it includes IDOT in-house expertise in ESCMs. This step entails close collaboration with IDOT personnel to add specifications contained in Plan Notes, Special Provisions, Supplemental and Standard Specifications for each ESCM. During Phase I of

work, several sources were inaccessible to the project investigators (e.g., files in MicroStation format). Also, the available resources did not readily facilitate preparation of specific materials in electronic format (through scanning).

2. *Implementing the quantitative assessment*: the quantitative assessment part of the ES was not active following Phase I of the project. The addition of a inference engine for the quantitative assessment extends the functionality of the ES to additional user categories (e.g., design engineers, consultant engineers, etc)
3. *Transitioning the ES from PC- to web-based platforms*: The PC-based ES synthesized the available information on ESCMs, but it was not built using state-of-the-art information transfer technologies. By adapting the PC-based ES to the web-based environment it is possible to take advantage of important capabilities of the web-based tools, such as the following:

    a. Centralized management of the ES knowledge base
    b. Efficient and quick upgrading of the database (in matters of hours the database can include the latest updates deemed necessary and be ready for users).
    c. Quick dissemination.
    d. Avoidance of installation conflicts due to the variety of PC hardware and software
    e. Minimum maintenance
    f. Efficient use of links to other relevant sources of information available on internet.

4. *Training*: including an instructor's manual, and delivers training for users. The ES developed in the first phase of the project is in a simple format with user-friendly interfaces. However, training sessions would be good opportunities to speed up product implementation/dissemination. Moreover, a training program could foster, through hands-on demonstrations and direct interaction with the users, further actions to enhance the functionality and content of the ES.

CONCLUSIONS

An extensive literature review on ESCMs revealed most of the available guidelines and instruction manuals for ESCM implementation are in conventional format. We've found that the hardcopy and compact disk manuals are not readily available for efficient use and requiring a familiarity with ESCMs in order for selection. The situation led the authors to initiate a simplified expert system that incorporates knowledge and inference rules derived by specialists in the field.

The paper demonstrates through Phase I of the project, we have successfully incorporated the literature review in a PC database platform. Phase II further evolved the ES's input and output capabilities by transitioning the PC version of the database to a World Wide Web version.

The final WWW ES product is an efficient decision-making tool requiring minimum preparation, quite user-friendly, and beneficial by updating capabilities by the computer administration. The newly created erosion and sediment control measure database may be accessed virtually by any specialist, from Highway Construction Engineers to State Environmentalists. The ES potential can be used as a design tool in connection with environmental protection plans and part of a training program for specialized DOT personnel.

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Improving Left-Turn Safety Using Flashing Yellow Arrow Permissive Indications

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ABSTRACT

Significant variability exists in the application of protected/permissive left-turn (PPLT) signal displays throughout the United States. PPLT signal phasing provides a protected phase for left-turns as well as a permissive phase during which left-turns can be made if gaps in opposing traffic allow, all within the same signal cycle. Although the intent of the Manual on Uniform Traffic Control Devices (MUTCD) is to provide a national standard, only general guidance is provided in the selection and use of PPLT signal displays. Additionally, the MUTCD does not require a separate PPLT signal display for PPLT signal phasing. Consequently, PPLT signal displays have been implemented in a variety of configurations throughout the United States.

PPLT signal phasing and corresponding displays can be found at approximately 29 percent of signalized intersections in the United States. The five-section cluster is the most common arrangement, used at approximately 63 percent of all PPLT intersections, but is not uniformly applied in placement, location, and use of supplemental signs. Within each PPLT signal display, the MUTCD requires a green arrow for the protected left-turn movement and a circular green indication for the permissive movement.

Problems with driver's comprehension of PPLT signal displays have been identified but not resolved. Specifically, the permissive (circular green) phase is a concern for many traffic engineers. The problem lies in the fact that drivers traveling through an intersection displaying a circular green indication may proceed straight through, with all other vehicles yielding the right-of-way. Drivers turning left with a circular green indication are required to yield the right-of-way to opposing vehicles before proceeding. Therefore, the circular green indication has been challenged on the premise that it provides two different messages. The safety of left-turn drivers requires that the permissive phase be an unambiguous signal display arrangement and/or indication because of its unique turning requirements.

To improve driver comprehension, traffic engineers in California, Delaware, Michigan, and Washington, among others, have replaced the circular green permissive indication with one of several unique indications. These unique indications include a flashing circular yellow, a flashing circular red, a flashing yellow arrow, and a flashing red arrow. Each of these permissive indications has been used in either a three-section or four-section signal display.

Research has shown that flashing permissive indications may lead to a higher level of driver comprehension and improve safety at PPLT intersections. Additionally, results of a national study have concluded that flashing yellow arrow permissive indications provide driver

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comprehension and safety benefits. Work is currently underway on making the flashing yellow arrow indication an accepted/approved indication in the MUTCD.

Key words: driver behavior—driving simulation—left-turn safety—protected/permissive—signal display

INTRODUCTION

The flexibility provided by the Manual on Uniform Traffic Control Devices (MUTCD) has led to multiple signal display arrangements and indications for Protected/Permissive Left-Turn (PPLT) applications (*1*). Many states have adopted either the five-section cluster (doghouse), horizontal, or vertical display, providing a green arrow for the protected phase and a circular green for the permissive phase. Problems with PPLT signal phasing, primarily related to the circular green permissive indication, have been identified but not resolved (*2*, *3*).

Many traffic engineers believe that the MUTCD circular green permissive indication is adequate and properly presents the intended message to the driver. Other traffic engineers believe that the circular green permissive indication is not well understood and therefore inadequate. The latter belief is based on the argument that left-turn drivers may interpret the circular green permissive indication as a protected indication, creating a potential safety problem.

To overcome this potential problem, traffic engineers have developed at least four variations of PPLT permissive indications. These variations replace the circular green permissive indication with a flashing circular red, flashing circular yellow, flashing red arrow, or flashing yellow arrow indication. Additionally, variations in signal display arrangement and placement are applied. This variability has led to a myriad of PPLT signal displays and permissive indications throughout the U.S. that may confuse drivers and lead to inefficient and unsafe left-turn operations.

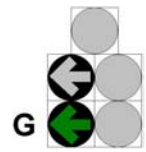
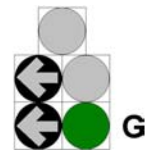
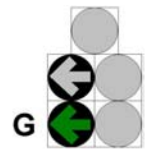
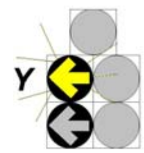
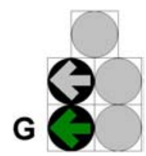
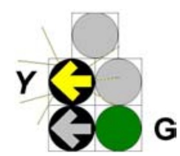
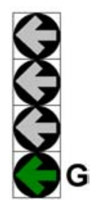
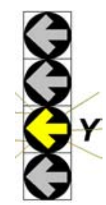
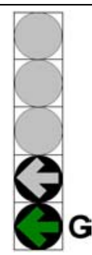
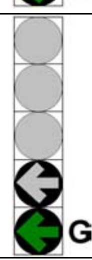
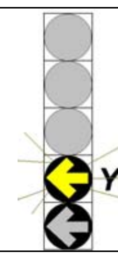
Variability in left-turn control led the National Cooperative Highway Research Program (NCHRP) to introduce project 3-54; a study of the various aspects of PPLT signal phasing. The objective of the NCHRP 3-54 project was to evaluate the safety and effectiveness of different signal displays and phasing for PPLT control through laboratory and field studies. Conducted over a seven-year period, NCHRP 3-54 is the most comprehensive study of PPLT displays to date. The research project identified current practice, studied driver understanding of known protected and permissive displays in the United States, analyzed crash data, analyzed operational data, and conducted a confirmation study using a full-scale driving simulator and several field installations to study driver understanding of the most promising permissive displays (*4, 5*). The results of a driver behavior and comprehension study included in NCHRP 3-54 are presented in the following sections.

RESEARCH OBJECTIVES

The objective of the research task described was to evaluate the safety and effectiveness of selected PPLT signal displays through a driver behavior and comprehension evaluation. The study was conducted using full-scale fixed-base driving simulators located at the University of Massachusetts – Amherst (UMass) and at the Texas Transportation Institute (TTI). An evaluation of the same PPLT signal displays in a static environment was also completed at both locations to provide comparison data to the simulator experiment as well as to previous research tasks.

METHODOLOGY

Twelve different PPLT signal displays were identified for evaluation (*4, 6*). The selected displays differ in permissive indication, arrangement, location, and through movement indication. Each of the PPLT signal displays included only the circular green and/or flashing yellow arrow permissive indications. The circular green permissive indication represented the current state-of-the-practice and the flashing yellow arrow permissive indication represented the most promising alternative based on previous research finding (*2* - *5*). Figure 1 depicts each of the PPLT signal displays evaluated.

Scenario[a]	Lens Color and Arrangement	Left-Turn Indication[b]	
		Protected Mode	**Permissive Mode**
1, 2			
3, 4			
5, 6			
7, 8	 or 	 or 	 or 
9, 10			
11, 12			

R = RED Y = YELLOW G = GREEN *Y* = FLASHING YELLOW

[a] 1, 3, 5, 7, 9, 11 – Circular green through indication; 2, 4, 6, 8, 10, 12 – circular red through indication

[b] The indication illuminated for the given mode is identified by the color letter

FIGURE 1. PPLT Displays Evaluated in Driving Simulator Experiment

The UMass and TTI driving simulators are pictured in Figure 2. Both simulators were fixed-base fully-interactive dynamic driving simulators in which drivers are capable of controlling the steering, braking, and accelerating similar to the actual driving process; the visual roadway adjusts accordingly to the driver's actions. The vehicle base of both driving simulators is a four-door Saturn sedan. Three separate images are projected to a large semi-circular projection screen creating a "visual world" field-of-view which subtends approximately 150-degrees.

Development of Simulation

One intersection approach was created for each of the 12 experimental PPLT signal displays; the characteristics of each approach were identical minimizing confounding variability. Additionally, several intersections that require the driver to turn right, proceed straight, or to turn left on a protected green arrow were included as part of the visual worlds. Further experimental variability was provided by creating multiple driving modules and starting positions. All experimental signal displays within the simulation rested in a circular red or arrow indication as drivers approached the intersection. Approximately 30 meters prior to the intersection stop bar, the PPLT signal display was triggered and changed from the red indication to the selected permissive or protected indication. Similarly, the through movement indication either remained red or changed from a circular red to a circular green indication.

Each of the PPLT signal displays was evaluated with opposing traffic at the intersection. Six opposing vehicles were used to create a predetermined gap sequence. Two vehicles were always positioned at the stop bar in the two through lanes opposing the left-turn driver. The remaining four vehicles were positioned further upstream in a three and seven seconds series of seven-three-seven-seven; therefore, opposing vehicles crossed the intersection seven, 10, 17, and 24 seconds behind the two initially queued opposing vehicles.

A second trigger, similar to that used to change the signal indications, was placed approximately five feet from the left-turn stop bar at each PPLT intersection to release the opposing traffic. This trigger position required drivers to make a decision as to the meaning of the PPLT signal indication and desired action before knowing the actions of the opposing traffic.

FIGURE 2. UMass and TTI Driving Simulators Used in the Experiment.

Drivers were navigated through the modules by guide signs provided on each intersection approach. In addition, drivers were asked to observe speed limit signs (30 mph), providing a higher level of realism and speed control during the experiment. Drivers' responses to each PPLT signal display scenario were manually recorded as correct or incorrect. Incorrect responses were further classified as being fail-safe or fail-critical. A fail-safe response was one in which the driver did not correctly respond to the PPLT signal indication, but did not infringe on the right-of-way of opposing traffic. A fail-critical response was an incorrect response in which the driver incorrectly responded to the PPLT signal indication and impeded the right-of-way of opposing traffic, creating the potential for a crash.

Video-Based Static Evaluation

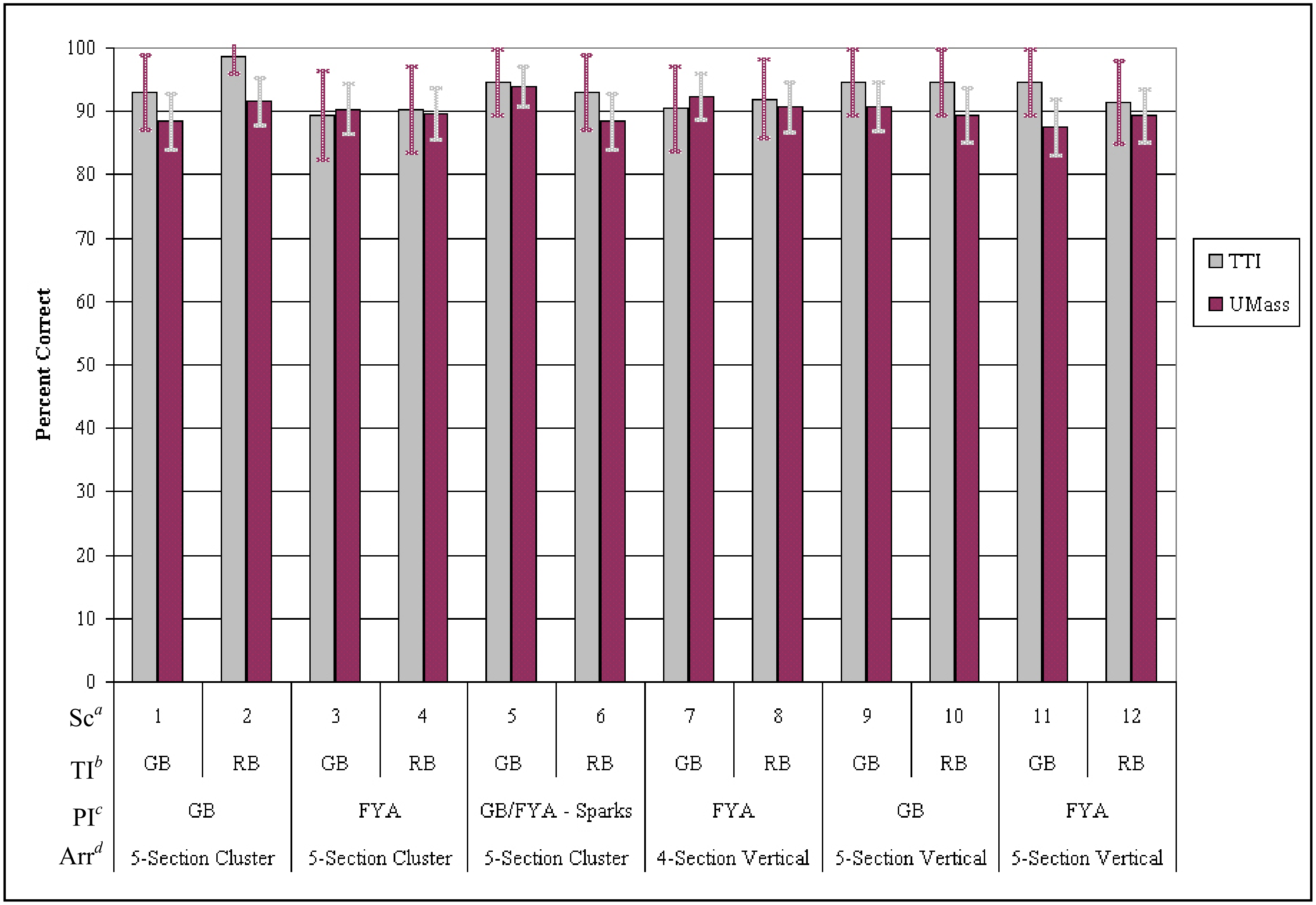
After completing the driving portion of the study, drivers were asked to participate in a static evaluation of PPLT signal displays. The static evaluation was administered using videocassette recordings of screen captures for the 12 PPLT displays. Each display was shown for 30 seconds during which time the driver verbally indicated how they would react. Data were recorded and combined with the driving simulator data to complete the analysis. An analysis of variance (ANOVA) procedure was used to evaluate and compare driver comprehension related to the 12 selected PPLT signal displays (7). For each analysis, the 95 percent confidence interval was calculated based on a binomial proportion.

RESULTS

Two hundred twenty-three drivers at UMass evaluated 2,528 scenarios with experimental PPLT signal displays; 93 drivers at TTI evaluated 874 scenarios. The percentage of correct responses for each of the 12 PPLT signal displays at UMass and TTI are presented in Figure 3. Note that the vertical line segment at the top of the solid bars in Figure 3 represents a 95 percent confidence interval for the results. To compare the data sets, the percent of correct responses was cross-analyzed across each of the 12 experimental displays evaluated by geographic location. The analysis found no statistically significant differences in the percentage of correct responses across the 12 PPLT signal displays ($p = 0.592$). Based upon this statistical analysis and because the UMass and TTI experiments were procedurally equivalent, the UMass and TTI data were combined for analysis.

Further evaluations of the data were completed considering independent variables of which the PPLT display is comprised, including the permissive indication, arrangement, location, and through indication. These results are presented in Table 1. Left turn permissive indications were either circular green (GB), flashing yellow arrow (FYA), or a simultaneous combination (GB/FYA) of the two displays referred to as the Sparks display. Arrangements evaluated were either five-section cluster, four-section vertical, or five-section vertical. Location was either shared or exclusive and described the location of the PPLT section head. The through indication was either circular green or circular red (RB).

The percentage of correct responses by permissive indication ranged from 90 to 92 percent; however, the differences in correct responses as a result of permissive indications were not statistically significant ($p = 0.433$). Similarly, the arrangement of the PPLT signal display was not significant in determining driver comprehension ($p = 0.747$), nor was the through indication ($p = 0.716$) or the location of the PPLT signal display ($p = 0.206$).



[a] Scenario identification number
[b] Indication for adjacent through lanes (GB = circular green; RB = circular red)
[c] Left-turn permissive indication (GB = circular green; FYA = flashing yellow arrow)
[d] PPLT signal display arrangement

FIGURE 3. Percent Correct for each PPLT Signal Display by Study Location (with 95% C.I.)

TABLE 1. Percent Correct by PPLT Display Component

PPLT Display Component	Level	Observations	Percent Correct	95% C.I.	Statistical p-value
Permissive Indication[a]	GB	1,136	91	2	0.433
	FYA	1,701	90	1	
	GB/FYA	565	92	2	
Arrangement[b]	5-section cluster	1,697	91	1	0.747
	4-section vertical	569	91	2	
	5-section vertical	1,136	90	2	
Thru Indication[c]	GB	1,707	91	1	0.716
	RB	1,695	91	1	
Location[d]	Shared	846	90	2	0.206
	Exclusive	2,556	91	1	

[a] Left-turn permissive indication (GB = circular green; FYA = flashing yellow arrow)
[b] PPLT signal display arrangement
[c] Indication for adjacent through lanes (GB = circular green; RB = circular red)
[d] Location of PPLT Signal Display

Analysis of Incorrect Responses

A significant amount of fail safe by movement responses were observed with scenario one; a five-section cluster in a shared location with a circular green permissive indication and adjacent circular green through indication. Across PPLT signal displays, no significant differences were observed in terms of the percentage of fail-critical non-serious or fail-critical serious responses ($p = 0.606$ and $p = 0.256$, respectively). Furthermore, there were no significant differences when all fail-critical responses were combined for analysis ($p = 0.407$).

Static Evaluation

Four hundred thirty-six drivers completed the static evaluation. Each driver was asked to respond with one of four choices after viewing the scenario. *Yield, then go if an acceptable gap in the opposing traffic exists* was the correct response for all 12 scenarios. *Stop first, then go if a gap in opposing traffic exists* was also considered a correct response. Driver comprehension was again determined by the percentage of correct responses; however, an analysis of incorrect responses was completed. Similarly, the components of the PPLT signal displays and demographic variables were isolated to identify any effect on overall driver comprehension.

Correct responses ranged from 73 to 89 percent for each of the 12 PPLT signal displays. A statistically significant difference in driver comprehension was found considering each of the 12 PPLT signal displays, ($p = <0.001$). In particular, scenarios three (five-section cluster, with flashing yellow arrow permissive indication, and circular green through indication), five (five-section cluster, with circular green/flashing yellow arrow permissive indication, and circular green through indication), seven (four-section vertical, with flashing yellow arrow permissive indication, and circular green through indication) and 11 (five-section vertical, with flashing yellow arrow permissive indication, and circular green through indication) had significantly high percentages of correct responses. By comparison, displays two (five-section cluster, with circular green permissive indication, and circular red through indication) and 10 (five-section vertical, circular green permissive indication, and circular red through indication) had significantly low levels of correct responses.

PPLT signal displays with the circular green permissive indication had significantly lower correct responses than PPLT displays with either the flashing yellow arrow or circular green/flashing yellow arrow permissive indications. PPLT displays in the four-section vertical arrangement had a significantly higher percentage of correct responses than displays with either the five-section cluster arrangement or the five-section vertical arrangement ($p = 0.003$).

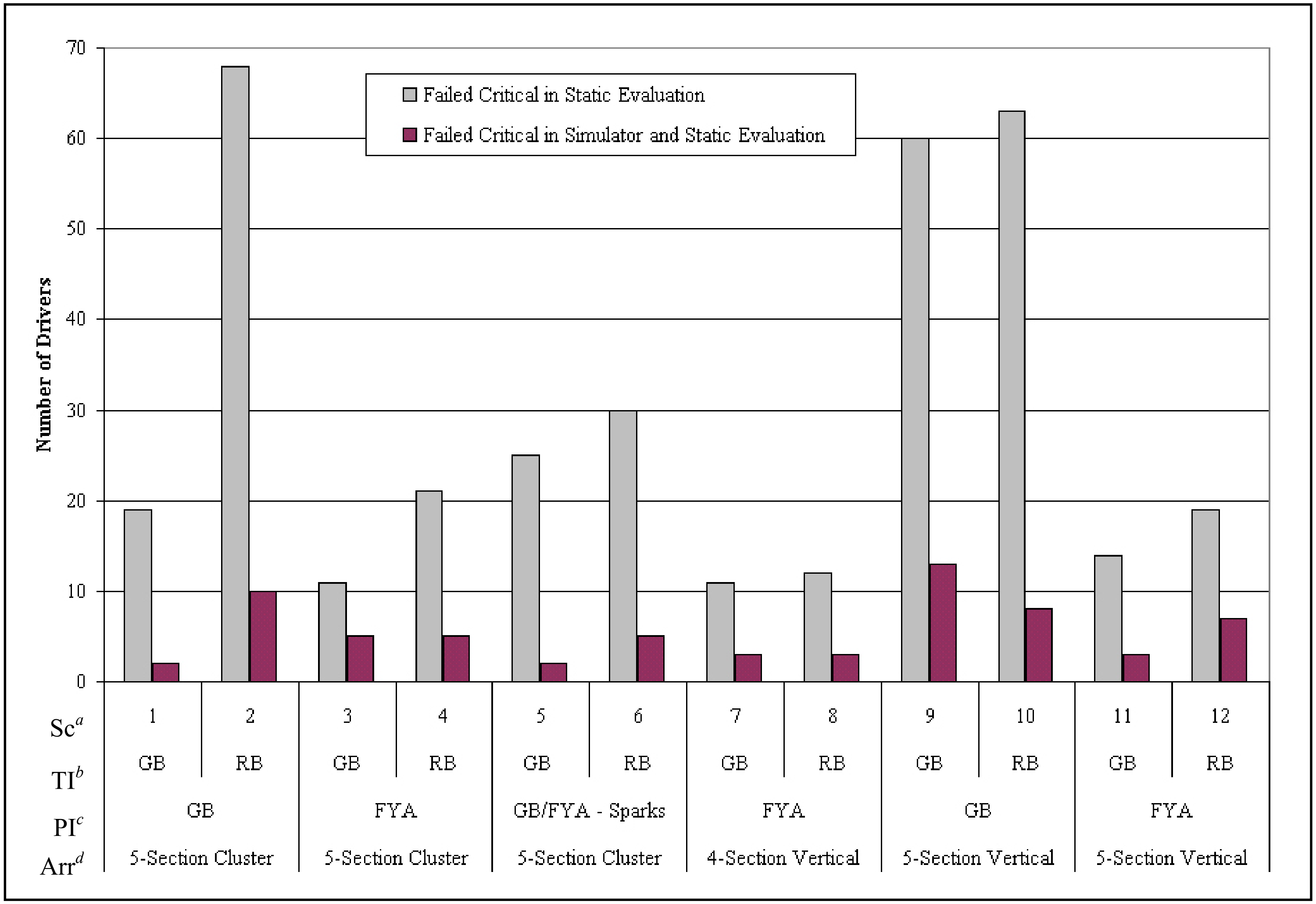
Location of the PPLT display was not statistically significant ($p = 0.170$). A significant difference ($p = <0.001$) was found between displays with the through movement circular green and circular red, with drivers responding correctly more frequently to displays with the circular green through movement.

Analysis of Incorrect Responses

An analysis of incorrect responses yielded statistically significant differences across the 12 PPLT signal displays ($p = <0.001$). A significantly higher number of fail-critical responses were generated from three scenarios, each including the circular green permissive indication. Specifically, scenarios two (five-section cluster arrangement, circular green permissive indication, and circular red through indication), nine (five section vertical, circular green permissive indication, and circular green through indication), and 10 (five section vertical, circular green permissive indication, and circular green through indication) were each associated with significantly more *go, you have the right-of-way responses.*

Comparison of Driving Simulator and Static Evaluation Results

There were 353 fail-critical responses in the static evaluation for which a direct comparison with the driver's response in the simulator was available. Of the 353 fail-critical responses from the static evaluation, drivers had responded correctly in the simulator environment 79 percent of the time. Only 19 percent of the 353 pairs resulted in fail-critical responses in both the simulator and static evaluation. Figure 4 presents the number of drivers with fail-critical responses for each of the 12 PPLT signal displays in the static evaluation, and the number of those drivers with fail-critical responses at the same display in the simulator.



[a] Scenario identification number
[b] Indication for adjacent through lanes (GB = circular green; RB = circular red)
[c] Left-turn permissive indication (GB = circular green; FYA = flashing yellow arrow)
[d] PPLT signal display arrangement

FIGURE 4. Comparison of Fail-Critical Responses in Simulator and Static Evaluation by Driver

SUMMARY OF FINDINGS

The findings of the driving simulator study showed that drivers responded correctly 91 percent of the time with no statistical difference between the 12 PPLT displays. No statistically significant difference in driver comprehension was found when the data were cross-analyzed by the PPLT display components including the permissive indication, arrangement, through indication, and location of the display. Additionally, there were no significant differences by the various PPLT display components in terms of the percentage of fail-critical responses. The lack of significant differences documented in this study is in itself a significant finding. The results indicate that the flashing yellow arrow is a viable alternative to the circular green permissive indication.

Overall, the permissive indication resulted in statistically significant differences of correct and fail-critical responses. Scenarios with the flashing yellow arrow permissive indication and the circular green/flashing yellow arrow simultaneous permissive indication had significantly more correct responses than displays with the circular green permissive indication. Additionally, displays with the circular green permissive indication were associated with significantly more fail-critical responses than displays with either the flashing yellow arrow or circular green/flashing yellow arrow permissive indications. PPLT scenarios with the four-section vertical arrangement had a significant amount of correct responses; however, only the flashing yellow arrow permissive indication was evaluated in this arrangement.

Displays with the circular red through indication resulted in a significantly lower percent correct response rate than displays with the circular green through indication. PPLT displays with the circular red through indication also resulted in significantly more fail-critical responses. This may be attributed to the fact that conflicting signal indications (red and green), even when not in the same signal display, are confusing to drivers. The location of the PPLT signal display did not result in statistically significant differences.

SUMMARY OF RELATED RESEARCH

As a result of the research described above, and as part of the NCHRP 3-54 project, field implementation of experimental flashing yellow arrow PPLT displays were completed in cooperation with six volunteer agencies located around the country. In September 2000, Montgomery County, Maryland became the first agency to implement the flashing yellow arrow permissive indication. Maryland's installation was subsequently followed by the City of Tucson, AZ; Jackson County, OR; the Oregon Department of Transportation; the City of Beaverton, OR; and Broward County, FL. Figure 5 illustrates the flashing yellow arrow permissive indication used in Woodburn, Oregon.

The research study and field implementation effort has identified several benefits of the flashing yellow permissive indication in a four-section vertical, all arrows, display including:

- Left-turn confusion is significantly reduced, especially related to shared signal heads;
- No supplemental sign with PPLT operation is required;
- All types of phasing can be operated by time of day (protected only, permissive only, protected/permissive, or permissive/protected);
- The display works at all signalized intersections;

- No louvers or precise head placements are required;
- The display can be mounted by pole, span wire, or median mount;
- The display can use a bi-modal lens in a three section display;
- The display can be used for right-turns;
- The "yellow trap" is eliminated when the flashing yellow arrow display is logically tied to the opposing through movement green indication.

FIGURE 5. Four-Section Vertical Flashing Yellow Arrow Permissive Indication

Based on the findings of all research tasks, the flashing yellow arrow indication was well understood in almost all deployment cases. Analysis of the research data suggest that the flashing yellow arrow indication is at least as safe as the circular green indication. Where deployed, the flashing yellow arrow indication was favored by almost all of the traffic engineers, field technicians, and citizens when compared to the traditional circular green PPLT indication. The flashing yellow arrow indication and display was found to have a high level of understanding, and a lower fail critical rate as compared to the circular green permissive indication. Hence, a significant safety improvement. The flashing yellow arrow display offers more versatile field application features (for example, it can be operated in a variety of operational modes by time-of-day, implemented on any signal mount, and any intersection configuration) as compared to the circular green indication.

Given these very favorable results, it is recommended that the flashing yellow arrow display be included in the MUTCD as an allowable alternative display to the circular green indication when used in PPLT control/operation. The four-section, all arrow display face should be the only display allowed, with the exception of a three-section display face with bi-modal lens (other potential variations including a five-section head are identified for consideration). Ideally, the flashing yellow arrow operation shall only be used in an exclusive signal arrangement. When used for left-turn treatments, the flashing yellow arrow shall be tied to the opposing through-green indication/display.

ACKNOWLEDGEMENTS

This research was conducted as a component of NCHRP 3-54(2) contracted by Kittelson & Associates, Inc., Portland, OR.

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Development of an In-House Automated Vehicle Location System

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ABSTRACT

Students, faculty and staff at the University of Illinois at Chicago (UIC) must move around a campus that is approximately two miles from east to west and one mile from north to south, and that continues to expand. A free shuttle bus service is a primary mode of transportation.

In order to enhance the user experience, and ultimately the service, a GPS-based monitoring system is being developed. Because of limited resources, staff in the UIC Facilities Department and the Urban Transportation Center devised a way to create a low-cost system built with "off-the shelf" hardware. The system's data is mapped on a public website so that users can 1) gain greater awareness of the bus routes and 2) determine the wait time until the next bus would arrive at their stop.

A secondary potential benefit of the system (after full implementation) is that it should allow for better operations management due to its ability to accurately locate buses. Currently, communications management with buses is achieved via two-way radios and supervisor oversight.

Implementation involved the installation of the hardware on each of the ten shuttle buses on the campus. The hardware on each bus included: a commercially available GPS locator, modem and two-way radio. An antenna was also installed on the highest building on campus (30 stories) to transmit the data. The data is transmitted at 15-second intervals over a publicly available radio frequency.

Key words: AVL—GPS—transportation planning.

INTRODUCTION

The 45,000 students, faculty and staff at the University of Illinois at Chicago (UIC) must move around a campus that is approximately two miles from east to west and one mile from north to south, and that continues to expand. The free shuttle bus service is a primary mode of transportation.

In order to enhance the user experience, and eventually the efficiency of service, a GPS-based monitoring system is under development. The data collected is mapped on a public website so that users can gain greater awareness of the bus routes and determine the wait time until the next bus will arrive at their stop. The website is under development to display the real-time location of the buses along the route. The program currently identifies the point on the route where the bus is, as well as the last point where it was detected, in order to determine its direction, which is shown as an arrow on the map. The project team is currently exploring ways to display buses on a route even if they change their scheduled run.

Future plans for the system include enhancements to the map for easier use, ADA compliance, installation of a second antenna for redundancy in data collection and ultimately map kiosks at strategic locations on campus.

SYSTEM DESCRIPTION

Hardware

In each bus a commercially available GPS locator (Garmin G-36 GPS) was installed, as well as a packet modem and a two-way radio receiver. The GPS hardware receives raw location data from the satellite, calculates the location, and sends the information out in bits. The modem on each bus receives the data in bits and converts it into tones. The two-way radio receiver on the bus transmits the tones to the radio receiver antenna, which is placed on the tallest building on campus (13 stories). The location information is then transmitted across campus via telephone wire to a receiving modem, which is connected to the dedicated server (IBM Netfinity 5000) at the Urban Transportation Center. There are sometimes gaps in the data due to a single antenna receiving data for a two-mile-wide area and therefore a second antenna is scheduled to be installed to provide for data redundancy.

FIGURE 1: Hardware Installed in the Buses.

The data is transmitted at 15-second intervals over a publicly available radio frequency. Figure 2 illustrates a representation of the GPS system.

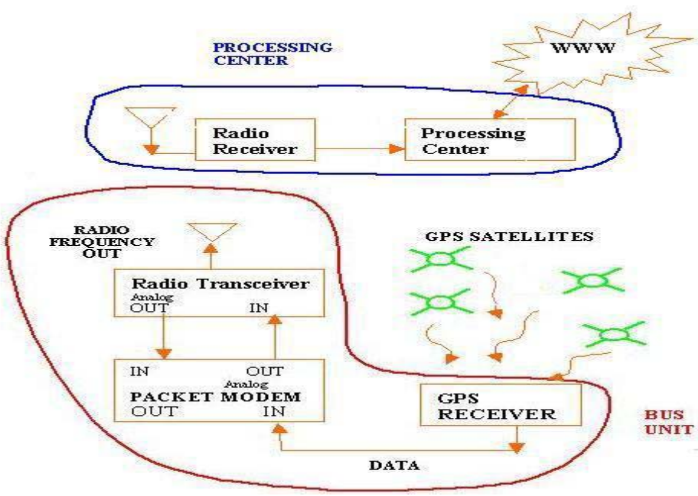

FIGURE 2: System Representation

Software

In order to have the bus locations appear properly on the map, the team drove the length of the bus route with a GPS receiver and marked the location of each bus stop and intersection. This GPS data series was then used to correspond to the longitude/latitude data in the map so that the bus route follows the correct path on the map.

The website was initially developed as a "jpeg" picture of the campus map on which the bus routes and the real-time location of the buses were displayed. The program identifies the point on the route where the bus is; each pixel has a direction associated with it, and its direction is shown as an arrow on the map. Because the jpeg file required so much memory to load onto each user's computer each time the data refreshed, on some computers the map flickered when it refreshed, a new approach to mapping was sought.

In a second iteration of programming the system, the map was rewritten in a Java program. Now only a small data file needs to be sent to update the map. Now it takes far less memory on users' computers and there is less likelihood of problems in the way the information is displayed. Roll-overs were added to the programmed map so that at each bus stop or building marked on the map the user can obtain exact information about the address.

Additional components to the map development have included usability for those who are disabled. Therefore, a high level of contrast is required on the map for color-blind users, as well as an option for the blind to call the dispatchers' office to obtain verbal information about bus location. Ultimately, the locations of the buses will also be provided via text so that software for the blind can read the text to the user.

Other considerations with respect to mapping were that there were multiple routes that needed to be displayed on the map: the daytime and nighttime/weekend shuttle. The team partnered with the bus dispatchers to develop a chart into which they could input each bus assignment and note the bus number and its specific route, which is communicated back to the master program. This ensures that the buses are coded to the proper route, as the routes are displayed on the map in different colors. Figure 3 shows the map of the campus and a representation of the online

The map has been in the beta testing phase since February and the response from students and faculty has been very positive. Suggestions have been made in terms of readability and user friendliness and those are being incorporated. The possibilities for this system in terms of improving productivity and quality of life for the students and faculty are very good, as the following calculations show.

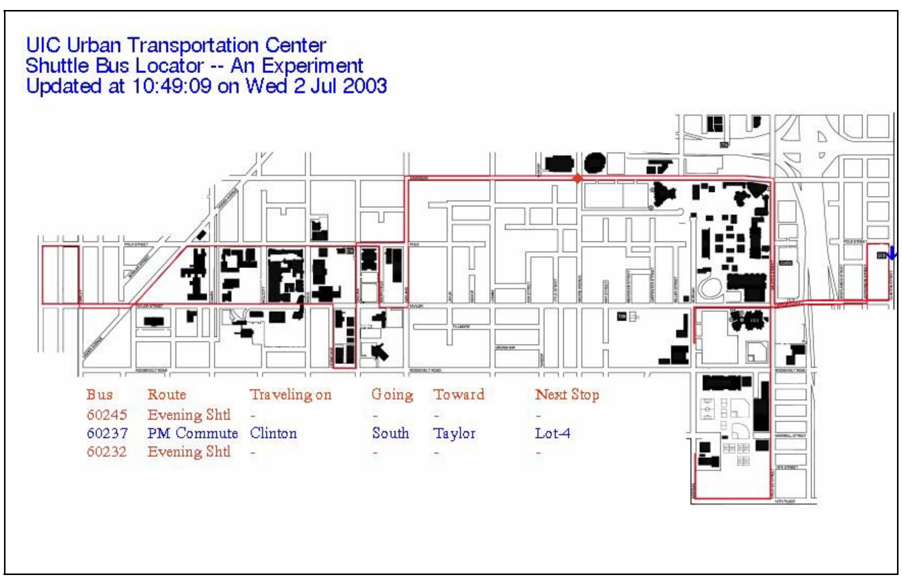

Figure 3: Online Map

SENSITIVITY ANALYSIS

The campus shuttle runs from 7:00 AM to midnight and is free for UIC students and faculty. This study only includes the shuttle bus service and not any other kinds of commuter transportation within the campus area. Based on data given by Facilities Management Administration and field data collection, estimations of riders' waiting time were carried out. The following section explains the methodology used for calculating the waiting time and also gives details on some of the assumptions used in the study.

Assumptions

For the purpose of some level of simplification, assumptions have been made as follows:

- All the buses start on time from their first stop (i.e. services building).
- The distance between bus stops is the same and drivers drive at the same speed.
- It takes 40 seconds for a bus to travel from one bus stop to another. It was found from field data collection that on average a passenger takes 1.1 seconds to board the bus.
- All the buses are the same new model buses and their door takes 10 seconds to open and close.
- The system transports the same number of riders per day.
- Buses do not wait at the college of medicine west for more time that the required to pick up users.
- People getting off the bus do not affect boarding patterns.

Methodology

After analyzing the data, the month of September 2002 was selected to represent the base condition. Three basic steps were necessary to analyze the data.

1. Using the number of passenger per hour per stop, find the number of passenger per minute per stop. This is done only for the main route.
2. Based on field measures, calculate boarding time for each passenger (1.1 sec), plus open/close door time (10 sec) for the buses used, and estimate a travel time between each stop (40 sec). Calculate an estimated arrival and departure time to each stop and therefore calculate the headway at each stop.
3. From 2 and 3, knowing the number of arrivals per minute to each stop and knowing the headways at any given stop, calculate waiting time at any given stop for any given route. Therefore, total waiting time for the system is known.

Based on the above methodology, it was found that the total waiting time for the system for the month of September 2002 was 459 hours. The following figure shows the sensitivity analysis of the potential? waiting time reduction given the percentage of riders using the website for bus location.

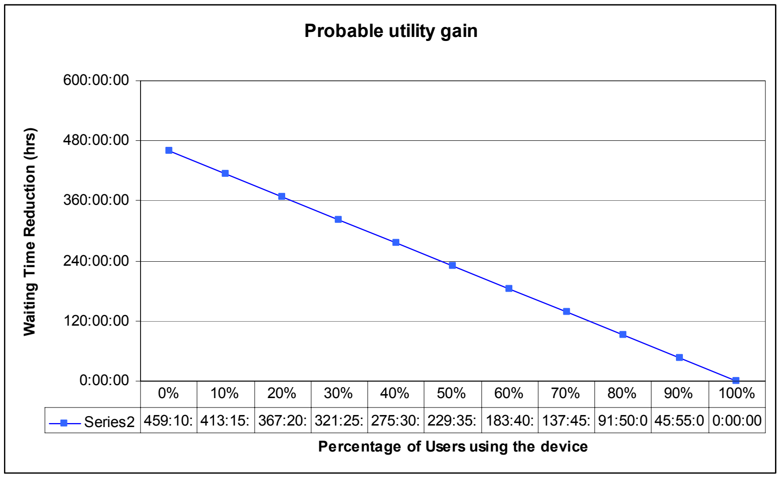

FIGURE 4: Sensitivity Analysis from the GPS system

CONCLUSIONS

For many commercial bus systems, the cost of GPS systems ranges from $5,000 to $30,000 per bus. This system designed in-house is dramatically less, at only $380 per bus. Despite the lower cost, the system is effective and it provides significant benefit to the users. The system will improve the mobility and accessibility of UIC faculty, staff and students.

Additionally, since the dispatcher and the supervisor have access to all the real-time data as well, they will know the exact location of the buses and their staff. It will make easier for the dispatchers to re-route buses, or expand the system in the future. It will also be easier to handle the deployment of spare buses either for high demand or an on-route bus breakdown. Most of all, this system provides an easier way to control delays, therefore on-time performance and consumer satisfaction should be improved.

ACKNOWLEDGMENTS

A project as complex as the design and implementation of an AVL involves many people at so many different levels. In the particular case of our project, we have had the involvement of faculty, students and staff from different college units and departments of the University of Illinois at Chicago.

Specifically, we would like to acknowledge the hard work and time spent of two individuals. Jim Limber, an engineer from the Physical Plant Administration unit at the University of Illinois at Chicago, is the mastermind behind the idea and George Yanos, a network specialist in charge of the hardware and software of the Urban Transportation Center at the University of Illinois at Chicago, is the web designer and programmer.

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Automated Methods for Collecting National Transit Database (NTD) Data

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ABSTRACT

In recent years, there has been a growing awareness of the need to use National Transit Database (NTD) data for operational purposes. Since the NTD contains the only standardized collection process of all urban transit providers in the nation, it has become an important transit evaluation tool. In many cases, however, there have been concerns about the validity of the NTD information that is distributed, even after final Federal Transit Administration (FTA) validation. Transit agencies are having difficulties in collecting some of the NTD data. Hence, they are requesting help with collecting data from the correct sources, ease of obtaining the NTD data, determining operational procedure guidelines to collect data more efficiently, and gathering data from their contractors. As part of this study, interviews were conducted with transit agencies to understand existing methods of collecting data and current issues and problems faced in doing so. In addition, research was conducted on available automated methods which can increase ease of data collection and accuracy.

Key Words: data—NTD—performance—public transit

INTRODUCTION

As part of its National Center for Transit Research (NCTR) Program, the Center for Urban Transportation Research (CUTR) has been working with the Florida Department of Transportation (FDOT) to conduct research that will assist transit agencies in their collection of the non-financial National Transit Database (NTD) information.

In recent years, there has been a growing awareness of the need to use NTD data for operational purposes. Since the NTD contains the only standardized collection process of all urban transit providers in the nation, it has become an important transit evaluation tool. In many cases, however, there have been concerns about the validity of the NTD information that is distributed, even after final Federal Transit Administration (FTA) validation. Transit agencies are having difficulties in collecting some of the NTD data. Hence, they are requesting help with collecting data from the correct sources, ease of obtaining the NTD data, determining operational procedure guidelines to collect data more efficiently, and gathering data from their contractors. As part of this study, CUTR conducted interviews with transit agencies to understand existing methods of collecting data and current issues/problems faced in doing so. In addition, CUTR conducted research on available automated methods, which would increase ease of data collection and accuracy. Findings of this research are included in this paper.

EXISTING AUTOMATED TOOLS

This paper summarizes existing automated methods (software/hardware) that are available to transit agencies in collecting statistics for the NTD. Most methods included herein are generally applicable to fixed-route and paratransit services. If an application is more appropriate for one type of service over another, it is indicated.

Several considerations in selecting appropriate applications include system size, budget availability, and characteristics of service area (large population base with high-growth versus small population base with limited growth, size of the service area, etc.). In addition, it should be noted that although these programs can be used to facilitate collection of the NTD data, this capability may not be the primary factor in deciding whether to acquire any given software or hardware. At times, the ability to collect the NTD data more efficiently is simply an ancillary benefit to the agency from implementing a certain automated method.

Information provided on automated methods includes an overall description of the application and its capabilities, strengths and weaknesses, and overall price ranges. It should be noted that pricing of these products ranges a great deal depending on the agency size and existing technology. In most cases, the price ranges included in this paper were obtained directly from a sample of suppliers and are intended to give a general idea on pricing. The actual cost of these products for a given agency tends to vary significantly depending on the size of the system, existing technology and equipment, the complexity and sophistication of the software and hardware, and other variables.

NTD REPORTING – BASIC INFORMATION MODULE

In the NTD reporting system, the Basic Information Module requires information on transit agency identification (form B-10), agency contacts (form B-20), and any contractual relationships (form B-30). Most of the information required in this module is standard (name, address, contacts, etc.) that tends to remain relatively stable over time. However, information on service area demographics such as population as well as statistics that need to be obtained from contractors vary on an annual basis.

To determine service area population, which is required in this module, agencies could use geographic information systems (GIS). These applications can also be used in determining directional route miles and will add mapping capabilities to other automated methods used to track inventory and estimate passenger miles, vehicle miles, and other information. GIS integrate data and spatial information to allow for advanced analysis. GIS data are stored in layers of information that identify trends and patterns not recognizable from tabular data. By adding a geo-spatial element to data, users are able to recognize patterns in the data like proximity, clustering and adjacency of data units. More advanced uses of the geo-spatial data would include calculating proportional area values, such as the service area of a route. By capturing the most fundamental parts of a transit system in GIS, like the routes and stops, an agency will establish the foundation for a robust GIS system. This foundation will allow the transit agency to automate many of its reporting requirements. Some examples include the directional route miles, one-quarter mile and three-quarter mile service area calculations, ridership by stop and route, bus stop amenities, and determining the level of Title VI and Americans with Disabilities Act (ADA) compliance.

Technology. GIS software has advanced significantly and nearly all desktop software platforms offer the same basic functions. A typical GIS application has general map production capabilities, with printing and map exporting functions, and thematic mapping (i.e., mapping data ranges for census demographics or ridership levels or identifying amenities of a bus stop inventory). All GIS platforms are able to import and read many data types. Data currently stored in Microsoft (MS) Access, MS Excel, Lotus, Paradox, and dBASE can be integrated with GIS easily. Additionally, GIS can export the data into other software platforms for integration into reports or presentations. A transit agency considering a GIS application should ensure that the application it purchases can easily read or import the file type in which its data are stored.

Despite of all the advances and similarities among GIS software, there are differences that should be considered. Each GIS platform utilizes its own native file format and the differences between each type should be considered prior to purchasing a GIS application. Certain GIS file types have advantages over others when it comes to transportation GIS. The GIS should easily utilize topology, an important feature for transportation GIS. Topology is the expression of the relationship between and within spatial data. For example, while GIS express the location of a feature, such as a road, topology expresses the connection of the road to the overall road network. Topology illustrates the adjacency and connectivity of GIS features. Some file formats have built in topology while others need a great deal of effort to incorporate it. Topology is important for modeling functions, such as shortest path calculations. With a complicated system to implement topology, distance calculations as well as trip planning can become quite cumbersome.

Many of the GIS software companies offer a suite of products, with a tiered system for different degrees of functionality. Agencies should consider the expansion abilities of the software for future uses. Additionally, many businesses form partnerships with GIS companies to make ‘add-ons’ to increase the functionality and simplify tasks. Larger GIS companies have more partners, which may weigh in a transit agency’s choice of GIS platforms.

While there are great similarities in many of the GIS platforms available, transit agencies considering implementation of the GIS should consider what other area agencies and municipalities are using. Since much of the beginning efforts of implementing GIS involve establishing the base data, capitalizing on data already created by other agencies is key to a quick start-up. Consequently, to ensure a quick start-up a transit agency may want to consider a GIS platform that is compatible with other area agencies' (i.e., the Metropolitan Planning Organization (MPO), regional planning council, and property appraisers) GIS platforms.

As mentioned previously, for NTD reporting purposes, GIS provide transit agencies with an automated tool to estimate various statistics such as the service area population, directional route miles, etc. In addition, it could enhance the capabilities of some of the other tools such as the automated vehicle locators (AVL), and automatic passenger counters (APC), etc.

One of the constraints of GIS is the necessary training and skill level. If transit planners are not familiar with GIS, an extensive training may be necessary.

Software integration is another concern transit agencies should consider in their purchase of a GIS platform. The GIS software platform should easily integrate with scheduling, APC and AVL systems, as well as database, spreadsheet, and word processing software currently utilized at the agency. It is important to research which file formats communicate easily with the desired AVL, APC, and scheduling software applications. By ensuring that the selected GIS application easily integrates with the existing transit applications, the implementation of GIS will be less complicated, allowing for quick deployment and opportunity to expand the use of the GIS.

There are a wide range of prices and functionality for desktop GIS, with prices ranging from $500 to $10,000. Agencies looking to begin implementing GIS should be able to do so for under $2,000.

NTD REPORTING – TRANSIT AGENCY SERVICE MODULE

The Transit Agency Service Module contains one form per mode which is used to report data on the transit service supplied by the agency and the transit service consumed by passengers. In providing information for the service module, the following automated methods are used.

Technology for Collecting Vehicle Information

Vehicle information required as part of this module includes the number of vehicles operated and available during maximum revenue service, actual vehicle hours, actual vehicle miles, actual vehicle revenue hours, actual vehicle revenue miles, and total scheduled revenue miles. The following automated tools are available for collecting these data.

Automatic Vehicle Location (AVL) Systems provide real-time vehicle location data and can be integrated to the Computer-Aided Dispatch (CAD) and other systems. At a minimum, each AVL deployment includes a specific location technology and a method of transmitting the location data from the vehicle to a central dispatch center. Many of the more recent transit applications of AVL systems have integrated the automated vehicle location component with other systems (such as communications, GIS, analysis software, dispatch/control systems, and scheduling software) to expand the use of AVL for more efficient fleet operations, on-time performance, schedule adherence, route/service data collection, security, and traveler information services. For NTD reporting purposes, AVL data can be used to track vehicle inventory and mileage (total and revenue miles) and provide information on directional route miles.

There are three primary methods of tracking vehicles: Signpost Technique, LORAN C

Technology, and Global Positioning System (GPS). Signposts determine position via a fixed installation of electronic beacons located at various bus stops or other points on the bus routes. The signpost devices constantly emit a low-powered signal (beacon) along with a unique identification, both of which can be detected by the vehicle's transmitter or receiver. When a bus passes a signpost, vehicle data and location are instantly transmitted back to a central location. This system is also known as the Fixed Beacon system. The distance traveled between signposts can be approximated because the signpost devices also monitor electric pulses emitted by the vehicle's odometer. The advantages of this type are its low cost and considerable experience. With this method, vehicles can be tracked to within an accuracy of 500 meters (1,640 feet or 0.3 mile). This type is unsuitable for demand response vehicles and more suitable for fixed route vehicles. Signpost AVL technology has given way to GPS technology.

LORAN C technology is land-based and consists of radio transmissions relayed through land connections. This provides two-way communication. Each station transmits pulses of timed signals, and a receiver mounted on the vehicle can calculate distance traveled by comparing the times it receives different signals from the different origins. Its main advantage is its flexibility, where any vehicle equipped with proper receiver can be tracked no matter what its route is. However, several sources cause signal interference, including power lines and substations, tall buildings, and fluorescent lights within the vehicles. GPS is largely replacing LORAN C.

The current state-of-the-art AVL systems for buses use GPS technology that locates the bus via satellite. GPS is a worldwide radio-navigation system consisting of 24 satellites and their ground stations. The satellites are in a geo-stationary orbit and are always looking at the same place on earth. One of the benefits of GPS is its accuracy in reporting time. Some AVL systems also use Dead Reckoning (DR) to ensure the best possible location information. DR uses sensors to measure speed, distance, and direction. It uses these inputs to calculate current location from the last known position. Although either GPS or DR will provide accurate location data, they work especially well together. Both systems have their limitations and the other system compensates for those. For example, GPS sometimes provides inaccurate information around tall buildings and cannot read under bridges or in tunnels. DR is not affected by these situations. However, DR needs accurate positional updates because its accuracy falls off as the distance between known positions increases. GPS provides the accurate positional updates required by DR.

Suppliers interviewed as part of this research stated that in most cases including only the GPS technology is sufficient and that the DR technology is only needed in a downtown environment with high density. DR tends to increase the cost significantly; therefore, it is important to evaluate the level of its necessity in relation to the additional cost.

Data from AVLs are usually displayed on a map. They also come in ASCII files and can be exported to various spreadsheet and database programs (such as Excel, Access, etc.). At times, these data are automatically exported to the agency's scheduling software and can be viewed in formats that software supports. The initial implementation of an AVL system tends to take about one to six months depending on the size of the agency, area, etc.

As mentioned previously, AVLs provide agencies with real-time vehicle location data and enable them to respond better to emergency situations, improve on-time performance and scheduling, and reduce fleet size (especially in the case of paratransit service since the agency has a better access to vehicle locations to fulfill requested trips), etc.

Some of AVL's limitations relate to the extent of the wireless coverage area (when using cellular technology) and quality of map data. In terms of wireless coverage area, the initial testing could be helpful in determining which areas need additional coverage. In addition, most suppliers provide an evaluation of the service area, which addresses this issue, among others. Some agencies experienced problems when the map data were not accurate (e.g., addresses were incorrect or data set was incomplete, etc.). Finally, it is important that the relevant transit agency personnel be trained appropriately on the usage of hardware and software. Suppliers interviewed for this project stated that training on hardware is relatively easy; however, if an agency does not have a strong information technology (IT) department, training on the software could be a more extensive process.

The cost of hardware ranges from $400 to $3,000 per vehicle while software cost ranges from $2,000 to $30,000 depending on capabilities. Additional costs may include those related to the infrastructure, supplier service fees, recurring fees for the wireless network provider, etc.

Mobile Data Terminals (MDTs) display short text messages, replacing the voice radio communication between the driver and dispatcher except in emergencies or other exceptional cases. They can automatically send vehicle location, passenger counts, engine performance, mileage, and other information. Some information such as passenger boardings and alightings may be sent when the passengers use their smart cards as they enter or depart the vehicle or when the driver pushes function keys on the MDT. The driver can use other function keys to send pre-recorded digital messages regarding vehicle and passenger status or in response to questions or prompts displayed on the MDT screen. As a result, MDTs can virtually replace note-taking and written manifests and becomes an entry point for data to perform system-wide passenger counts and vehicle performance analysis.

Implementation of MDTs typically takes approximately three to six months. The necessary training is included in this time frame and takes about four to eight hours. MDTs are used in conjunction with dispatch/scheduling software and data are reported in the format used by these software.

MDTs can be an important tool in the full-automation of tracking passenger counts, vehicle mileage, and other information. Because the figures received will be automated, errors due to manual recording, data entry, etc. will be eliminated. The limitations of MDTs relate to the wireless coverage area. If a vehicle moves outside of the coverage area, related statistics will be lost. However, appropriate testing prior at the beginning of the installation process can minimize this risk. In addition, it is important to gain the support of the operators, who need to keep the equipment in good shape and report any problems.

Cost of the hardware ranges from $1,000 to $4,500 depending on the availability of AVL, modems, etc. Cost to interface with the scheduling software ranges from $15,000 to $35,000.

Technology for Collecting Passenger Information

Passenger information required by NTD includes number of riders and passenger miles. Some of the automated methods that can be used toward these include the following.

Electronic registering fareboxes (ERF) are typically used to count passengers in buses as they board. ERFs allow agencies to collect ridership data in a greater detail (by route, trip, fare categories, etc.). Of 24 agencies that responded to the survey used in this research, 13 use ERFs in counting passengers while the remaining agencies keep manual records. Primary manufacturers of the fareboxes used by Florida agencies are GFI and Cubic, with the majority using GFI. Agencies can retrieve the data in their garages without a physical connection or by using dynamic or periodic remote retrieval of farebox data.

Some of the advantages of using electronic fareboxes as cited by responding agencies included increased ability to collect fares, greater accuracy of data in comparison to manual collection, and ability to group information in various forms. Disadvantages cited were the necessity of manual manipulation/entry of results, at times inaccurate data due to operator errors, difficulties in identifying the source of the errors during validation, and not offering the capability to measure miles (vehicle miles, passenger miles, etc.). An additional disadvantage cited in Transit Cooperative Research Program (TCRP) Synthesis 29, titled "Passenger Counting Technologies and Procedures," (1) related to mechanical/equipment problems such as currency jams, aging coin mechanisms, difficulty in reading swipe cards, overloaded vaults, and reliability of time/date stamp that records when trips were made.

According to TCRP Synthesis 29 (1), as in the case of any new technology, an initial "debugging" period is experienced by agencies that start using ERFs. This period averages almost 18 months, with a range of six weeks to six years.

Regarding pricing, one supplier provided a range of $10,000 to $20,000 per farebox, including software and hardware. Another vendor stated that the hardware cost would range from $11,000 to $13,000 per box while software cost would range from $34,000 to $54,000.

Automatic Passenger Counters (APC) are currently used by two Florida agencies and at least one other agency in Florida is in the planning stages of incorporating this technology. APCs count passengers as they board and alight a bus and record times at each stop, which allows agencies to calculate passenger miles without using ride-checkers. APCs also provide information on directional route miles required for NTD. Because the full APC systems include an AVL component, they can provide all of data provided by AVLs in addition to other information.

Two different technologies used by APCs include infrared sensors and treadle mats. Infrared sensors are typically mounted near the bus doors. There are two types of infrared sensor technologies: active and passive. Active infrared sensors need the reflection of objects passing through the door and dark colors do not reflect well. In addition, active infrared systems need periodic re-calibration. Passive infrared sensors can only detect "change in heat," which means if there is no movement, there cannot be any detection. Some manufacturers provide a combination of these two infrared technologies to provide more accuracy.

Treadle mats are mounted to the vehicle steps and contain switches that close as passengers step on the mat. The transitions and times between closing and opening switches determine passenger flows. According to the TCRP Synthesis 29 (1), in certain climates treadle mats can be difficult to maintain, but most observers report no difference in accuracy between the two technologies.

To correlate the data to specific bus stops, AVL systems can be used. According to the experience of agencies interviewed as part of the TCRP Synthesis 29 (1), availability of an AVL system reduces the cost of APC units, which no longer need to handle data storage, data transmission, and odometer interface.

Finally, agencies retrieve the data either at their garages without a physical connection, by establishing a direct downlink at the garage, or by transmitting data remotely via radio while the bus is on the street.

Both Florida agencies that use APCs are highly pleased with results and feel that this technology benefits their agencies in terms of cost savings (elimination of checkers and data entry), increased accuracy of data, and additional usage of data obtained (in redesigning the system, determining routes that should be eliminated, and reallocating the resources more efficiently, etc.). One agency is now requiring new buses to be equipped with APCs so that eventually all the buses in the system will have APC units.

Overall, agencies have been satisfied with APCs (*1*). Some of the problems experienced related to the software, including the necessity to develop/upgrade analytical programs, time-consuming data processing, and consistent maintenance requirement of databases containing schedule and bus stop information. Some of the hardware problems reported included equipment failures, maintenance problems, and the durability of APC units on the buses. For signpost-based systems, signpost detection and difficulty in coordinating signposts with bus route assignments were mentioned. Finally, it was important that APCs be accepted by all personnel of the agency and to have active management of the system.

Similar to other agencies throughout the country, Florida agencies equip only a certain number of their buses with APCs in order to minimize the cost. They rotate these buses appropriately to comply with the sampling requirements of FTA for NTD reporting.

Agencies surveyed for the TCRP Synthesis 29 (*1*) reported that the break-in or debugging period for APCs averages 17 months, very similar to the electronic fareboxes.

If the agency already has an AVL system, the cost of adding APCs (as a sub-system) ranges from $900 to $2,000 per bus. The price to implement a full-system ranges from $6,000 to $8,000 per bus. It is also possible to lease APCs, which may be feasible if they are used for a limited purpose. The lease price depends on the number of units leased as well as the length of the lease.

Hand-Held Units can be used by checkers to record data for sampling in estimating ridership and passenger miles. This technology allows agencies to eliminate data entry and therefore tends to increase accuracy. For this purpose, it is possible to use generic computer equipment, such as laptops, or personal data assistants (PDAs), which are small hand-held computers that electronically reproduce driver manifests, among serving other functions. These units tend to be directly connected to a host computer to upload and download data. Required information for each route is stored on a host computer and is downloaded to hand-held units when needed. Ridership data is uploaded when the check is completed. Changes in schedules or stop lists are made in a single location (on the host computer).

For paratransit services, drivers can pick up PDAs that are loaded with a day's worth of trip data in the morning and return them to the office at the end of the day. Each day's data are downloaded and the PDA is recharged. It accomplishes the combined job of a scheduling and dispatching software of MDTs but with less capital cost and more driver effort.

Because the data entry is eliminated, hand-held units tend to increase the accuracy of the data. At the same time, some agencies mentioned that they lost the valuable written comments they used to receive from checkers/drivers, which were instrumental in understanding the reasons for data irregularities. One Florida agency that started using hand-held units recently reported that, although the agency still has the same number of checkers (and therefore no obvious cost savings), they are able to complete twice as many surveys (increased accuracy of annual estimates reported in the NTD). Staff at this agency also felt that the accuracy of the data has improved since the error-prone data entry process was eliminated. One problem the agency faced was having to download and upload the data only in the host computer, which required checkers to coordinate with each other and transport all of the data to a central office.

In comparison to MDTs, PDAs tend to be more affordable. One of the concerns about PDA units relates to life of the hardware (i.e., may not be very sturdy), which is increased with industrial grade units. Another concern is that these units could be stolen or lost relatively easily due to their multiple functions and small size.

Depending on memory and other features, the price range for these products is $200 to $1,000.

Smart Cards contain information that can be queried or supplemented by a card reader. These cards have an integrated circuit chip, a central processing unit, and an operating system like a small computer. The reader activates the card to identify the passenger, obtain stop data, determine the source and amount of fee, or other information. Smart Cards can be integrated with the agencies' accounting systems, scheduling software, and AVL systems.

Depending on the complexity of the project, the implementation of Smart Cards can require six to 24 months, which includes a training period of up to two months. The use of Smart Cards by transit agencies is still limited and none of the Florida agencies interviewed reported using Smart Card technology for NTD purposes.

According to TCRP Synthesis 29 (*1*), agencies that have tested Smart Cards are positive about their experiences and the potential of this technology. According to the same source, problems identified in demonstration projects included a lack of integration with the farebox and other on-board equipment and software, software bugs, data retrieval (particularly the need for training), and hardware problems. The time commitment for training personnel in the implementation of Smart Cards, retrieving and formatting data, etc., can be extensive. The agency's maintenance department's support for installation and maintenance of additional equipment strongly influences the transition to Smart Cards. Operators also need to be informed of the value of such technology. Agencies without ERFs would require operator intervention to record cash fares in a Smart Card system, and operators tend to be the ones who must deal with passenger complaints regarding Smart Card malfunctions.

It should be noted that, in terms of collecting passenger information for NTD, this technology may be not yet be effective for fixed-route services. In the case of fixed-route service, it is important to achieve a high-level of penetration among transit users to provide any meaningful ridership data beyond the number of people who are using Smart Cards.

Suppliers were unable to provide a unit price and explained that the cost of implementing Smart Cards depends on many factors including type of fare collected, existing back-office operations, level of usage (how extensive, how many modes, etc.), and other variables. According to one supplier, implementing a complete Smart Card system in a large urban area can range from $20 million to $100 million.

Technology for Collecting Data Related to Periods of Service and Days of Operation

Agencies use either printed schedules or their scheduling programs in order to report information on periods of service and days of operation. These programs are also used in conjunction with automated methods used to collect passenger and vehicle information described above.

Depending on the level of automation, scheduling and validation software have capabilities to provide integrated databases for routes, riders, vehicles, and garages, etc. These programs tend to interface with APC and AVL systems, MDTs, PDAs, and advanced fare collection technology, which eliminates the need of data entry and/or achieves a higher level of automated data integration. In the case of MDTs and PDAs, the scheduling software sends/uploads the scheduled trips prior to the beginning of the service. At the end of the service, output from these units are downloaded to the software for data manipulation. Some of these programs have sampling capabilities to be used in the estimation of passenger miles and ridership statistics.

Implementation of this type of software takes about five to 12 months while the necessary training ranges from 1 to 20 days, depending on the complexity of the program.

The cost of scheduling software varies significantly depending on the complexity and the level of automation. Based on information provided in a 1999 report prepared by North Carolina State University, “Small Urban and Rural Advanced Public Transportation Systems,” (*2*) the software cost for fully-automated scheduling programs ranges from $4,000 to $2 million. Discussions with transit agencies also indicated a large range. Agencies who recently acquired scheduling software stated that a part of the reason for such a large range relates to how suppliers bundle their products. Scheduling software is sold in modules with each module providing different services. If an agency can limit its purchase only to the needed module, the cost is relatively lower. However, at times, the agency’s existing technology may not be compatible with the new program and some of the other software/hardware may need to be upgraded as well, which increases the cost.

NTD REPORTING – RESOURCE MODULE

The Resource Module contains three forms addressing employees (form R-10), maintenance performance (form R-20), and energy consumption (form R-30). Employee information required includes the number of transit agency employees (person count) and their total work hours by labor category and by mode. These statistics are typically tracked by accounting/payroll systems, which fall under the automated methods used for financial information and are not included in this study. Statistics required in the maintenance area include revenue vehicle system failures and hours spent on inspection and maintenance by the transit agency’s service personnel for directly operated modes. In terms of energy consumption, agencies are requested to report vehicle fuel consumption for directly operated service only.

Fleet Management Programs provide transit agencies with an automated tool to track vehicle inventory, fuel purchases, vehicle mileage, and maintenance information, among other capabilities. They contain vehicle fleet information with each vehicle’s history, current status including mileage, and scheduled maintenance. Agencies can use this list to provide some of the statistics required in the Asset Module as well as those required in the Resource Module. These programs track fuels and other fluids used by vehicles by individual vehicles. With electronic fuel interface (EFI) capabilities, information from automated fueling systems can be transferred to fleet management programs without requiring manual data entry.

Fleet management programs document planned and unplanned maintenance work completed on each vehicle, which provides the agency with a list of revenue vehicle system failures. In addition, some of the programs allow agencies to specify certain checks and balances and give warnings for figures that appear abnormal. Fleet management programs provide data in ASCII files, which can be exported to spreadsheets, databases, or other software. In addition, these programs tend to offer report generation option, which allows users to generate a variety of reports.

These applications significantly reduce or eliminate data entry. Although accuracy of the data increases, it is still important to validate the data. Operating a program with built-in checks and balances makes it easier to do so.

Depending on the inventory size and complexity of the program, the cost for these programs can range from $2,000 to $200,000.

Fuel Dispensing and Management Applications are necessary in the full automation of data collection on fuel purchases. Pumps equipped by these systems allow access through several methods including simpler methods such as keying in the vehicle number as well as more automated methods such as magnetic stripe cards, smart read/write keys with embedded chips, and intelligent vehicle modules which allow direct communication to the vehicle without driver interaction. At times, these applications also provide information on vehicle number and life-to-date mileage each time a vehicle is serviced.

Output from these applications can be imported to fleet management programs or are directly provided in several formats. Depending on the suppliers, these formats include ASCII files (comma-delimited or comma-separated), spreadsheets (Excel, Lotus, etc), databases (Access, FoxPro, dBASE), and HTML.
Depending on the complexity of the system, installation/implementation can range from four days to three weeks. Required training takes one to three days.

Staff at agencies that use automated fluid dispensing methods believe that the information they obtain is quite accurate. Florida agencies that are currently tracking energy consumption manually reported difficulties in obtaining accurate figures and reading gas station attendants' handwriting, etc. They felt that automated methods would simplify the process and increase accuracy.

Depending on the size and complexity of the system, the cost for these products can range from $15,000 to $100,000.

NTD REPORTING – ASSET MODULE

The Asset Module includes forms addressing stations and maintenance facilities (form A-10), transit way mileage (form A-20), and revenue vehicle inventory (form A-30). In many cases, agencies have a small number of stations and maintenance facilities and can complete these NTD forms by memory. Revenue vehicle inventory can be tracked by using AVLs (discussed previously) or fleet management programs. In addition, it is possible to input and track station and maintenance facility information in fleet management programs. These programs were discussed in greater detail earlier in this paper.

NTD REPORTING – SAFETY AND SECURITY MODULE

The Safety and Security Module requires the completion of several forms. Forms requiring general information include the Incident Mode Service Form (S&S-10), Ridership Activity Form

(S&S-20), and Security Configuration Form (S&S-30). Those dealing with incident reporting are the Major Incident Reporting Form (S&S-40), and the Non-Major Incident Reporting Form (S&S-50).

Figures for the Ridership Activity form can be obtained by using automated methods to collect ridership information, as discussed previously. The remaining forms are typically completed based on paper reports prepared or by using information tracked in a spreadsheet. Agencies tend to not need automated methods specific to this Module.

OTHER/GENERAL USE

In addition to above mentioned programs/automated methods, most agencies use spreadsheets like Excel, Lotus or Quattro Pro and some agencies use database applications such as Access and FoxPro. Florida agencies use these applications either by themselves or in conjunction with other software to track a variety of variables including vehicle inventory, vehicle/revenue hours and miles, annual passenger data, vehicle system failures, fuel purchases, employee hours, directional route miles, safety/security information, and other statistics. In addition, some of the agencies provide their contractors spreadsheets that have hidden formulas and checks and balances built in for contractors to enter their data. This facilitates tracking the necessary statistics for contractors and gives them an indication of possible errors.

Although these applications are not as sophisticated as some of the others described previously, they are readily available, inexpensive, easy to use, and do not require extensive training. While they facilitate calculations and keep track of data, because they are not fully automated, agencies need to enter data unless they are using databases in conjunction with other software. At times, they may be customized to fit an agency's specific needs. Usually a local programmer performs the work based on the needs.

Prices for spreadsheets and databases range from $100 to $600 (without customization). Some products may come "bundled" with new computers. In addition, if other departments of the City/County governments already own a copy, getting the right to install an additional copy may be quite inexpensive.

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Evaluation of First-Year Florida MPO Transit Capacity and Quality of Service Reports

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ABSTRACT

As an application of the transit quality of service framework presented in the first edition of the *Transit Capacity and Quality of Service Manual* (TCQSM), the Florida Department of Transportation required all Metropolitan Planning Organizations (MPOs) in the state where fixed-route transit service operates to analyze those services based on the six measures identified in the TCQSM: service frequency, hours of service, service coverage, passenger loading, reliability (on-time performance/headway adherence), and transit versus auto travel time. This first-year evaluation compiles the analyses provided by the participating MPOs and provides an assessment of the aggregate performance of the transit systems. A larger part of the study focused on the examination of the actual process used by the MPOs and transit systems to evaluate their services. Recommended changes to improve and refine the process for future years are presented based on the first-time experiences of the MPOs. This evaluation serves as a model for other areas in the country interested in applying the customer-oriented assessment of transit based on the TCQSM.

Key words: performance—public transit—quality of service—transit capacity

INTRODUCTION

The Florida Department of Transportation (FDOT) is interested in the application of the new transit quality of service framework presented in the first edition of the *Transit Capacity and Quality of Service Manual* (TCQSM). This framework is seen as a tool to augment systematic evaluation of transit systems performed by FDOT. The goal is to provide a benchmark evaluation of transit systems within a specific time period such that the performance of the systems can be assessed from the transit users' point of view. FDOT required that the Florida Metropolitan Planning Organizations (MPOs) where fixed-route transit service operates coordinate an effort to evaluate those services within their regions with respect to the six measures identified in the TCQSM.

This study involved the collection, compilation, and analysis of these reports as gathered from the MPOs. The reports were examined to make preliminary assessments of the overall performance of the transit systems in the state in terms of these new measures. While individual system results are not presented, the six transit capacity and quality of service (TQOS) measures are presented in aggregate form for the state as a whole.

More than a general analysis and presentation of the results is contained herein; this study also sought to evaluate the *process* undertaken by the transit systems and MPOs in completing this effort. It is understood that the results from this first-time endeavor might not be as meaningful as results obtained in future attempts. This process is new for all involved and several issues arose which impeded the achievement of optimal results for most agencies. It was FDOT's intention to discover and implement the data collection and reporting methodologies that will lead to the most valid TCQS results with minimum effort by the participants. Possible remedies and improvements to the process, based on the experiences of this first year, are provided in the form of a series of recommendations.

The purpose of the TCQS measures is to establish a means of evaluating the quality of transit service, from the users' perspective, that can be comparable to the level of service measures used for roadways, which are also designed from the perspective of the user (i.e., level of congestion). It is the hope of FDOT that the routine implementation of this procedure in the future will lead to increased investment in transit services throughout the state by prompting the allocation of resources toward the improvement of transit services with poor TCQS measures, similar to the response when roadways are deemed to perform poorly. This study represents the first known statewide use of this new customer-oriented transit performance evaluation procedure, and the results of this project will be beneficial to other DOTs, MPOs, and transit systems throughout the country that may be interested in the application of the performance measures found in the TCQSM.

FDOT TRANSIT QUALITY OF SERVICE INITIATIVE

In 2001, FDOT required that Florida MPOs where fixed-route transit exists organize an effort to evaluate those fixed-route services in terms of the six transit quality of service measures in the TCQSM (*1*). The six TQOS measures evaluated are:

1. service coverage
2. service frequency
3. hours of service
4. transit travel time versus auto travel time
5. passenger loading
6. reliability (on-time performance or headway adherence)

The TQOS framework, as presented in the First Edition of the TCQSM (1), focuses on transit service availability, comfort, and convenience from the users' point of view and culminates in these six measures. The first three measures, service coverage, service frequency, and hours of service, relate to the availability of transit service to the user. The measures of travel time (transit versus auto), passenger loading, and reliability are associated with the comfort and convenience to the transit user. Each measure is expressed on a scale from "A" to "F," similar to roadway level of service measures, with "A" denoting the best quality of service and "F" representing the worst quality of service.

Each measure, except service coverage, was to be applied on a typical weekday p.m. peak period. Service coverage was evaluated for the typical weekday. A typical weekday was defined as Tuesday, Wednesday, or Thursday, and the p.m. peak period was defined from 4:00 p.m. to 6:00 p.m. The p.m. peak was chosen to mirror the p.m. peak period analysis procedures identified in FDOT's *Level of Service Handbook* for highways. The TCQS evaluations were to be conducted in March 2001, with a final report from each MPO area due to FDOT by July 1, 2001.

The evaluation process began with the selection of major activity centers in each study area. Large areas with populations of 200,000 or more were to select at least 10 activity centers, while smaller areas were to select at least 6 activity centers. The objective was to choose activity centers where demand is high for people in the community to travel to and travel from. Guidelines were provided for the areas to aid in the selection of the activity centers. Once the activity centers were chosen, trip pairs were developed from each activity center to all the other activity centers. From the local travel demand model, total trip demand (auto and transit), measured in trips per hour, was generated for each O-D pair.

The measures for service frequency quality of service (QOS), hours of service QOS, and transit travel times (for use in the travel time QOS measure) were developed using existing transit route maps and schedules produced by the individual transit systems for the public. Auto travel times, necessary to complete the process for calculating the Travel Time QOS measure, were derived from the local travel demand model.

Passenger loading and reliability data were required to be measured for only the 15 O-D pairs with the highest travel demands based on the model results. Measurements on these trip pairs were to be made at the maximum load point for trips departing the origin between 4:00 p.m. and 6:00 p.m. If a passenger would need to transfer from one transit route to another to complete the trip, data were collected for only the first segment of the trip. Reliability information was to be recorded using the arrival time of the vehicle at the maximum load point. Passenger loading QOS was calculated using automatic passenger counter (APC) data for two transit systems, while all others used field measurements. Reliability QOS could be calculated using automatic vehicle location (AVL) data, but all participating systems, except two, used field measurements. For both passenger loading and reliability, either 10 observations or three days of peak observations should have been made, whichever is greater.

Service coverage QOS most easily could be determined by using geographic information systems (GIS) technology. However, if GIS software was not available in an area, a manual method, described in the *Agency Reporting Guide* (*2*), could be applied. Two of the participants in this evaluation utilized the manual technique for measuring service coverage. Data on population, households, and employment was needed by geographical unit such as traffic analysis zone (TAZ) or census block group. While the reporting agencies were to indicate the type of data used and the year that the data represented, there was no specification as to exactly which data or which year should be used. This makes sense, since various areas around the state may have different types of data more easily available or more recent than others. However, the various data used by the participating agencies did not facilitate a consistent aggregation of the service coverage QOS for the state as a whole.

EVALUATION OF FIRST-YEAR TCQS REPORTS

This section addresses the process undertaken by the participating agencies and also analyzes the resulting TQOS measures. Each of the MPO's reports was collected and reviewed to provide an overall assessment of how well the MPOs and transit systems conducted the evaluation.

Review of the Process

All but one of Florida's MPOs that were required to participate in this effort did so and submitted a report to FDOT. This resulted in 17 MPO reports representing 18 fixed-route transit systems. While a few participants submitted only the completed evaluation spreadsheets, others prepared additional written materials ranging from a simple memorandum to detailed reports. The transit agencies represented in this evaluation are listed below.

- Broward County Transit
- Escambia County Area Transit
- Gainesville Regional Transit System
- Hillsborough Area Regional Transit Authority
- Jacksonville Transportation Authority
- Lakeland Area Mass Transit District
- Lee County Transit
- Lynx (Central Florida Regional Transit Authority)
- Manatee County Area Transit
- Miami-Dade Transit
- Ocala/Marion MPO (SunTran)
- Palm Beach County Transportation Agency
- Pasco County Public Transportation
- Sarasota County Area Transit
- Space Coast Area Transit
- Tallahassee Transit
- Volusia County dba VOTRAN
- Winter Haven Area Transit

All of the participating agencies selected at least the minimum number of activity centers. Several participants indicated that the auto travel times derived from the local models were suspect. The issue of comparability between the theoretically estimated auto travel times and the transit travel times recorded from actual transit schedules was a contentious one.

The compilation and reporting of the service frequency, hours of service, and transit travel time data for the O-D trip pairs were relatively straightforward, with the relevant information being readily available from published transit schedules. Complications tended to arise with the collection of the loading and reliability data for the top 15 O-D pairs, and stemmed from both the determination of the top 15 pairs and the methods applied to collect the pertinent information.

Review of the Statewide Results

To provide a benchmark evaluation of Florida transit systems, performance was analyzed using the six TQOS measures. As stated previously, these measures represent the passengers' point of view and are denoted by a scale of "A" through "F," with "A" representing the best service from the passenger's perspective, and "F" representing the worst service.

Service Frequency QOS

This measure is one of the most relied upon when determining customer satisfaction, and improving frequency is often considered when transit systems wish to strengthen core ridership and attract new riders. While transit-dependent riders often must adjust to prevailing schedules, it is very difficult to attract choice riders with infrequent service. As Table 1 shows, nearly half of the trip pairs in the evaluation (48.4 percent) received a Service Frequency QOS E, meaning that service was available only once during the hour.

Hours of Service QOS

For those passengers who depend on transit service, inconvenient hours of operation require adjustments of activities and schedules to utilize the service. Those with alternative means of travel may simply opt not to use transit when the hours of service are limited. As Table 1 indicates, most of the trips in the evaluation (28.9 percent) represent systems in which daytime service is typical (QOS D). Nearly 40 percent of the total trips evaluated represent systems that provide some type of evening or night service on at least some routes (QOS A, B, or C).

Transit vs. Auto Travel Time

This measure compares the travel time between selected origins and destinations using both transit schedules and model-derived estimates for automobile travel. As mentioned previously, many participants expressed concern over the model estimates being used to calculate auto travel time. Some indicated that the only valid technique would be to determine auto travel times by actually test-driving an auto on the trip on the same day(s) transit measurements are taken. As identified in Table 1, nearly 30 percent of the trips evaluated for travel time would be considered QOS F. However, nearly five percent of trips evaluated were determined to be as fast or faster by transit than by auto (QOS A).

TABLE 1. Summary of All O-D Pairs: Service Frequency, Hours of Service, and Travel Time QOS

QOS	Service Frequency QOS			Hours of Service QOS			Travel Time QOS		
	State	Small	Large	State	Small	Large	State	Small	Large
A	1.1%	0.0%	2.6%	6.3%	0.7%	14.1%	4.9%	7.2%	1.7%
B	1.2%	0.4%	2.4%	11.3%	0.2%	27.0%	20.0%	25.6%	12.0%
C	12.7%	5.8%	22.5%	21.1%	17.2%	26.6%	17.1%	18.9%	14.4%
D	25.7%	24.4%	27.7%	28.9%	35.1%	20.3%	14.4%	13.0%	16.5%
E	48.4%	59.5%	32.8%	19.8%	29.6%	5.8%	13.7%	12.4%	15.6%
F	10.8%	10.0%	12.0%	12.6%	17.1%	6.2%	29.9%	22.9%	39.7%

Note: "State" denotes total pairs; "Small" is < 50 peak vehicle systems; "Large" is > 50 peak vehicles.

Passenger Loading QOS

While a crowded bus signals high ridership, in the case of this evaluation a crowded bus represents an undesirable situation for the transit passenger. For many participants, it was inconceivable that mostly empty transit vehicles warranted the better QOS. However, loading from the perspectives of the transit rider and transit provider represent two very different views. Since this evaluation represents the perspective of the passenger, QOS A indicates that there are so many seats available that passengers can choose where to sit and do not need to sit next to any other passenger(s). This condition exists until the vehicle is half full. At the other end of the spectrum, QOS F represents those situations where all seats are occupied and there are at least half that many more passengers standing in the transit vehicle.

As presented in Table 2, QOS A far exceeded the other levels of service with 83.9 percent of the observed trips. Very few of the agencies' top trips were crowded such that QOS F was earned. In fact, the seven trips with QOS F came from one transit agency.

TABLE 2. Summary of Top 15 O-D Pairs: Passenger Loading and Reliability QOS

QOS	Passenger Loading QOS			Reliability QOS		
	State	Small	Large	State	Small	Large
A	83.9%	94.6%	67.1%	20.4%	24.8%	13.4%
B	9.0%	3.9%	17.1%	0.5%	0.0%	1.2%
C	1.4%	0.0%	3.7%	10.4%	10.9%	9.8%
D	0.0%	0.0%	0.0%	1.4%	0.0%	3.7%
E	2.4%	1.6%	3.7%	11.8%	13.2%	9.8%
F	3.3%	0.0%	8.5%	55.5%	51.2%	62.2%

Note: "State" denotes total pairs; "Small" is < 50 peak vehicle systems; "Large" is > 50 peak vehicles.

Reliability QOS

The reliability measure reflects a comparison of actual versus scheduled arrival times of transit vehicles at stops/stations that reflect the maximum load point of the first segment required to take the trip via transit. On-time performance is a critical factor when evaluating transit service, as it is indicative of the degree to which passengers can depend on the system. On-time performance is defined as the arrival of the transit vehicle within five minutes of the printed time on the schedule. The distribution of statewide Reliability QOS results in Table 2 reveals that, while the majority of the top trips made reflect poor reliability (55.5 percent received QOS F), there are several trips for which the agencies have been able to maintain a high level of on-time performance (20.4 percent received QOS A).

Service Coverage QOS

The final measure is indicative of passengers' satisfaction related to whether the system provides service to the areas they want to go. The QOS measure for service coverage is the percent of the transit-supportive area served for each transit system. For this evaluation, an area is considered transit-supportive if it has a minimum population and/or employment density to support at least hourly service. A density of three housing units per acre or four employees per acre is required, and the area must be within walking distance (within one-quarter mile of a bus stop or one-half mile of a rail or busway station) to transit service.

Table 3 shows that 5 of the 17 participating agencies (29.4 percent) represent QOS F, i.e., have coverage of less than 50 percent of the transit-supportive area. While this suggests that several agencies are not providing access to areas having sufficient population or employment activity, Table 3 also indicates that nearly 65 percent have coverage in at least 60 percent of their transit-supportive areas (at least QOS D).

TABLE 3. Summary of Service Coverage QOS Results

QOS	Statewide	Small Systems (< 50 Peak vehs.)	Large Systems (> 50 vehs.)
A	4 systems (23.5%)	2 systems (20.0%)	2 systems (28.6%)
B	0 systems (0.0%)	0 systems (0.0%)	0 systems (0.0%)
C	4 systems (23.5%)	3 systems (30.0%)	1 system (14.3%)
D	3 systems (17.6%)	1 system (10.0%)	2 systems (28.6%)
E	1 system (5.9%)	1 system (10.0%)	0 systems (0.0%)
F	5 systems (29.4%)	3 systems (20.0%)	2 systems (28.6%)
Total	*17 systems (100.0%)*	*10 systems (100.0%)*	*7 systems (100.0%)*

Note: “State” denotes total pairs; “Small” is < 50 peak vehicle systems; “Large” is > 50 peak vehicles.

CONCLUSIONS AND RECOMMENDATIONS

The first-year Transit Capacity and Quality of Service Evaluation proved to be a valuable learning experience for everyone involved: participating MPOs and transit systems, FDOT, and consultants and researchers who assisted in this effort. Undoubtedly, the developers of this evaluation process can also learn from this first statewide application of their work. While a summary of the statewide Florida TQOS results is included in this paper, significant emphasis is placed on the process of the evaluation itself. Exhaustive interviews were conducted with representatives of each participating agency (MPOs, transit systems, and consultants) to obtain insight as to how the process unfolded in each area and to identify obstacles, problems, and issues that arose. This section summarizes the major issues that surfaced during the course of the evaluation and presents a series of recommendations that will help ensure the most valid results from this process in the future.

1. The minimum number of activity centers seemed adequate. Regarding the selection process, some participants indicated that they looked at this task as a useful exercise to determine where people are coming from and going to, and to see how well transit serves those centers. However, some indicated that the temptation would be strong to select centers of activity already well served by the existing transit system so as to demonstrate better QOS measures. While FDOT is aware that QOS measures are not expected to be very strong statewide, at the local level, some individual agencies felt the need to look after their own interests and present their transit system in the most positive light for fear of local media gaining access to and misinterpreting the purpose and results of the evaluation. As a result, **it is recommended that activity centers be reselected for each evaluation, as appropriate, to reflect new growth and travel patterns. Also, it might be best for transit systems to have less involvement in the selection; the MPO or an objective party**

should oversee the selection process. A balance between origins and destinations should also be achieved.

2. From the O-D pairs derived according to the activity centers, the local travel demand model provides estimates of total travel demand (auto and transit) for each trip, which are then ranked. passenger loading and reliability QOS measures are then applied to the 15 O-D pairs with the highest travel demands. While this seems straightforward, most of the participants in this evaluation experienced moderate to extreme difficulty in determining the travel times between the activity centers (models will measure between the centers of TAZs, not point-to-point) and expressed discontent that theoretically-estimated travel demands and travel times were being compared to actual transit loads and travel times. Some participants believe that, although it would be labor-intensive, the best way to do the comparisons would be to take field observations on auto travel between the activity centers, i.e., drive the trip in an auto on the same day(s) the transit observation(s) are made. The travel demand models in each individual area often use different years' data, are updated on different schedules, and provide results in varying forms. **Individual areas need to be aware of exactly what information the local model is providing, and must be sure that any peak and seasonal factors are applied as appropriate. Since this evaluation is primarily the MPOs' responsibility, MPO staff should take the lead in working with the model.**

3. Another issue deals with selection of the top 15 trips and the occurrence of both directions of movement between two activity centers in the top 15 trips. Movement between residential areas and employment centers is different between other pairs of destinations such as between an airport and a CBD, for example. Nonetheless, participants were to rank all of their O-D pairs and select the 15 pairs with the highest travel demands for further measurements. Passenger Loading and Reliability QOS results in this study represent only the top 15 trips. However, several participants analyzed the resulting top 15 O-D pairs and tweaked them to either remove one pair's direction of movement if both were included or to be sure that all activity centers were represented. The latter reason may deal more with the initial selection of activity centers if trips between some were not recognized in the top 15 pairs. Otherwise, **if the top 15 trips include both directions of movement between one or more pairs, and the travel demand results are not identical, then both trips should be included in the final analysis. To ensure consistency across agencies in the state, all participants should analyze their top 15 O-D pairs according to the model results. Further exploration is needed on this issue, and should be addressed in any future update of the *Agency Reporting Guide*.**

4. The time frame for the QOS measurements is the p.m. peak period as defined from 4:00 p.m. to 6:00 p.m. This ensures consistency with roadway traffic measurements. However, if one of the trips among an agency's top 15 is from a residential area to a center of employment, such as an industrial park or CBD, it is logical to assume that this trip has the highest travel demands in the a.m. peak. In such a case, the travel demand model is forecasting all-day demands without accounting for time period. One participant, which had such a trip, measured the reverse direction in the p.m. peak. This makes intuitive sense, but this problem would be eliminated if measurements could be taken in the a.m. peak, as well. Several agencies suggested the idea of including the a.m. peak. **It is recommended that participants analyze the resulting top 15 O-D pairs and determine the time period (a.m. versus p.m. peak) during which the individual trips would be expected to have the higher travel demands. Then, the trip could be measured during the appropriate time period. If no valid determination can be made, then the measurement should default to the p.m. peak period.**

5. Thresholds for passenger loading are determined using the square footage available per passenger or the number of passengers per seat. This accounts for standee loads and, understandably from the riders' perspective, the more crowded the vehicle, the lower the QOS. However, one's level of comfort with a crowded vehicle or even a standee load is usually inversely proportional to the length of the trip. To be certain, there are some individuals who would never be comfortable standing for any length of time. However, for many, standing for a shorter length of time can be acceptable. **Further exploration is needed to determine whether the length of the relevant trip or segment can be incorporated into measurements for Passenger Loading QOS.**

6. For the purposes of obtaining passenger loading and reliability information, if a transit trip between activity centers necessitated a transfer, measurements were to be taken at the maximum load point of the first segment required for the trip. Many participants believe that this methodology did not result in a meaningful representation of the entire trip. **For transit trips that include one or more transfers, it is recommended that service frequency and hours of service information be averaged over the routes required to accomplish the trips. The maximum load point along the entire trip should be determined, and then measurements for passenger loading and reliability taken on the segment that represents that maximum load point.**

7. Each agency was to collect field observations for passenger loading and reliability during March 2001. Several participants believe that this window of time is too short. For some, especially those with less service frequency, it was difficult to obtain the minimum number of observations within the time frame due to staffing issues. Systems with less frequent service could collect fewer observations per day and thus needed staff in the field for more days than the systems with higher peak frequencies. In addition, if trip observations were missed or collected incorrectly, there were often little or no other opportunities to collect the information as required. Some participants faced with this situation collected data after March 2001, while others substituted other O-D pairs outside the top 15 or simply completed the evaluation using the fewer number of observations. While the need to resample trips due to data collection error will undoubtedly decline as the TCQS process is refined and familiarity among the participants improves, **it is recommended that the time frame for collecting passenger loading and reliability data be increased from the current four-week time period to a six- or eight-week period.**

8. Yet another issue to consider regarding the time frame for collecting the data pertains to the varying peak months experienced by agencies in various geographic locations throughout the state. While locations in central and south Florida tend to experience peak travel demands and ridership between February and April, locations in northern Florida tend to experience spikes during the summer. It is understood that one of the intentions of this first-year effort was to examine a "snapshot" of transit performance across Florida. However, it was also the intention to measure typical weekday transit performance in the p.m. peak of the peak travel time. **If it is indeed the case that FDOT wishes to measure the performance of its transit systems during peak ridership months, then participants should be able to choose the time frame based upon individual agency ridership variations. Participating agencies would need to provide evidence that the selected time frame represents the ridership peak. In this case, the statewide results would be presented in terms of how well Florida's transit systems perform, overall, during their peak periods. The statewide results could be compiled at the end of a calendar or fiscal year to allow time for each participant to complete the evaluation.**

9. If the recommendation to widen the window of time for the collection of field data is implemented to allow for a six- or eight-week time frame, then the requirement to collect the information only on Tuesdays, Wednesdays, or Thursdays should stand. These three days are representative of the "typical" weekday and are the days used for collecting traffic information.

However, given the shorter four-week time period, it may be feasible to allow some systems to collect data on Mondays, as well. Several of the participants representing smaller areas indicated that their weekday ridership is flat, and that there is no statistical variation Mondays through Thursdays. These smaller areas also tend to have the less frequent transit services, necessitating additional data collection days to acquire the minimum number of observations. **It is recommended that, if a participating agency can show valid data to prove that Monday ridership is not statistically different from ridership on Tuesdays, Wednesdays, or Thursdays, then that participant should be able to use Mondays to collect field data. Data collection on Fridays would not be allowable.**

10. The reliability measurements caused problems for many. Reasons for the generally poor Reliability QOS results offered by participating agencies were numerous. Most indicated that transit schedules are not written for the peak periods and that "everyone" is always late during the peaks. Some observed that poor roadway LOS resulted in poor Reliability QOS since transit vehicles must negotiate the congested traffic conditions. Also, it is often the case that a maximum load point, where measurements should be taken, occurs at a transfer center, where several buses may meet for timed transfers. Often the transit vehicles wait for each other so if one runs late they all will be late. Another issue is when recovery time is built into a schedule so that a vehicle may arrive at a transfer center that represents a maximum load point more than five minutes early (designating it as not on-time according to many participants' interpretation of the guidelines), but will leave on schedule. Finally, participants noted that, for example, being 30 minutes late is counted the same as being 6 minutes late. **The results of the reliability QOS measures should lead to a closer look at a system's schedules to be sure they are realistic for peak conditions. Closer examination is needed of the threshold definitions of the reliability QOS measure. A sliding scale may be appropriate so that a worse QOS level is associated with a greater number of minutes late.**

11. Some participants in this evaluation speculated that the application of the TCQS measures to route segments between activity centers is not a complete representation of the transit service. Some believe that, by using the TCQS measures to evaluate whole routes, the results would be easier to understand and would render a more accurate portrayal of system performance. However, other participants realize that, to best evaluate how well transit serves the trips with the highest travel demands, it is necessary to evaluate the trip itself. As such, **this report recommends that the evaluation of the transit trips between major O-D pairs continue in subsequent evaluations.**

12. Results from the service coverage QOS analysis were difficult to interpret on a statewide level due to the fact that different methodologies and data representing various years were used by the participants. **Participating agencies should all use data representing the same year in calculating service coverage, if possible.**

13. Training courses and materials were provided to the participating agencies in advance of this first-time effort. Many participants had little trouble with the process and/or were pleased with the support provided. The training courses were, by all accounts, extremely helpful. However, some areas were unable to send representatives to the training, and they tended to encounter more difficulties during the evaluation. **It is recommended that, with the experience gained by all involved with this first-time effort, additional training be held in the future. Also, the Agency Reporting Guide should be updated to include new or modified procedures and a clarification of other issues.**

14. Strong opinions were voiced by participants regarding the fact that no additional funds were provided to the agencies to conduct this required evaluation, particularly by transit systems that had to shoulder a larger portion of the work involved in completing this effort. Perhaps a few of the

MPOs, realizing there were no new funds for this project, expected the transit systems to take on more of the tasks, resulting in the notion of "passing the buck," as expressed by one participant. Costs to conduct this evaluation ranged from "negligible" to $50,000, with an average of $4,500. **Clearly, in some areas, a much higher level of resources was expended on this evaluation than should have been necessary. With proper advance planning and by taking advantage of less expensive local labor options if needed (e.g., college students, senior groups, volunteer organizations, etc.), costs should be kept at a minimum. It is anticipated that future efforts will cost less as participants prepare earlier and become more familiar with the process.**

15. The evaluation was originally intended to be an annual effort. However, given the fact that local travel demand models are not updated annually and the fact that transit services do not typically experience significant changes from year to year, the **benefits (i.e., useful information) from annual evaluations may outweigh the costs. Therefore, it is recommended that the TCQS Evaluation be conducted in full as part of the Long Range Transportation Plan Update process. Each agency that is expected to participate should be sure to plan appropriately for the proper collection and reporting of the TQOS measures.**

The items and recommendations presented herein are intended to provide an overall assessment of Florida transit performance in terms of the six TQOS measures included in the First Edition of the Transit Capacity and Quality of Service Manual. More importantly, this paper evaluates the process of the first-year statewide implementation of these measures and summarizes the experiences of those involved. It was the objective of this study to provide FDOT and other interested parties guidance on refining the process to extract meaningful and valid results in the future, with minimum effort. Overall, further research is needed in areas regarding the selection of O-D pairs, data collection for passenger loading and reliability, and the thresholds for the TQOS measures, particularly reliability and overall on-time performance issues. With better, consistent results, the aim of evaluating statewide transit service on an "A" through "F" scale, similar to roadway LOS, can move Florida closer to the ultimate goal of increasing investment in public transit services and can serve as a model for other states with the same objectives.

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Strengthening of Steel Girder Bridges Using FRP

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ABSTRACT

This paper documents two projects funded through the Federal Highway Administration's Innovative Bridge Research and Construction (IBRC) program. The IBRC program was developed to assist bridge owners in applying emerging technologies in bridge engineering. In these projects, the Iowa Department of Transportation employed techniques for strengthening steel girder bridges using carbon fiber reinforced polymers (CFRP). Two bridges were strengthened using CFRP composite materials in an effort to improve the live load carrying capacity of the bridges. In one case, a bridge was strengthened using CFRP post-tensioning rods in the positive moment regions. In the other case, a bridge was strengthened by installing CFRP plates to the bottom flange of girders in the positive moment regions.

Key words: beam—carbon fiber reinforced polymer—strengthening

INTRODUCTION

Many state, county, and local agencies are faced with an ever-deteriorating bridge infrastructure composed in large part of relatively short to medium span bridges. In many cases, these older structures are composed of rolled or welded longitudinal steel stringers used as part of a continuous slab-on-girder bridge. Most of these bridges continue to serve as an integral part of the transportation system yet need some level of strengthening due to increases in live loads or loss of capacity due to deterioration. The bridges are usually not critical enough to warrant replacement so a structurally efficient, but cost- effective, means of strengthening needs to be employed. In the past, the use of bolted steel cover plates was a common retrofit option for strengthening such bridges. However, the time and labor involved to attach such a retrofit can be prohibitive.

Among various strengthening methods, the use of carbon fiber reinforced polymers (CFRP) composite materials is very appealing in that they are highly resistant to corrosion, have a low weight, and have a high tensile strength. In the last decade, the use of these composite materials has emerged as a promising technology in structural engineering *(1)*. With this nationwide recognition, two projects, funded through the Federal Highway Administration's Innovative Bridge Research and Construction (IBRC) program, were initiated to investigate the effectiveness of using CFRP composite materials to strengthen existing steel girder bridges. Two bridges were strengthened using CFRP composite materials in an effort to improve the live load carrying capacity of the bridges. In one case, a bridge (Bridge 1) was strengthened using CFRP rods that had been post-tensioned in the positive moment regions. In the other case, a bridge (Bridge 2) was strengthened by installing CFRP plates to the bottom flange of girders in the positive moment region.

OBJECTIVES

The two primary objectives of these projects were to investigate the effectiveness of CFRP composite materials to strengthen existing, structurally deficient steel girder bridges and to identify changes in structural behavior due to addition of strengthening system and time. To accomplish these objectives, testing of the installations was coupled with theoretical calculations.

STRENGTHENING WITH CFRP POST-TENSIONING RODS

Description of Bridge 1

Bridge 1 (number 3903.0S 141) which was strengthened with post-tensioning (P-T) CFRP rods is a 210 ft x 26 ft three span continuous rolled shape steel girder-bridge in Guthrie County, Iowa, on State Highway 141. The bridge is currently rated with an HS 11.0 Inventory Rating and an HS 18.7 Operating Rating. The bridge has a total length of 210 ft consisting of two end spans of 64 ft and a center span of 82 ft with four beams spaced at 8 ft -3 in. on center. The roadway width is 26 ft allowing two traffic lanes and a narrow shoulder. This bridge is considered to be a borderline case which may soon require posting.

(a) Side view

(b) Road way

FIGURE 1. Photographs of the Bridge 1

Strengthening System

The strengthening system utilized was developed based on the strengthening recommendations of Klaiber et al *(2)* and material performance data provided by the manufacturer in an effort to increase the load carrying capacity of the bridge. CFRP rods (3/8 in. in diameter) were selected due to their outstanding mechanical characteristics and non-corrosive nature. Anchorage assemblies consisting of 5 in. x 5 in. x 3/4 in. stiffened angles 7 in. in length, 1 in. couplers, and steel tube anchors, were used to connect the P-T CFRP rods to the bridge. Each post-tensioning bracket (connected to the web of the steel beams with 1 in. diameter high strength bolts) connects four CFRP rods to the web of the beam near the bottom flange. Material properties of CFRP rods used are listed in Table 1.

TABLE 1. Material Properties of CFRP Rods

Diameter, in.	Tensile Strength, ksi	Tensile Modulus, ksi	Elongation at ultimate	Fiber Content
3/8	300	20,000	1.5 %	65% by Volume

Installation of CFRP Post-Tensioning Rods

The P-T system was installed in the positive moment region of the exterior girders in all three spans. The anchorage assemblies were bolted to the webs of the beams with 1 in. diameter A325, high-strength bolts torqued in accordance with the manufacturer's recommendation (Figure 2a) at locations based on design calculations and field measurements. The CFRP post-tensioning rods were then placed in position between each pair of anchorage assemblies on both sides of the web (Figure 2b). Application of the post-tensioning force was completed in several steps at each location (subsequently referred to as "events"). A nominal force of 12 kips was applied to all rods (four rods per location) in a symmetrical manner. Photographs of the application of P-T force in West and Center span are shown in Figure 2 (c) and (d) while the location of the post-tensioning system and photographs of the completed installation are presented in Figures 3 and 4, respectively.

(a) Installation of anchorage assembly

(b) Placement of CFRP rod

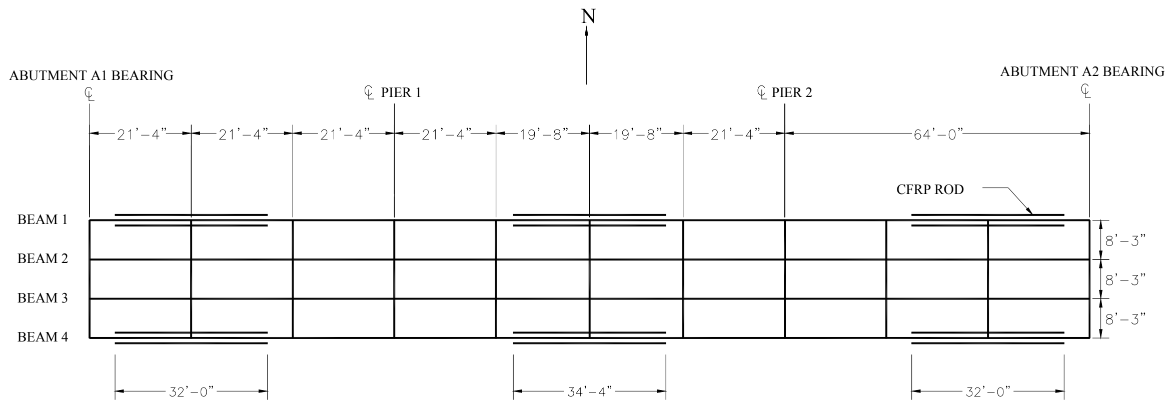
(c) Application of P-T force in West-end span

(d) Application of P-T force in Center span.

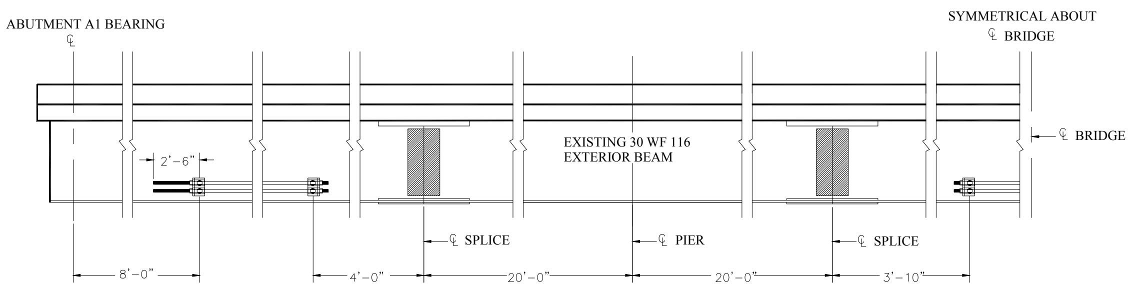
FIGURE 2. Photographs of the Installation of the P-T System on Bridge 1

Load Testing

The bridge was instrumented to measure the strains at strategically selected locations and tested before installation of the post-tensioning system, immediately following post-tensioning, and after approximately one and two years of service to assess changes in performance resulting from the addition of the P-T system and time. Standard 3-axle dump trucks, driven at a crawl speed, were used in the load tests and data were collected continuously as the trucks crossed the bridge. Data were collected for four different load paths with two test runs being conducted for each path. The locations of strain gages and the truck paths used are shown in Figures 5 and 6, respectively.



(a) Plan view



(b) Side view

FIGURE 3. Location of the Post-Tensioning System

(a) CFRP Rods on West-end span *(b) CFRP rods on Center span*

FIGURE 4. Photographs of the Completed CFRP P-T System on Bridge 1

Results

Typical bottom flange strain data at the center span from each load test are presented in Figure 7. The increase of strain due to the P-T force applied on Beam 1 in the Center span is illustrated in Figure 8 where each "event" defines a specific step of the overall P-T process.

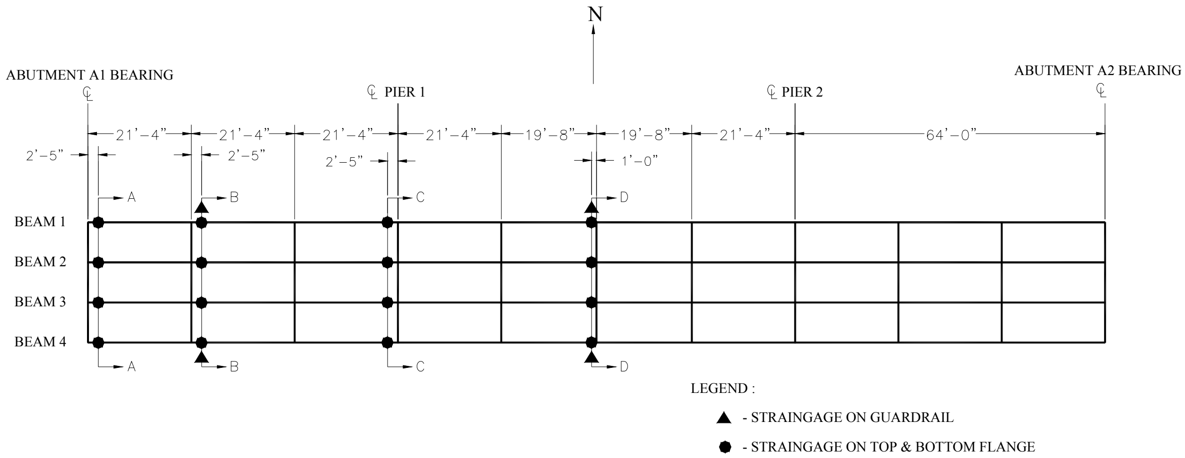


FIGURE 5. Location of Strain gages on Bridge 1

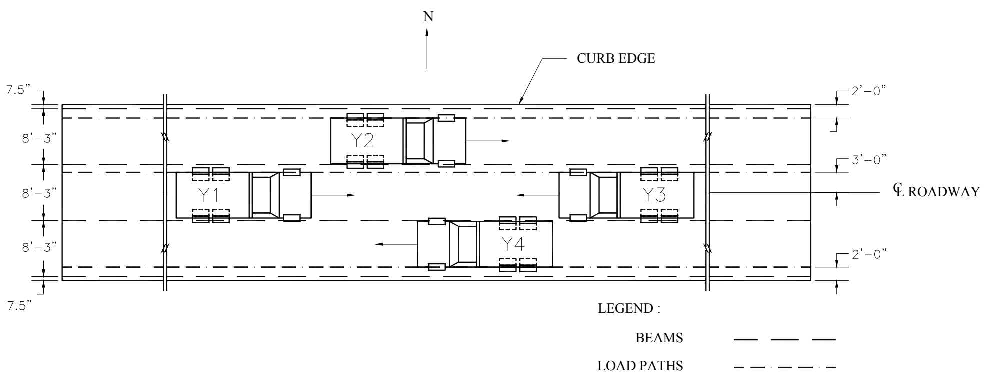


FIGURE 6. Truck paths used on Bridge 1

Each load test produced fairly consistent strain readings. This consistency in strain is informative in that it indicates that the P-T system did not alter the behavior of the bridge over the two years of service. Although it is not possible to precisely account for all the sources of strain, it is evident from the consistency of the strain data that the installation of the P-T system had negligible impact on changing the stiffness of the bridge. In general, good agreements in strain data were observed.

Post-tensioning the positive moment region generates strain opposite in sign to those produced by dead and secondary load. During the application of the P-T system, strain was measured to investigate the response of the structure due to the application of the P-T force. When the force was applied to a certain location on the exterior beam, that beam would experience more strain than other location. For example, when the north exterior beam (Beam 1 in Figure 5) on the Center span was post-tensioned (between the events 48 and 64 in Figure 8), the strain due to the P-T force increased significantly compared to other locations where the increase in strain were minimal.

A classic analysis was performed on the post-tensioned bridge utilizing an HS-20 truck as a live load. Depending on the location, it was obtained that approximately 5 to 10 percent of the live load moment was reduced by the P-T moment.

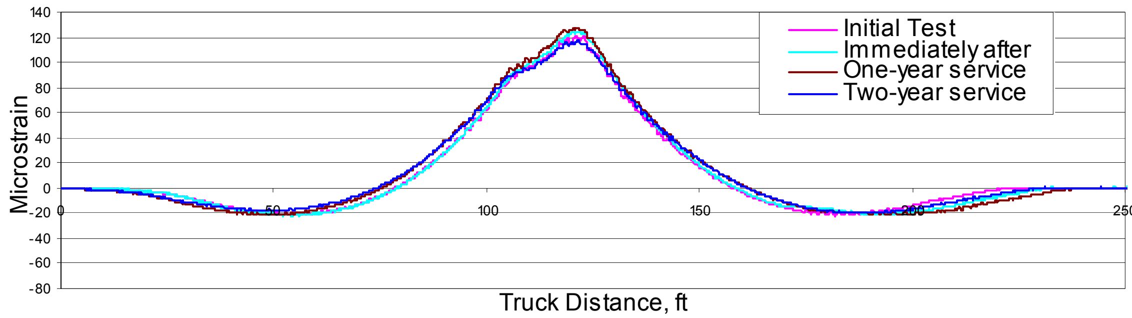


FIGURE 7. Bottom Flange Strain Data (Path Y2, Beam 1 on Center Span of Bridge 1)

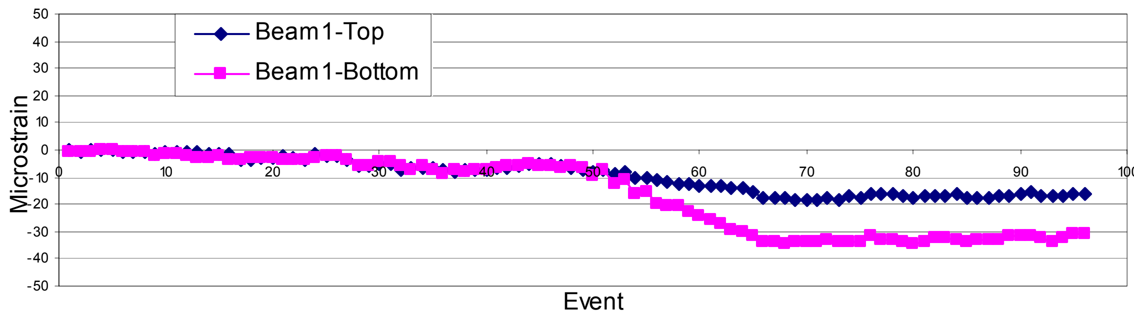


FIGURE 8. Increase of Strain Due to the Post-Tensioning Force on Bridge 1

STRENGTHENING WITH CFRP PLATES

Description of Bridge 2

Bridge 2 (number 7838.5S092) strengthened with CFRP plates is a 150 ft x 30 ft three-span continuous I-beam bridge in Pottawattamie County, Iowa, on State Highway 92. Originally the bridge was a non-composite structure. In 1965, the bridge was widened with the addition of exterior girders that were made composite with the deck. The bridge has a total length of 150 ft consisting of two end spans of 45.5 ft and a center span of 59 ft and is supported by six beams: two W27x84 exterior I-beams installed in 1967, two W27 x91, and two W27x98 interior I-beams. The roadway width is 30 ft allowing two traffic lanes and a narrow shoulder.

(a) Side View

(b) Road way

FIGURE 9. Photographs of Bridge 2

Strengthening System

Previous research by Al-Saidi *(3)* at Iowa State University Bridge Engineering Center established the effectiveness of CFRP plates for improving the strength of steel composite beams. With this foundation study available, the next step was taken to implement this strengthening method on an existing steel girder bridge. The material used for strengthening is a pultruded carbon fiber reinforced polymer consisting of continuous unidirectional carbon fiber in an epoxy matrix, specially designed for flexural strengthening. The CFRP plates were selected due to their outstanding mechanical characteristics, non-corrosive nature, and relative ease of application. The material properties of the CFRP plates used are listed in Table 2. The strengthening system consists of preparation of the bonding surface and installing the CFRP plates to the beam surface with a high strength epoxy adhesive for the transfer of force to the high strength CFRP plates. The design of the CFRP plates was completed for the Iowa legal load utilizing the Load and Resistance Factor Design (LRFD) approach. Calculations indicated that the overstressed beams could be adequately strengthened by the addition of CFRP plates bonded to the bottom flange of the beams.

TABLE 2. Material Properties of CFRP Plates

Ultimate tensile stress, ksi	Modulus of elasticity, ksi	Ultimate strain
300	20,000	1.5 %

Installation of CFRP Plates

The CFRP plates were installed on both interior and exterior beams in the positive moment region of all three spans. A successful installation is highly dependent upon careful surface preparation and maintenance of adequate bond conditions. The steel beam surface to which the CFRP plates were to be bonded was roughened to a 'coarse sandpaper' texture by sandblasting with high-pressure air jets to remove the paint and any unsound materials on the bonding surface. Both the sandblasted beam surface and bonding surface (sanded side) of the CFRP plates were cleaned with acetone to remove all dirt and debris. The prepared beam surface was treated with a thin layer of primer to prevent potential galvanized corrosion induced by a galvanic reaction between the beam surface and carbon fibers and to provide an improved substrate for the epoxy adhesive. After the primer was set and tack-free, an appropriate amount of epoxy was applied evenly across the surface of both the beam and the sanded side of the CFRP plates. Finally, the CFRP plates were placed on the designated surface and then pressed and rolled thoroughly to remove any trapped air pockets in the epoxy adhesives. Some of installation procedures are illustrated in Figure 10, and a detailed layout of the plates used can be seen in Figure 11.

A different number of CFRP plates were used on each span to investigate the effect of varying amount of CFRP plates, to compare the response of the different scheme used in different conditions, to investigate the ease of construction of multiple layers, and to evaluate the durability of the installation. In the West end span, a pair of 20 ft long CFRP plates (1 layer) were installed, side by side with a space of approximately ½ in. in between, in the positive moment regions of Beams 1, 3, and 4. Three layers of CFRP plates 20 ft in length were installed on East end span. The center span was strengthened with two layers of CFRP plate 25 ft in length. Beam 6, in all spans, had half of the plates installed on the bottom of the bottom flange and half on the top of the bottom flange. This was done to investigate the performance and in-service durability under detrimental environmental conditions (i.e., direct exposure to sunlight, rain, etc.).

(a) Primer application

(b) Cleaning CFRP plates

(c) Epoxy application on CFRP plates

(d) Epoxy application on beam

(e) Placement of CFRP plate (f) CFRP plate installed.

FIGURE 10. Installation of CFRP Plates on Bridge 2

Load Testing

A diagnostic initial load test was conducted prior to the installation of CFRP plates to establish a baseline static behavior of the unstrengthened bridge. The second field-testing will take place in August 2003 followed by two other tests in two years of period (one in 2004 and one in 2005) to assess any change in performance and behavior resulting from the installation and over time.

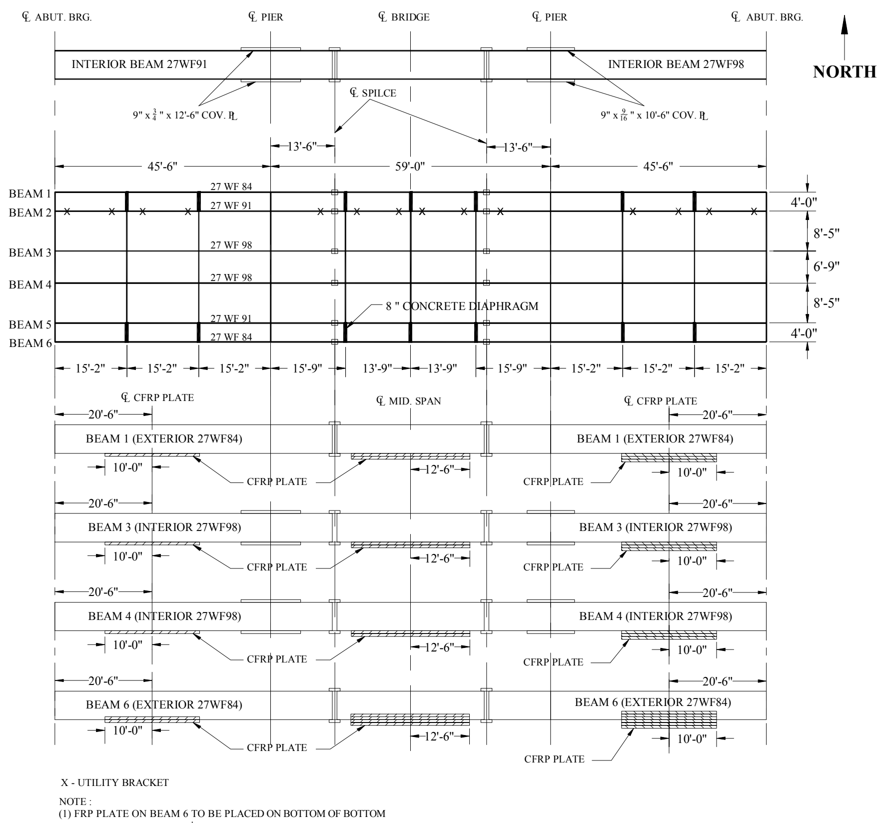


FIGURE 11. Detailed Layout of CFRP Plates on Bridge 2

SUMMARY

As has been described, two existing, structurally deficient steel girder bridges were strengthened utilizing CFRP composite materials. These innovative, yet economical strengthening methods appear to be effective strengthening solutions. Although P-T does not significantly reduce live load deflections, it does increase the live load carrying capacity of the bridge by generating strain opposite to those produced by dead load and, thus, allows the bridge to carry additional traffic live loads. Based on classic analysis utilizing an HS-20 truck, it was determined that approximately 5 to 10 percent of live load moment was reduced by the P-T moment. Finally, the overstressed beams of Bridge 2 were strengthened to enhance their capacity through the addition of CFRP plates attached to the tension flanges. Conclusions on performance and behavior of Bridge 2 will be made as follow-up test takes place.

ACKNOWLEDGEMENTS

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Monitoring and Evaluation of the Iowa River Bridge Launch

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ABSTRACT

Due to the presence of an environmentally sensitive river valley, the U.S. 20 bridge crossing the Iowa River was constructed using a unique superstructure launching procedure. Launching involves pre-constructing the bridge superstructure and then “pushing” the superstructure into place. This is the first example of launching a plate girder highway bridge in the United States. Although not a common construction technique, launching can be a cost-effective alternative. This is especially true when environmental or other constraints limit the environmental impact typically associated with traditional construction.

Monitoring of the Iowa River Bridge consisted of monitoring specific structural behaviors during various stages of construction. The launching operations were monitored such that locations of distress could be identified and so that design assumptions could be verified. Both the bridge superstructure and substructure were monitored at various times. For the substructure, monitoring included measuring strain in the pier columns, rotation of the pier cap, and general displacement of the substructure system. Monitoring of the substructure was performed during four launches. Superstructure monitoring included measuring longitudinal strains at selected cross-sections in the steel girders, longitudinal strains in select cross-frame members, and contact strains in the girder bottom flange and web. In addition, the force required to launch the bridge was monitored.

The monitoring of the Iowa River Bridge was an important effort due to the very limited amount of data available on bridge launching. Engineers need to properly design bridge structures for not only in-service conditions but also for construction load cases which are more complex for a launching method than for conventional construction techniques.

Key words: environmental sensitivity—monitoring—plate girder highway bridge—superstructure launching procedure

Note: Preparation of this paper was still in progress at the time of publication; final results will be presented at the symposium.

Bridge Load Rating Using Physical Testing

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ABSTRACT

The problem of an aging and rapidly decaying infrastructure is an issue facing many agencies charged with maintaining a fully functioning transportation system. Numerous bridges of marginal condition must frequently be posted, resulting in detours with increased travel time and distances. However, when tested, these bridges often exhibit strength and stiffness characteristics beyond traditional codified parameters and beyond calculated rating procedures. The use of diagnostic load testing for the purpose of load rating has become an accepted practice for addressing these bridges by many public agencies. Commercial equipment and analytical tools, like the Bridge Diagnostics, Inc. (BDI) system, have simplified the process of testing, modeling, and rating bridges.

This paper presents a current effort at Iowa State University to evaluate and document the applicability, ease-of-use, and accuracy of a system for load rating of bridges through physical testing. To illustrate the use of the physical load testing, results from the load rating of one of seven bridges that were part of the current research are presented. A typical bridge was instrumented with 40 strain transducers and tested with known loads using the BDI system. Several finite element models of the bridge were then developed and calibrated based on the observed behavior and the field measured strain. Results from the calibrated model were then used to carry out load rating calculations which were then compared to traditional rating calculations. The resulting ratings showed a general increase over the traditional codified ratings. For the subject bridge, various configurations of strain gages were investigated with respect to general influence on modeling and rating accuracy. The results of this sensitivity study are also presented.

Keywords: bridge testing—Load rating

INTRODUCTION

The 2001 Iowa National Bridge Inventory (NBI) Report indicated that of the 25,138 bridges in Iowa, 7,102 (29%) are either structurally deficient or functionally obsolete. While many of these bridges may be strengthened or rehabilitated, some simply need to be replaced. Before implementing one of these options, one should consider performing a diagnostic load test on the structure to more accurately assess it's load carrying capacity. Frequently, diagnostic load tests reveal strength and serviceability characteristics that exceed the predicted codified parameters. Usually, the codified parameters are conservative when predicting the load distribution characteristics and the influence of other structural attributes; hence the predicted rating factors are often conservative. In cases where calculations show a structural deficiency, it may be very beneficial to apply a tool that utilizes a more accurate model that incorporates field-test data; at a minimum, this approach would result in more accurate load ratings and frequently results in increased rating factors. Bridge Diagnostics, Inc. (BDI) developed hardware and software that is specially designed for performing bridge-ratings based on data from physical testing. The hardware consists of pre-wired strain transducers, a data acquisition system, and other components. The software consists of three separate programs for visually evaluating test data, developing an analytical model, analyzing and calibrating the model, and performing load-rating calculations with the calibrated model.

BRIDGE TESTING SYSTEM

The Structural Testing System (STS) is the field component of the testing system, and consists of four main elements: the BDI Intelliducers, the BDI STS Units, the BDI Autoclicker, and the BDI Power Unit. The main purpose of using the STS is to collect bridge behavior data. Specifically, collecting strain data as a truck with known dimensions and weight is driven over the bridge. It is common to position the truck in at least three different transverse positions: the outer wheel line placed at two feet from each curb and the truck centered on the bridge. Additional positions may also be included if needed. Typically, the truck will be driven in each lane twice to verify that the recorded strains are consistent.

The BDI Intelliducer, shown in Fig. 1, is the strain transducer used with the BDI system for measuring bridge response. Each Intelliducer measures 4.4 in. x 1.2 in. x 0.4 in., with either a 15-ft or 25-ft wire attached and has the ability to identify itself to the rest of the system with a unique number (i.e., 4696, 4788, etc.) that can be identified and recognized by the STS power unit (described subsequently). From this unique number, the system has the ability to calibrate and zero the gage using a pre-stored gage calibration factor. Intelliducers may be used on many different surfaces, including, but not limited to, steel, concrete (reinforced and pre-stressed), and timber. For gage placement on reinforced concrete structures, gage extensions should be implemented to increase the 3-inch gage length; the longer length enables surface strains to be averaged over a greater distance, thus reducing the effects of aggregate and cracks in the concrete.

FIGURE 1. A BDI Intelliducer in Use on Top of a Concrete Curb

The BDI STS Unit transfers the data collected from the Intelliducers to the Power Unit (described subsequently). Each STS Unit is capable of collecting data from four Intelliducers and has the capability of storing 50,000 data points during a single test. At the conclusion of a test, the data are transferred to the Power Unit. The unit is equipped with six connection points, four transducer connections, a "line out" connection, and a "line in" connection. All of the connections are quick-lock, military-style. The "line out" transmits data to the Power Unit. The "line in" connection is designed to attach to other units in series and/or parallel through the use of Y-cables. This wiring configuration is a significant advantage over traditional transducer wiring in that only a single cable is connected to the Power Unit.

The Power Unit powers the Intelliducers and transmits commands to the system during the test. Each transducer requires a 5-volt excitation voltage that is provided by the Power Unit. The unit has the ability to operate under two different power sources: DC current from an automobile battery or AC current from a small portable generator or inverter.

The BDI Autoclicker measures and transmits the load vehicle position to the Power Unit through the use an electronic eye and hand-held radio transmitters. A reflective strip placed on the load vehicle's tire triggers the electronic eye. Thus, every wheel revolution creates a "click" in the data. These "clicks" are used to convert data collected in the time domain to the truck position domain.

The control functions of the system are performed by the STS software. The software is run in a Microsoft Windows environment on a laptop computer that is attached, via a parallel port, to the Power Unit. The system is relatively easy to use with pull down menus and large command buttons. The main software menu window contains most of the information that is critical to the load test. Items such as sample frequency, test length, and file output name are easily accessible in the main window. Other options specifically related to Intelliducers such as channel gain, initial offset, and filtering are located in the advanced options menu. Careful attention should be given to these settings to ensure proper data collection.

The BDI Software Package is the analytical modeling part of the testing system, and consists of three main components: WinGRF – data presentation, WinGEN - model generator, and WinSAC - structural analysis and correlation. All elements serve different purposes, but each is essential to the overall process. Each component has been developed such that data can be seamlessly moved from one application to another. These three components are described in detail in the following paragraphs.

WinGRF is used for graphical data presentation, and is the first step in the modeling process. With a known truck position plots can be viewed to observe bridge behavior information, such as the presence of end restraint, non-symmetric behavior, etc as the truck crosses the bridge. Plots, such as neutral axis location, may also be constructed if the distance between a pair of top and bottom Intelliducers has been input in the program.

WinGEN is a finite element model generator developed specifically for bridge modeling. This application allows the user to create models using beam and shell elements. A 2-D model can be created using the WinGEN; however, it is also possible to create a 3-D model using a commercial drafting program, such as AutoCAD, which is then imported into WinGEN.

After a model has been created in WinGEN, the WinGEN output file is used in WinSAC. WinSAC performs analytical calculations and also constructs iterative analytical solutions by changing user defined optimization parameters within user defined boundaries. The resulting model, in the best way possible, represents the actual bridge behavior given user entered constraints. Typical variables chosen as optimization parameters are beam moments of inertia, modulus of elasticity of slabs, and rotational restraint at the supports. The user sets the appropriate boundaries, so that the final optimized variables are within reasonable values. Analytical accuracy is reported in terms of total error, percent error, percent scale error, and correlation coefficient.

TESTING OF BOONE COUNTY BRIDGE #11

Boone County Bridge #11, located in northern Boone County, IA, is a non-composite, simple-span, steel-girder bridge with a timber deck and no skew carrying L Rd. over a small stream one mile North of 130th Street. Based on a cursory visual inspection and photographic documentation, all steel-girders except one appeared to be, with the exception of some light rust, in good condition. The girder on the far West side was bent at midspan (possibly hit by a large object during a flood). The timber deck is in good condition. A photograph of the underneath side of the bridge is illustrated in Fig. 2 This bridge has a span length of 38 ft – 10 in. from centerline to centerline of bearings with a roadway width of 17 ft and an overall width of 19 ft – 9 in. (one 12 ft traffic lane and two 2 ft – 6 in. shoulders).

The timber deck consists of a 4-in. thick wood plank system with a 6-in. gravel overlay without structural connection to the girders. The superstructure is comprised of eight girders and four lines of diaphragms bolted to the girders. The substructure consists of expansion bearings and timber backwalls. The exterior beams and the six interior beams are the same size and are spaced on 2 ft – 6 3/8 in. centers. As shown in Fig. 3b, four of the eight girders were instrumented near the abutments, at midspan, and at quarterspan near the North abutment and two of the remaining four girders were instrumented near the North abutment and at midspan. Each instrumented sections had an Intelliducer installed on the bottom surface of the top and bottom flanges as previously described such that a total of 40 Intelliducers were installed at 20 locations.

A loaded tandem-axle dump truck with a total weight of 49.58 k was used in the tests. Data were collected for three truck paths with two runs conducted for each path. Path Y1 was oriented such that the driver's side wheel line was 8 ft – 11 in. from the far East girder (with the outer wheel line placed 2 ft from the centerline of the East girder), and Path Y2 positioned the truck approximately over the center of the bridge with the driver's side wheel line 11 ft – 11 in. from the East girder. Finally, Path Y3 was oriented with the driver's side wheel line 15 ft – 8 in. from the East girder (the outer wheel line was placed 2 ft from the West girder). Truck path information is summarized in Fig. 3.

FIGURE 2. Photograph of Bridge Superstructure

The experimental data showed that compression was induced in the top flange and tension occurred in the bottom flange near the abutment. This indicates that this bridge has minimal end restraint. The location of the neutral axis was found to be approximately at mid depth of all sections; hence, non-composite action could be verified.

Based on the initial review of the data briefly discussed in the previous paragraph, an analytical model (Model 1) was created using one element between each girder in the transverse direction and twelve elements in the longitudinal direction. Although the experimental data indicate insignificant end restraint, rotational springs were included for all girders at the centerline of the abutment bearings to verify this behavior. As a result of the experimental data indicating that all girders behave non-compositely, the girders in the analytical model were created as one uniform, non-composite section along the length of the bridge.

Table 1 summarizes the optimized parameter results for Model 1 using only the Intelliducers at midspan (L5-L10) and near the North abutment (L15-L20). These results indicate that all optimized parameters (excluding the springs) compare very well with the initially selected parameters. An example of the accuracy of the model is shown graphically in Fig. 4. All results compare well (e.g., less than 2% error with a correlation coefficient of 0.99).

The rating model was created using the optimized model (Model 1) with the appropriate rating trucks and dead load on the structure. Dead load applied to the structure included the self-weight of the steel girders, a 4-in. thick timber deck, a 6 in. x 15 in. wood curb applied on the exterior girders, a weight of 25 lb/ft distributed uniformly over both exterior girders to take into account the steel rail on top of the wood curb, a uniform load distributed over the interior beams to take into account the dead load of the diaphragms, and an additional 6-in. deep gravel overlay on top of the timber deck.

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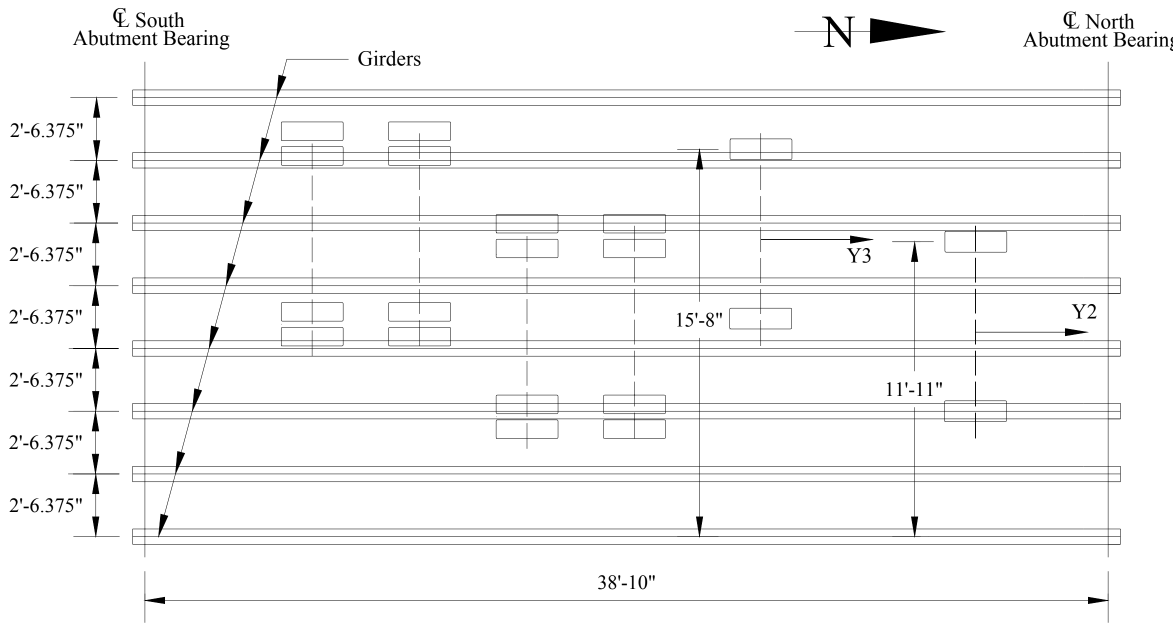
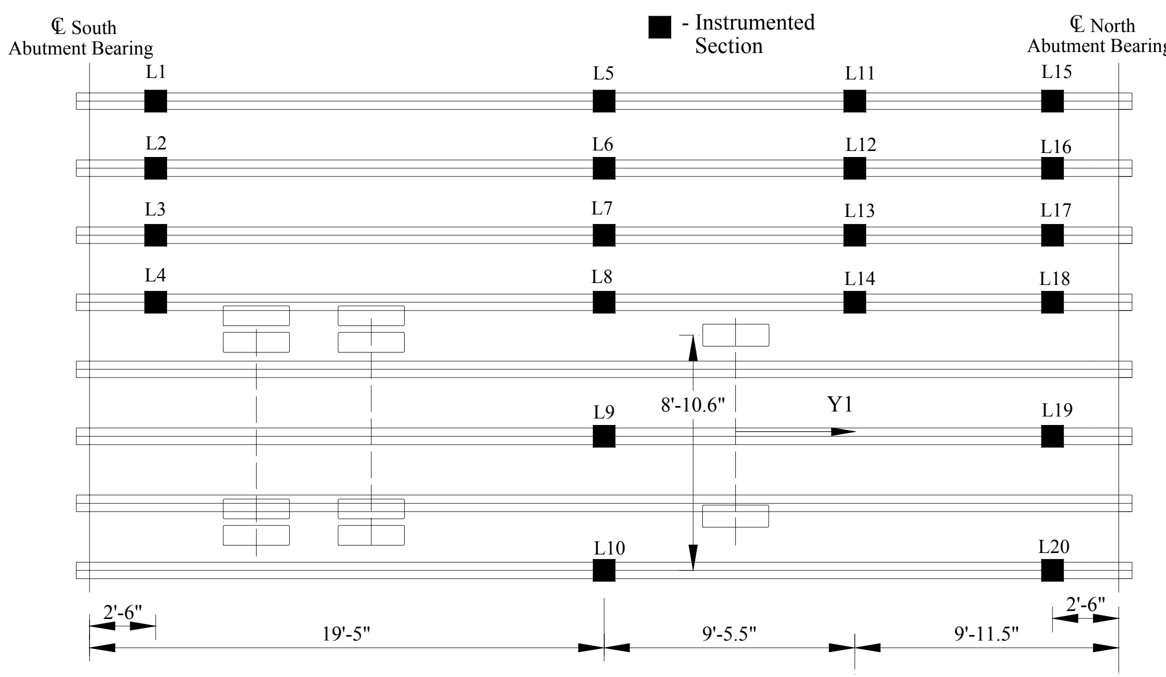


(a) Overall bridge dimensions and truck paths Y2 and Y3.



(b) Truck path Y1 and gage locations.

FIGURE 3. Overall Dimensions, Gage Locations, and Truck paths

TABLE 1. Adjustable Parameters for Model 1

Section	Property	Units	Initial	Optimized
Girder	I_y	in^4	1,230	1,255
Timber deck	E	ksi	1,000	845
Spring (rotational)	K_y	in-k/rad	0	29,210[a]

[a] Corresponds to approximately 8 % fixity.

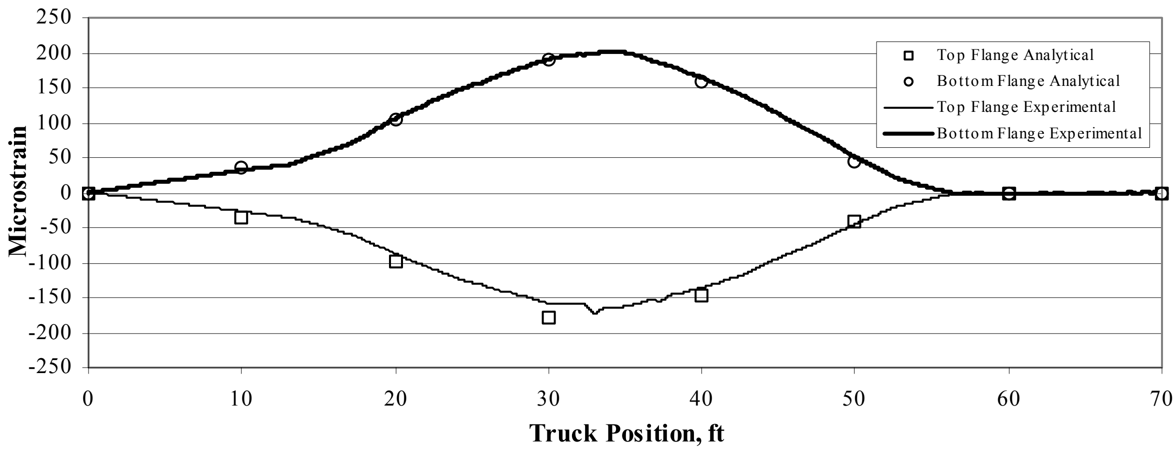


FIGURE 4. Typical Strain Plots for Path Y3 at Location L5 using Model 1

Individual member capacities were calculated following appropriate AASHTO Standard Specifications. Ratings obtained using the LFD Method (by applying AASHTO Standard Specifications) and by using the BDI Software are presented in Table 2. Table 3 summarizes the percent difference in inventory ratings between the LFD Method and the BDI Method (note: a positive percent difference indicates that the BDI Software rating value is greater than the LFD Method rating value). The critical rating condition is for flexure at the interior girder (0.92 by the LFD Method and 1.31 by the BDI Method for a difference of 42 %). It should be pointed out that lane loadings were also investigated in accordance with AASHTO Standard Specifications and were determined to not be critical.

TABLE 2. Rating Factors by the LFD and BDI Methods

Section	HS-20				H-20				Type-3			
	Flexure		Shear		Flexure		Shear		Flexure		Shear	
	Inv.	Ope.	Inv.	Ope.	Inv.	Ope.	Inv.	Ope.	Inv.	Ope.	Inv.	Ope.
	LFD Method											
Interior Girders	0.92	1.53	3.94	6.57	1.16	1.94	5.76	9.62	1.17	1.95	5.32	8.87
Exterior Girders	1.00	1.67	4.22	7.04	1.27	2.12	6.41	10.70	1.27	2.13	5.91	9.87
	BDI Method											
	1.31	2.18	4.78	7.97	1.58	2.64	6.09	10.16	1.75	2.92	6.63	11.06
	1.54	2.57	7.61	12.70	1.97	3.29	11.56	19.29	1.99	3.33	10.37	17.31

TABLE 3. Percent Difference in Inventory Ratings between LFD Method and BDI Software

Section	HS-20		H-20		Type-3	
	Flexure	Shear	Flexure	Shear	Flexure	Shear
Interior Girders	42.4	21.3	36.2	5.7	49.6	24.6
Exterior Girders	54.0	80.3	55.1	80.3	56.7	75.5

As previously described, Intelliducers used in the testing were located near the abutments, at midspan and at the quarter-span near one end. However, only the Intelliducers located at midpan and near the North abutment were included in the optimization process. After the optimized model was developed (based on the limited number of gages), the bridge was analyzed to predict the behavior at the locations not used in the optimization process. The purpose of this was to verify that it is possible to predict strains at locations where no gages are attached. It was found that the predicted strains (example shown in Fig.5) correlate very well with the experimental strains. The model accuracy with all gages included had an error of only 2.1%.

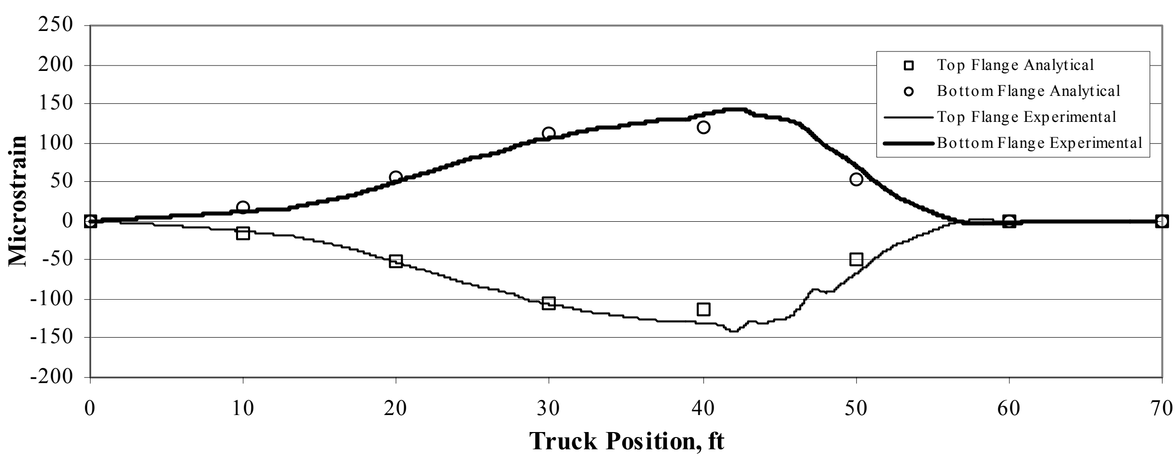


FIGURE 5. Typical Strain Plots for Path Y3 at Location L13 Using Model 1

DISCUSSION AND CONLCUSIONS

The field testing of Boone County Bridge #11 answered many questions about the structure. The girders were found to act non-compositely and that there was very little fixity at the abutments. In addition, it was found that there was very little edge stiffening present. A review of the rating factors revealed that the critical rating factor was 42% higher when determined using the BDI method.

Commercial systems, such as the BDI system, have been found to be effective tools to implement the testing, modeling, and rating of existing bridges. With an aging and rapidly decaying bridge inventory, the effective management of marginal condition structures is becoming a pressing issue. Diagnostic load testing for the purpose of load rating is the only currently available technique for determining accurate load carrying characteristics. Most bridges exhibit strength that exceeds that which traditional calculations predict and results in more accurate and increased load rating. Identification of the "reserve" strength often delays when bridges must be rehabilitated or replaced which results in significant long-term cost savings. While it is recognized that developing load ratings through diagnostic testing costs more than load ratings by traditional hand calculations, the long-term savings resulting from extending a bridge's useful life may offset these costs.

ACKNOWLEDGMENTS

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Process to Identify High Priority Corridors for Access Management Near Large Urban Areas in Iowa Using Spatial Data

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ABSTRACT

Iowa's highways play a dual role of serving through traffic and providing direct access to adjacent land and development. When access via driveways and minor public roads from arterial and collector roadways to land development is not effectively managed, the result is often increased accident rates, increased congestion, and increased delays for motorists.

Although access management is often thought of as an urban problem, some of the most difficult access management issues occur in areas at and just beyond the urban fringe. Fringe areas are the most rapidly developing areas in Iowa. Like most other states, Iowa is becoming more urbanized, with large urban centers accounting for more and more employment and inbound commuting from rural hinterlands. In urban fringe areas considerable commuting occurs inbound to employment centers within the suburban areas and urban cores. Two-lane and four-lane arterials that were originally designed to serve long-distance, high speed travel may also need to serve growing numbers of commuters and sometimes will also have land development and recreational facilities such as trails and parks in placed alongside. Unless access is carefully managed, such highways can lose their effectiveness in terms of serving through travel. They can also become considerably less safe rather quickly.

Iowa has completed and received national attention for its program of access management research. This research project was conducted to assist the Iowa Department of Transportation in systematically identifying "commuter corridors" radiating out from urban areas that are the most likely to need attention in terms of access management. Existing as well as likely future indicators of access management issues are considered. The project focused on four-lane expressways and two-lane arterials most likely to serve extensive commuter traffic. This research used available spatial and statistical data to identify existing and possible future problem corridors with respect to access management. It involved the development of a scheme for ranking "commuter routes" based on their need for attention to access management. To do this, a number of Iowa Department of Transportation, local government, and other data sources were integrated using geographic information systems technology. Sources integrated included crash data, land use data, U.S. Census data, roadway configuration data, traffic data, and remote sensing data (e.g., orthophotography and satellite imagery.)

Key words: access management—crash analysis—geographic information systems—high priority corridors—spatial data

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INTRODUCTION

Access management is a process that provides or manages access to land development while simultaneously preserving the flow of traffic in the surrounding system in terms of safety, capacity and speed. Managing access involves the control of spacing, location, and design of driveways, medians/median openings, intersections, traffic signals, and freeway interchanges. The most common access management problem in Iowa involves allowing a high density of direct driveway access via private driveways to commercial properties located alongside arterial highways, roads, and streets (*1*). Access issues are thought to be a contributing factor in over 50 percent of all highway crashes, however this figure is much higher in built-up urban and suburban areas than in rural areas.

This project was intended to produce a strategy for addressing current and future access management problems on state highway routes located just outside urban areas that serve as major routes for commuting into and out of major employment centers in Iowa. There were two basic goals for the research project. They were to:

1. Develop a ranking system for identifying high priority segments for access management treatments on primary highways outside metro and urban areas.
2. Focus on corridors that are the main commuting routes at present and that will be in the future.

The Iowa Department of Transportation (Iowa DOT) wanted to be able to specifically focus on finding a limited number of four-lane corridors with at-grade intersections (expressways) that ought to be given high priority for pro-active attention to access management based on both current safety problems and future growth in traffic and development.

METHODOLOGY

The research methodology consisted of two distinct activities. The first focused on finding corridors that exhibited signs of having access management problems at present. In Iowa, there are a limited number of routes where capacity and operations are issues. Therefore, this stage of the research focused entirely on safety and safety data. This process involved the following steps:

1. Identify non-freeway, state jurisdiction highway segments that represent likely commuter routes. This was accomplished by examining traffic flows and selecting route segments in close proximity (30 minutes or less travel time) to metropolitan areas and other urban places with 20,000 or more population. A total of 109 segments were identified around the state for further examination.
2. Map the 109 segments into ESRI ArcView geographic information systems (GIS). ArcView was used as the analysis platform for the entire analysis.
3. Gather three years of crash and traffic data for all 109 segments from Iowa DOT databases. Iowa DOT has a very comprehensive, GIS-based crash data system with over 10 years of data and approximately 70,000 crash records per year; this system covers all highways, roads and streets in the state (*2*). For the commuting corridors analysis, data from the years 1997 through 1999 were used. Year 2000 data were not yet available for use and 2001 data were still being compiled.
4. Query out those crashes that could potentially involve access points, in particular all crashes that involved left-turning and right-turning vehicles. This was done to avoid including crashes such as animal/vehicle crashes, snow and ice-caused crashes, and single vehicle run-off the road crashes. These are common sorts of crashes in exurban and rural areas, but are not primarily caused by access management problems.
5. Develop four ranking indicators for the corridors that took into account crash frequency, crash

rate, crash severity, and the percentage of total crashes that might be access-related.
6. Calculate indicator statistics for each segment.
7. Rank the segments based on the four indicators.
8. Develop an index for each indicator where 1.00 equaled the highest-ranked segment.
9. Identify the Top 25 segments for each indicator.
10. Develop a listing of segments that ranked high on four, three, and two indicators as a preliminary composite ranking tool.
11. Calculate and assign composite rankings to each of the 109 corridors based on anticipated savings in crashes (and losses) from applying access management treatments.
12. Conduct an illustrated review of each of the Top 25 ranked corridors to identify specific locations where access management issues exist and potential treatments that could be used to mitigate current and future problems. These reviews were completed using a combination of digital video log data and windshield surveys. The Iowa DOT maintains and updates a complete digital video log of its system every other year; the video log system is a very useful tool for access management studies in that it provides a way to view intersections and driveways on all state routes while remaining in the office.
13. Complete an analysis of other corridors that do not currently appear have access related safety problems but that could in the future due to a combination of high forecast commuting traffic growth, proximity to a fast-growing urban area, a high density of private drives and public road intersections, and a low classification on the Iowa DOT access management classification system. (The Iowa DOT uses an access management priority classification system that runs from 1 to 6 where 1 is a completely access-controlled freeway or Interstate and 6 is a route that mainly serves local traffic. Higher driveway densities are allowed on routes that are assigned higher numbers.) This last analysis was conducted so the Iowa DOT districts could work on access management on a pro-active basis rather than wait for safety problems to develop before acting.

The research team and Iowa DOT safety and access management staff met to judge whether the results of step 10 appeared reasonable used this preliminary composite ranking. It was decided that the routes that ranked high were indeed likely to be routes with access management problems. Once the general validity of the results was established, a method of combining the rating factors into a Composite Ranking was developed.

The second major portion of the research involved the development of a statewide commuter traffic model. This was accomplished using an ArcView/TRANPLAN interface that the Center for Transportation Research and Education developed for an earlier Federal Highway Administration funded project (*3*). This model has a number of other potential uses for the Iowa DOT, but was constructed for this project in order to produce estimates of future commuting traffic growth or decline on the 109 commuter routes being analyzed. The statewide model has the following attributes:

- 2940 zones (based on U.S. Census block groups); this is about one zone for every 1,000 persons in Iowa, with about 30 zones per average-sized county in Iowa. The zonal structure is small in metropolitan areas and large in sparsely populated rural areas.
- Trip productions were based on population estimates and forecasts.
- Trip attractions were based on employment estimates and forecasts.
- The model used a 1999 base year and a 2004 forecast year.
- The data source for the productions and attractions was Geolytics Incorporated's CensusCD+Maps Version 3.0 product, which provides Census data and forecasts by Census Tract and Block Group.
- The main focus of the model was on estimating growth and decline in trips, not accuracy in estimating actual trips.

The model network included all Primary (state jurisdiction) and some secondary and municipal roads where needed to fill out the network. Average travel speeds were set at 50 miles per hour on most links in the network, but 30 miles per hour on local roads (connectors) and 65 miles per hour on Interstates. Friction factors for the model were borrowed from the Des Moines Area Metropolitan Planning Organization's model but extended to provide for a practical maximum trip length of 100 minutes. Multi-state flows were not modeled and there were no "external" zones, so absolute traffic estimates in the model are likely to be inaccurate in areas near the state borders. An "all or-nothing assignment" and visual validation were used.

RESULTS

The following ranking indicators were used to initially identify high priority corridors:

- Crash Frequency—This indicator represents the number of crashes that appear to be access-related, in particular those that involve turning vehicles. All turning crashes were included, whether they occurred at private driveways or public road intersections.
- Crash Rate—This indicator is the frequency of access related crashes per million vehicle miles traveled (VMT).
- Crash Loss/Severity—This indicator measures the estimated cost of access related crashes in dollars, including an estimate of the cost of fatalities, personal injuries, and property damage.
- Percentage of Total Crashes That Are Likely Access Related—This indicator represents the percentage of total crashes that appear to be access-related.

The distribution of ranking indicators was compiled for all 109 corridors on each of the four indicators. Access-related crash frequency ranged from a high of 529 over a three- year period down to zero. The mean frequency was 60. Access-related crash rates ranged from a high of 5.61 per million vehicle miles traveled down to zero. The mean rate was 1.21. Access-related crash losses ranged from a high of $43.5 million down to zero, with a mean loss of just over $5 million for a three-year period. The percentage of crashes deemed to be related to access ranged from a high of 33.3 percent to a low of zero percent; the mean value for this indicator was 10.6 percent. The 109 corridors being analyzed are located primarily outside built-up urban areas. If similar percentage calculations were conducted inside urban areas, it is very likely that these percentages would be significantly higher.

Results from the statewide model were used to identify those corridors that should be expected to experience significant changes in commuting travel in the future.

Table 1 is a typical ranking table; in this case, the top 25 corridors in the state based on the percentage of crashes that appear to be access-related is presented. Figures 1 through 4 are maps of the four basic rankings and show all 109 corridors broken down roughly into quartiles. Seven of the 109 corridors ended up ranked in the top 25 on all four indicators; another five corridors ranked in the top 25 on at least two corridors, not counting those that ranked in the top 25 on only frequency and loss.

TABLE 1. Example Corridor Ranking Table for Top 25 Corridors Based on Percentage of Access-Related Crashes

COMM_ID	PERCENTACC	RANK	INDEX
85	33.3%	1	1.00
43	30.2%	2	0.99
71	29.6%	3	0.98
51	27.5%	4	0.97
73	26.8%	5	0.96
42	26.7%	6	0.94
92	26.7%	6	0.94
107	23.7%	8	0.93
32	22.9%	9	0.93
49	22.6%	10	0.92
76	21.9%	11	0.91
7	21.2%	12	0.90
70	20.8%	13	0.89
72	20.8%	14	0.88
4	20.5%	15	0.86
39	20.5%	15	0.86
24	20.0%	17	0.84
54	20.0%	17	0.84
22	18.9%	19	0.83
66	18.8%	20	0.82
75	18.6%	21	0.81
59	18.6%	22	0.80
67	17.9%	23	0.79
34	17.5%	24	0.79
9	16.7%	25	0.78

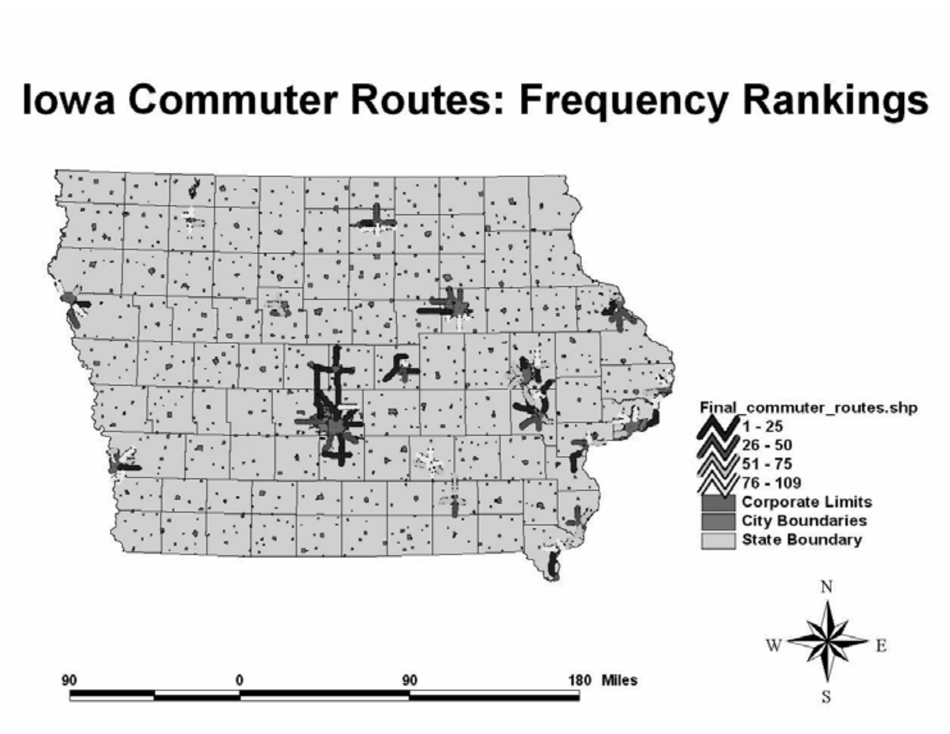

FIGURE 1. Statewide Map of the Corridors Based on Their Rank in Terms of Frequency of Access Related Crashes

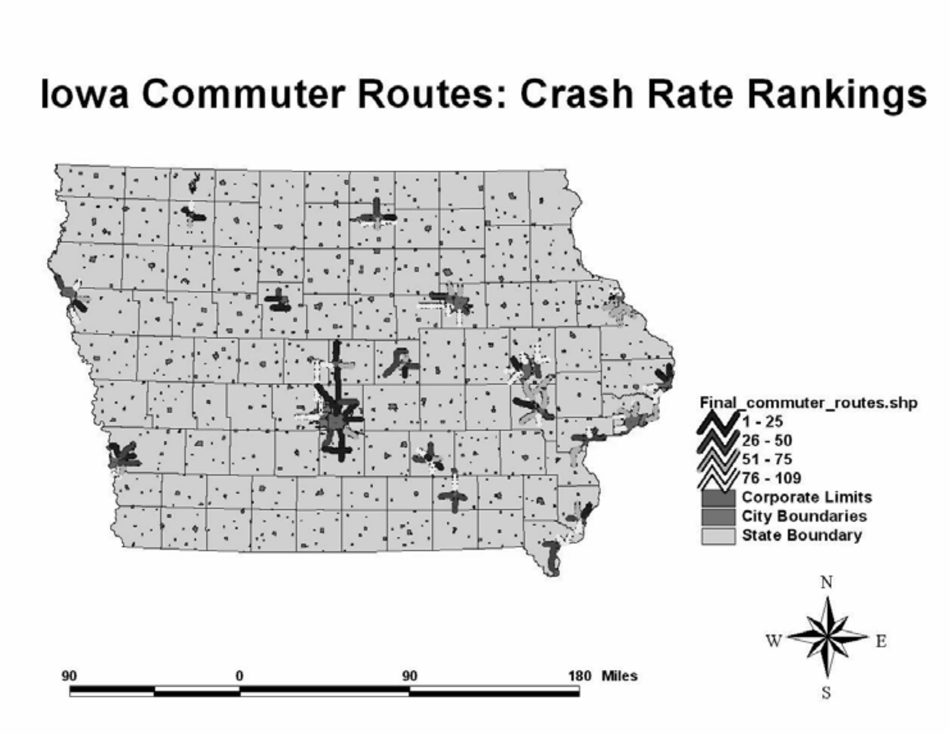

FIGURE 2. Statewide Map of Corridors Based on Their Rank in Terms of the Rate of Access Related Crashes

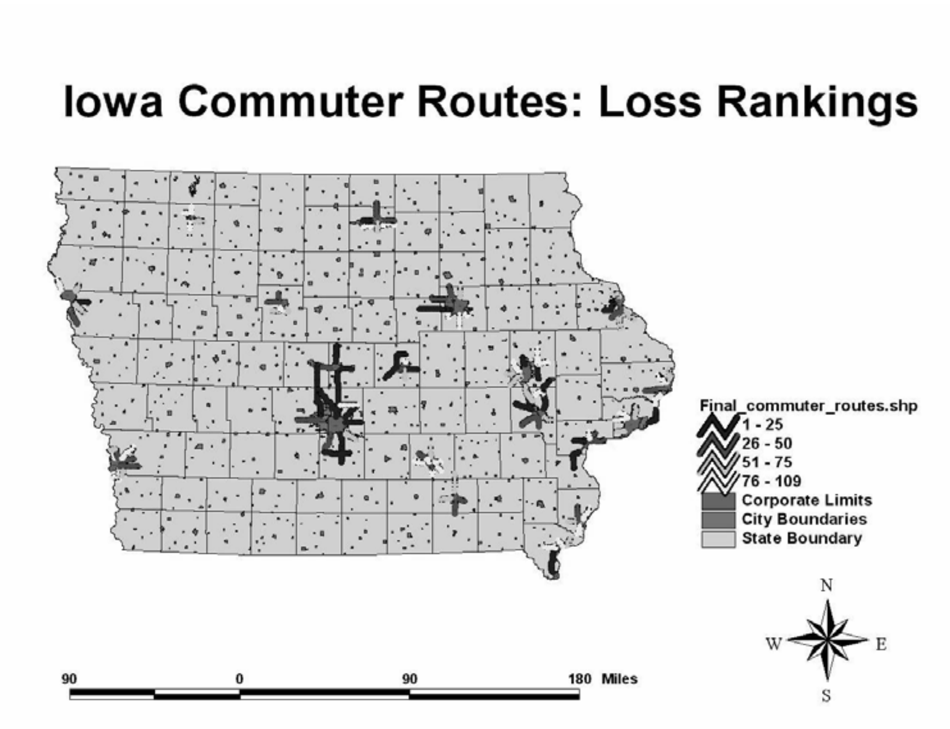

FIGURE 3. Statewide Map of Corridors Based on Their Rank in Terms of Losses from Access Related Crashes

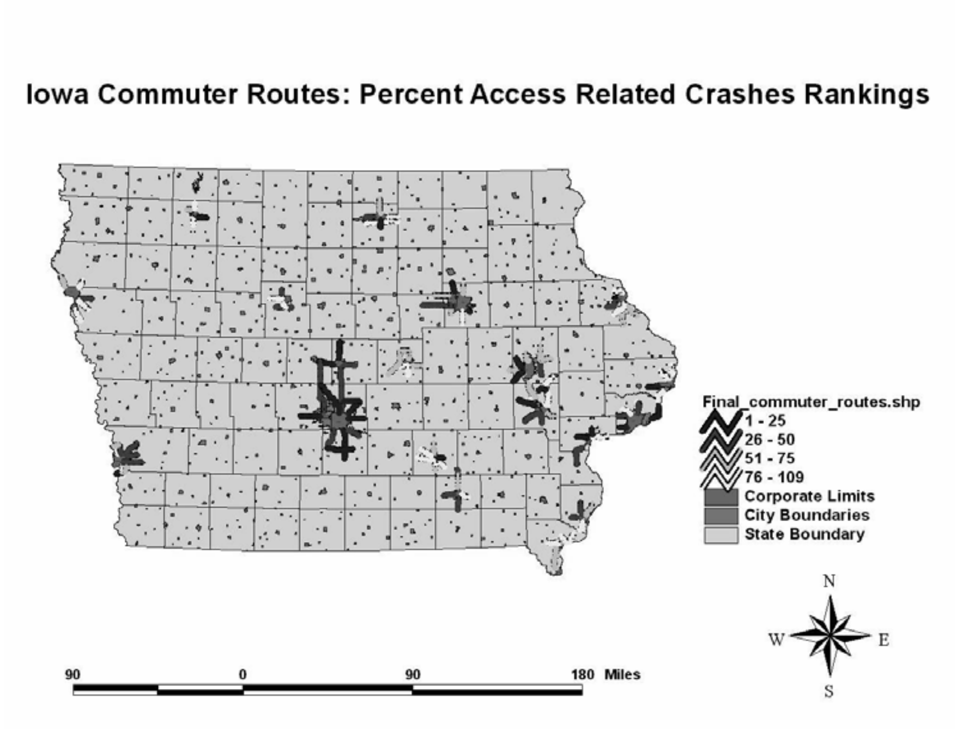

FIGURE 4. Statewide Map of Corridors Based on Their Rank in Terms of Percentage of Access Related Crashes

Figure 5 provides an illustration of long-term population growth forecasts by county in Iowa produced using data from a private forecasting company, Woods and Poole Economics. Future growth in the state is expected to be largely concentrated in two regions: Des Moines/Ames and Cedar Rapids/Iowa City. Figure 6 shows the mapped results for the base year commuter flow estimate for Iowa from the statewide model. The heaviest concentrations of work travel are in central Iowa (near Des Moines and Ames) and east-central Iowa (around Cedar Rapids and Iowa City). Figure 7 provides an indication of absolute 1999 through 2004 change in commuting forecast by the model. Change ranges all the way from negative on a few routes in isolated rural areas to almost 2000 trips per day increase on a few routes near Des Moines. Figure 8 provides a map of forecast percentage change in average daily commuting traffic. Change from 1999 to 2004 ranges from negative six percent on a few rural routes to almost 20 percent on other routes. The largest percentage changes are actually found outside metropolitan areas near some key rural employment centers that attract many commuters from the surrounding countryside.

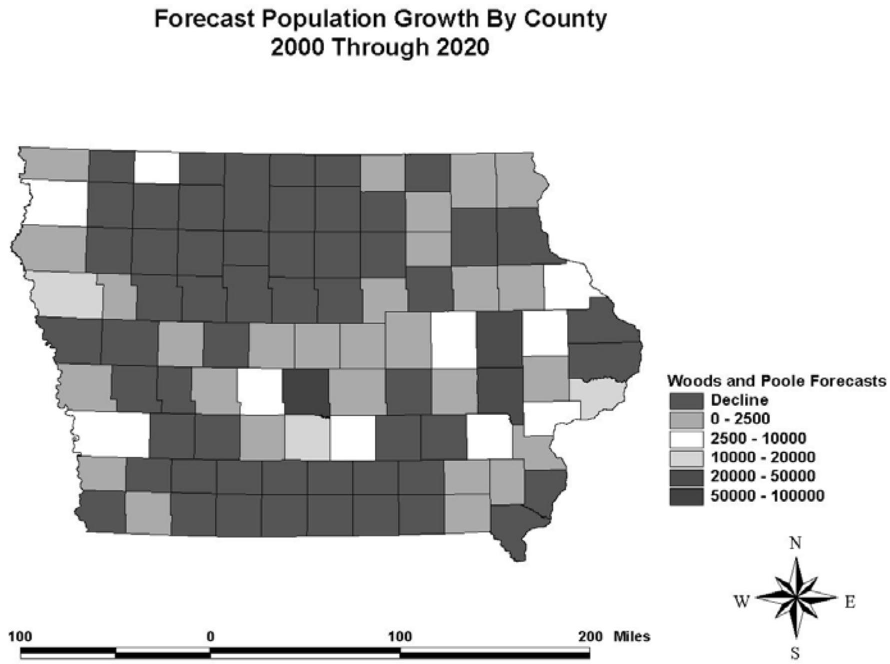

FIGURE 5. Year 2020 Population Forecast Map Produced Using Data from Woods and Poole, Inc.

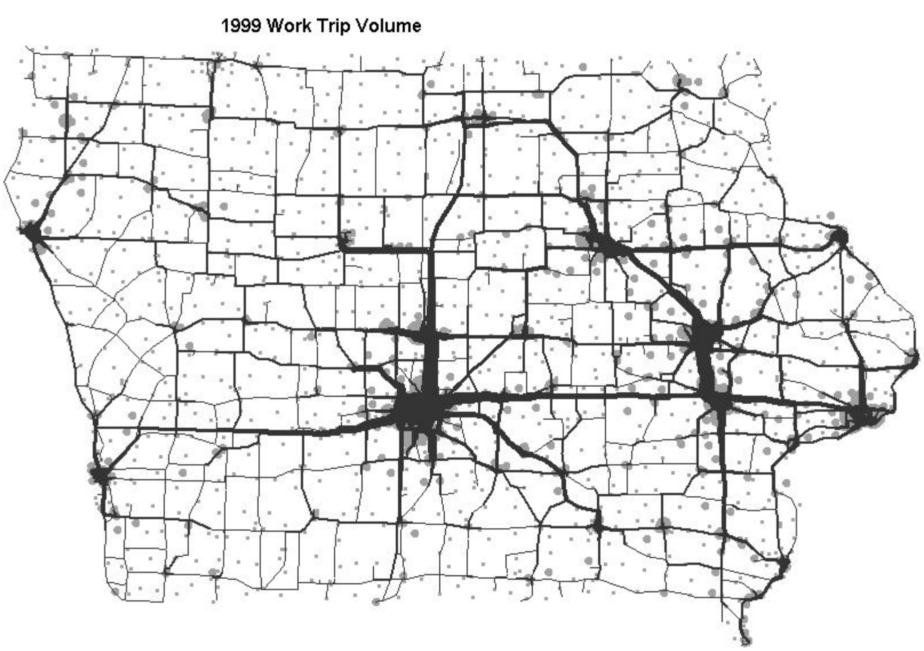

FIGURE 6. Statewide Model Commuting Travel Estimates for Base Year 1999

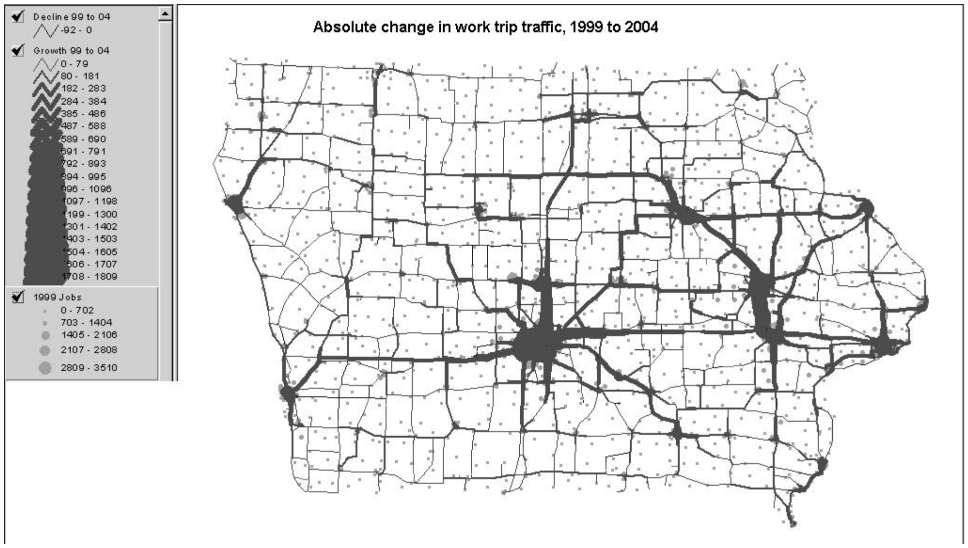

FIGURE 7. Statewide Model Estimated Change in Commuting Travel, 1999–2004

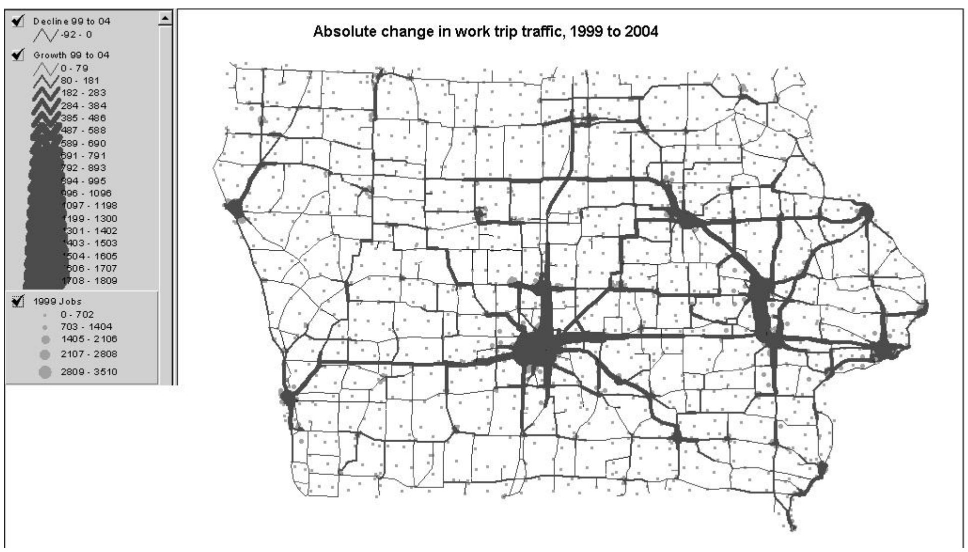

FIGURE 8. Statewide Model Estimated Percentage Change In Commuting Travel, 1999–2004

Some overall findings of the analysis were as follows:

- Frequency and loss were (as might have been expected since loss is partially a function of crash frequency) highly rank-correlated; because of this, both indicators were not used together in developing final composite priority rankings
- Most of the highest ranked routes were on two-lane rural cross-sections, but a few were four-lane expressways with at-grade private driveways and public road intersections
- Most of the highly ranked corridors in terms of current safety problems were in Central Iowa, near the Des Moines metropolitan area and Ames; this region is the fastest growing region in the state in terms of both population and employment. In fact eight of the twelve corridors that most consistently showed up in the rankings were in this region. The corridors in this region are also forecast to have the most future growth in commuting activity. The Cedar Rapids/Iowa City region had the second largest concentration of problem corridors, accounting for two more corridors. Only two corridors that consistently topped the rankings were from other parts of the state.

APPLICATION TO POLICY AND PRACTICE

To take a more preventive measure towards access-related crashes, the corridors were analyzed to determine if access-related crashes would decrease from a hypothetical roadway treatment to better manage access. For this, a "potential improvement" factor was created, using, among other data, forecast traffic growth data generated from the travel demand model.

The calculations for potential improvement were performed as follows:

- Assume the access-related crash rate will stay the same if no changes to the roadway are made.
- Assume that 50 percent of access-related crashes could be avoided by making improvements in access control. This figure is based on typical results found in Iowa through "before and after" access management safety case studies (*1*).
- Use the percentage annual average daily traffic (AADT) growth rate forecast in the travel demand model to create a VMT Factor, using 20 years (also a variable) as the analysis time period. Example: If annual growth rate is 2%,
  - VMT Factor = $(1.02)^{20} = 1.48$
  - $1 + (1.48-1)/2 = 1.24$ (average)
- Compute the number of expected access-related crashes during the 20 years if there were no road treatment. Example:
  - = 20 * VMT Factor * Access Crash Rate * VMT
  - = 20 * 1.24 * 5 * 36 = 4464 crashes
- Multiply the expected number of access-related crashes with no treatment by 50 percent to calculate the expected number of access-related crashes after road treatment.

The corridors were then ranked by potential improvement, or the reduced cost of access-related crashes with road treatment.

The potential improvement factor development process identified the number of expected future crashes for each corridor, along with the expected number of future crashes for each corridor with roadway treatment to ease access problems plus a crash loss reduction in dollars. These corridors should represent the best opportunities in Iowa for pro-active access management strategies, for example master corridor management agreements between the Iowa DOT, municipalities, and counties.

The “pro-active corridors” were ranked by their potential for improvement in terms of future access-related crash loss reduction; the top 25 ranked corridors are represented in Figure 9. Once again, the highest “potential improvement” or “pro-active” corridors turned out to be highly concentrated in a few geographic areas. For example:

- The top 25 corridors for potential improvement contain all eight of the commuter route corridors near Des Moines; this is a strong signal that most Des Moines commuter routes could significantly benefit from road treatments to ease access management problems
- The top 25 corridors for potential improvement also contain all five of the commuter routes near Ames. Other commuter destination cities in Iowa have no more than two routes each in the top 25 ranked corridors by potential improvement; it is clear that both the Ames and Des Moines areas could both greatly benefit from road treatments aimed at managing access
- The Ames and Des Moines commuting regions together contain seven of the top ten corridors ranked by potential improvement. Many of the highest ranked corridors on this indicator are in a single Iowa DOT district (the most urban and fastest growing district), indicating that a corridor management strategy for that district might be a high policy priority.

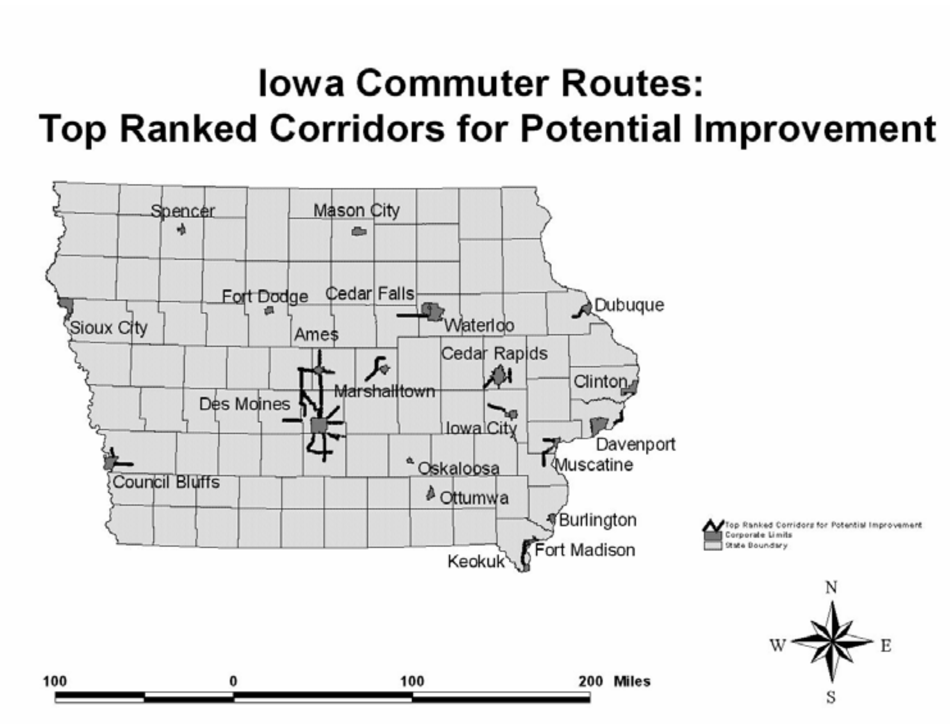

FIGURE 9. Top Ranked Corridors for Potential Improvement (“Proactive Corridors”)

The final report of this project has been published, and the findings of this project led to a follow-on research project designed to develop a template for corridor management studies and cooperative agreements in Iowa. As of the writing of this paper, two pilot corridor management projects are underway near Dubuque (U.S. Highway 20) and Des Moines (Iowa Highway 163). These corridors were among the highest-ranked “pro-active” corridors identified in the research project.

ACKNOWLEDGMENTS

The Iowa Department of Transportation Office of Traffic and Safety provided funding for this project. The results presented in this paper are preliminary in nature; a final report for the project is being reviewed by the Iowa DOT and should be published during 2002. Tom Welch and Larry Heintz of the Iowa DOT provided valuable input on traffic safety matters and Iowa's access management policy related to this project. Zach Hans of the Center for Transportation Research and Education provided valuable assistance in combining and using spatial data. CTRE Transportation Research Specialist Randy Boeckenstedt also assisted with the analysis. Several Iowa State University undergraduate and graduate students played important roles in this project. Lee Edgar coded the statewide model used to estimate and forecast future commuting flows. Kyle Kosman and Jamie Luedtke performed the GIS analysis work and created thematic maps used in presenting the findings.

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Iowa’s Statewide Urban Design Standards Promote Improved Access Management

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ABSTRACT

In the late 1980s, the majority of local units of government and the water works in central Iowa formed a committee to develop a common set of design standards and specifications for public works infrastructure to be used throughout the metro area This common set of standards included standard drawings and specifications for such things as storm sewers, sanitary sewers, and urban streets. The City of Des Moines and a local engineering and planning consulting firm, Snyder and Associates, coordinated the project and developed the manual with guidance from an advisory committee comprised of engineers from the member cities. The common set of standards helped the member cities realize lower bids from contractors, who were now able to design and build to the same standards no matter what community they were working for.

In the late 1990s, a complete update of the *Urban Design Standards Manual* was undertaken. At that time, the 34 central Iowa member cities decided to dedicate an entire chapter of the manual to access management standards for urban streets. This decision was reached partly as a result of the findings of research and technology transfer work conducted by the Center for Transportation Research and Education (CTRE) at Iowa State University and sponsored by the Iowa Department of Transportation; the research indicated how valuable managing access could be in terms of improving the safety of major urban streets in Iowa.

The new access management chapter in the *Urban Design Standards Manual* included material regarding general access management concepts and definitions, access permitting, entrance types, conflict points, driveway spacing, driveway design guidelines, turning lane and two-way left-turn lane guidelines, internal circulation design guidelines for commercial developments, and a section on access management and pedestrian and bicycle safety. A second chapter on traffic impact studies also includes material related to access management. Major sources of information for the new chapter were the Iowa State University/CTRE materials, the Iowa Department of Transportation access management standards for state primary highways, and the National Highway Institute access management course notebook.

Iowa is now taking the *Urban Design Standards Manual* statewide. Iowa will be the first state to have a set of statewide urban design specifications. A committee has been established and an expanded intergovernmental agreement has been drafted to allow additional cities and metropolitan areas to adopt the standards. CTRE has taken on the role of coordinating and keeping the standards manual updated. As new communities and metro areas come on board via the intergovernmental agreement, cities in Iowa will effectively adopt a uniform set of access management guidelines for its city street system.

Key words: access management—urban design standards

Proceedings of the 2003 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2003.

INTRODUCTION

For almost 25 years, municipalities in Iowa have been working together on a set of shared urban design standards and specifications for public works infrastructure. This effort was designed to lower letting costs in the largest metropolitan area in Iowa for such items as arterial street construction. In 1995, a Governor's Blue Ribbon Task Force report recommended that this concept be expanded to a statewide set of urban design specifications as a cost-saving measure. From 1998 through 2000, this idea gradually became a reality and the Iowa Statewide Urban Design and Specifications (SUDAS) project took shape.

SUDAS has ultimately become an ideal vehicle for the promotion of best practices in urban street design, such as the incorporation of the principles of access management. Lessons learned by the Iowa Department of Transportation (Iowa DOT) and local jurisdictions through years of access management planning, research, and project experience are being incorporated into the SUDAS manuals. The benefits of this approach are now being realized in terms of lower cost urban street projects that are engineered better and which recognize the careful balance that has to be maintained between serving through traffic and providing direct access to land development.

SUDAS BACKGROUND AND HISTORY

Starting in 1988, the City of Des Moines and neighboring jurisdictions developed a Central Iowa Urban Design Specification administered through Iowa Code 28E intergovernmental agreements. By 1999, the Central Committee Standards had grown to 35 units of government with nearly a half a million residents. The initial Central Iowa program evolved to provide a foundation for the SUDAS documents.

In 1995 the Iowa Governor charged a Blue Ribbon Task Force with finding better methods to maximize the benefits derived from the Iowa Road Use Tax Fund (RUTF). The task force recommended that Iowa adopt common standards for construction specifications and construction equipment to increase project bidding and lower costs (*1*).

In 1998, a statewide steering committee was formed that included representatives from the Iowa chapter of the American Public Works Association, Iowa County Engineers Association, Consulting Engineers Council of Iowa, and the Iowa DOT, as well as contractors and other stakeholders throughout the state. The steering committee completed a feasibility report in 1999 that recommended the Center for Transportation Research and Education (CTRE) develop and staff a program to adapt the Central Iowa Specifications to Statewide Urban Design and Specification documents.

In 2000, the Iowa DOT provided a grant that allowed CTRE to begin work. Metropolitan Planning Organizations (MPOs) and Regional Planning Affiliations (RPAs, which are the transportation planning organizations for parts of Iowa outside metropolitan areas) provided matching funds of 60 percent to the Iowa DOT grant. The City of Des Moines contributed much of the preliminary legal work necessary to create the organizational structure and licensing agreement. The Iowa DOT also supplied additional legal assistance and ongoing technical help through its Office of Design. The Central Iowa Committee granted CTRE license to use and update the documents for the entire state; copyright will eventually be handed over to CTRE and its parent, Iowa State University.

District committees were formed for each of the six Iowa DOT Districts to review existing manuals and recommend changes to bring them up to the "state of the practice." Before adoption, all standards and specifications are subject to approval by a majority vote of the Statewide Steering Committee.

During 2000 and 2001, CTRE staff visited the 23 metropolitan planning organizations and regional planning affiliations in the state at least twice explaining the program and obtaining funding resolutions. During 2002, 49 meetings were held with these groups to exchange ideas and solutions, and consider recommended amendments. To make SUDAS possible, over 200 city, county, and Iowa DOT engineers, plus other state, federal, and industry representatives contributed untold hours of expertise in an environment of collaboration and compromise.

In early 2003, the first edition of the finalized amendments to the *Iowa Statewide Urban Standard Specifications and Statewide Urban Design Standards Manuals* were issued (*2*).

ACCESS MANAGEMENT AND SUDAS

Access management has quickly become a significant component of the Statewide Urban Design Standards. Access management is a process that manages access to land development while seeking to preserve the flow of traffic on the surrounding road system. Sound access management practices can lead to safer roads that also provide better service to motorists.

In 1996, the Iowa DOT established an Access Management Task Force as part of its umbrella Iowa Safety Management System (Iowa SMS) development effort (*3*). The task force worked with CTRE at Iowa State University and other researchers at the University of Northern Iowa to improve awareness of how access management could lead to safer and better functioning highways, roads, and streets in Iowa. The task force and research team also investigated the effects that access management projects have on local business vitality. The results of this research were very supportive of an expanded access management program in Iowa (*4*) For instance, the research showed that case study roadways had crash rates that were 40 percent less on average following access management projects. At the same time, the vast majority of businesses were found to be unaffected by access changes.

In order to encourage wider adoption of access management principles in Iowa by state, city, and county roadway officials, the task force commissioned a variety of outreach tools, including the following:

- a website
- presentations at numerous conferences in and near Iowa
- a statewide conference, a videotape, a set of brochures
- a guidebook
- a set of answers to frequently asked questions about access management for use in public and stakeholder involvement situations

In combination, these efforts greatly increased awareness of access management and the benefits and impacts of incorporation of access management treatments. Many of these documents are available electronically on the websites listed in the references for this paper.

As SUDAS was developed, the SUDAS Steering Committee and the consulting firm then managing the development and update of the *Statewide Urban Design Standards Manual* identified the need for a section or an entire chapter on access management. This effort was closely coordinated with the Access Management Task Force's outreach materials so that a consistent message would result. For example, the Manual borrowed a considerable amount of material from the *Iowa Access Management Handbook* (*5*). Several national sources were also used the development of the access management chapter. In particular, the National Highway Institute (NHI) training course notebook on access management and materials from leading states such as Florida, Colorado, New Jersey, and Oregon were used to add content to the SUDAS chapter on access management (*6*).

The current version of the *Statewide Urban Design Standards Manual* contains a chapter on traffic impact studies and site impact analysis that complements the 44-page access management section. The SUDAS access management materials are better termed guidelines or suggestions rather than outright standards. The access management chapter in the *Statewide Urban Design Standards Manual* includes extensive material regarding the following:

- general access management concepts and definitions
- access permitting processes in Iowa
- driveway entrance types
- conflict points and their importance to traffic safety
- driveway spacing and corner clearance
- driveway geometrics and design guidelines
- turning lane and two-way left-turn lane guidelines
- internal circulation design guidelines for commercial developments
- access management impacts on pedestrian and bicycle safety

This material can be obtained as a hard copy, on a CD-ROM, or be downloaded from the World Wide Web (http://www.iowasudas.org/). The web-based version is free; the paper and CD-ROM versions are made available at a modest cost. The appendix to this paper shows examples of diagrams and tabular material from the *Statewide Urban Design Standards Manual* related to access management.

There have turned out to be several major benefits of having the access management material present in the SUDAS documents. These are that:

- Access management principles are available in one place for hundreds of local governments in Iowa plus their engineering consultants. When projects are being designed with SUDAS, they are more likely to incorporate sound access management principles since local governments and design consultants throughout Iowa will be using the standards as a guide. This improves the functionality and safety of designs.

- Access management principles are now considered early in the design process since SUDAS is the standard reference that engineering and planning firms and local governments will consult first.

- The access management materials in SUDAS are available for city engineering staff to use in educating stakeholders such as city council members and for city planning staff to use in educating planning and zoning commissioners about the benefits and importance of access management since a variety of material has been included that answers the questions that commonly arise when designs and access-related features are being considered.

- The access management materials in SUDAS are adaptable for use in stakeholder education and involvement. SUDAS provides more depth on design issues than other public involvement materials for access management that are used in Iowa.

Although no formal studies of usage have been conducted, there is ample anecdotal evidence to show that the SUDAS access management materials are well used in Iowa. The SUDAS staff indicates that a number of suggestions to improve and expand on the access management chapter have been received.

There have also been complaints to the effect that the access management guidelines are too lenient or unreasonably tough. Desirable corner clearance distance has been a particularly well-discussed area of the guidelines.

FUTURE DIRECTIONS FOR SUDAS

SUDAS is about to undergo a major change. The contents are being reviewed and completely updated so that they may be used statewide in Iowa rather than just in the Des Moines metropolitan area. Completion of the SUDAS documents update for statewide use is expected during 2004 with regular annual updates starting shortly thereafter. Fifty meetings are scheduled around the state during 2004 to develop amendments. Until the statewide manual updates are finalized and the copyright is transferred to Iowa State University/CTRE, local jurisdictions can use the existing central Iowa design and specification manuals distributed by CTRE, with the addenda of changes.

These common standards and specifications are expected to develop design, construction, and contracting uniformity among communities. By reducing bias toward particular materials or methods and encouraging additional bids, the annual statewide savings for Iowa in Iowa RUTF dollars is estimated at nearly $9 million.

Equally important, the statewide standards will provide a central means of transfer for emerging technologies and tested processes. CTRE will keep the manuals up to date, relieving individual communities of that burden and keeping cities informed about new procedures.

For instance, content revision represents an opportunity to incorporate new knowledge about access management. For instance, the newly released *National Access Management Handbook* will be available to help improve the access management chapter in SUDAS. In addition, comments that have been received by users of the initial version will be considered prior to modifications being issued. A particularly challenging issue being addressed as this paper is written is the accommodation of Americans with Disabilities Act sidewalk and crossing design provisions in the access management section of SUDAS.

CONCLUSION

When the SUDAS updates are completed in 2004, Iowa will be one of only a few states with border-to-border uniform urban design and specification standards. Several other states are closely monitoring Iowa's process, products, and results. The SUDAS process represents an excellent opportunity for the transportation design community to jointly promote and adopt important concepts such as access management.

APPENDIX: EXAMPLE FIGURES AND TABLES FROM THE IOWA STATEWIDE URBAN DESIGN AND SPECIFICATIONS MANUAL (CHAPTER 5, SECTION 5, ACCESS MANAGEMENT) (*7*)

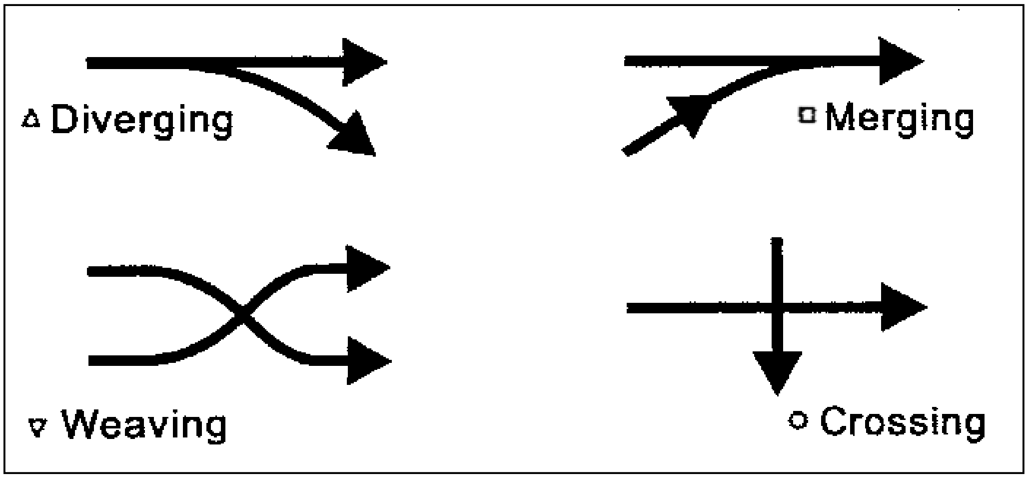

FIGURE 1. Graphic Explaining Conflict Point Terminology

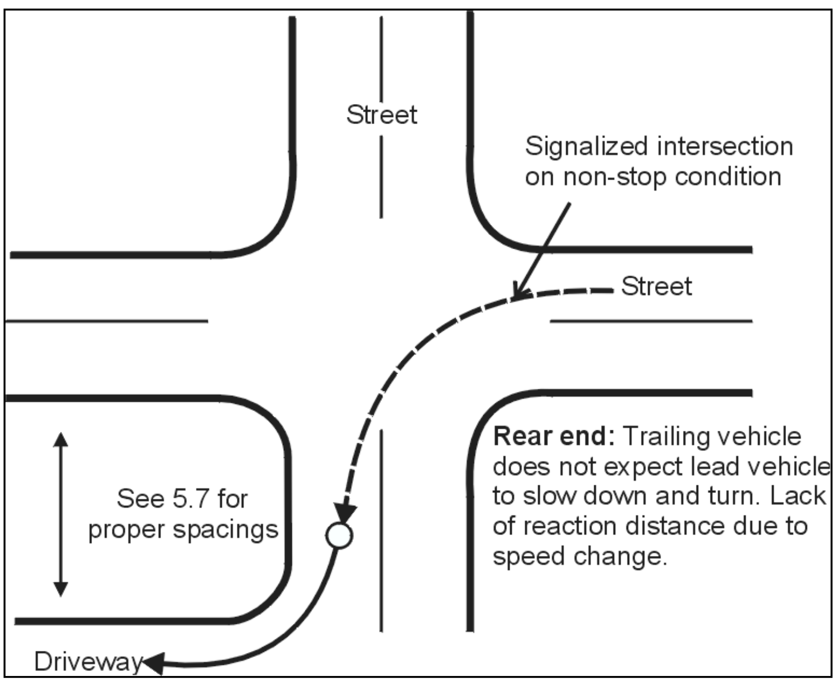

FIGURE 2. Graphic Explaining Driveway Corner Clearance

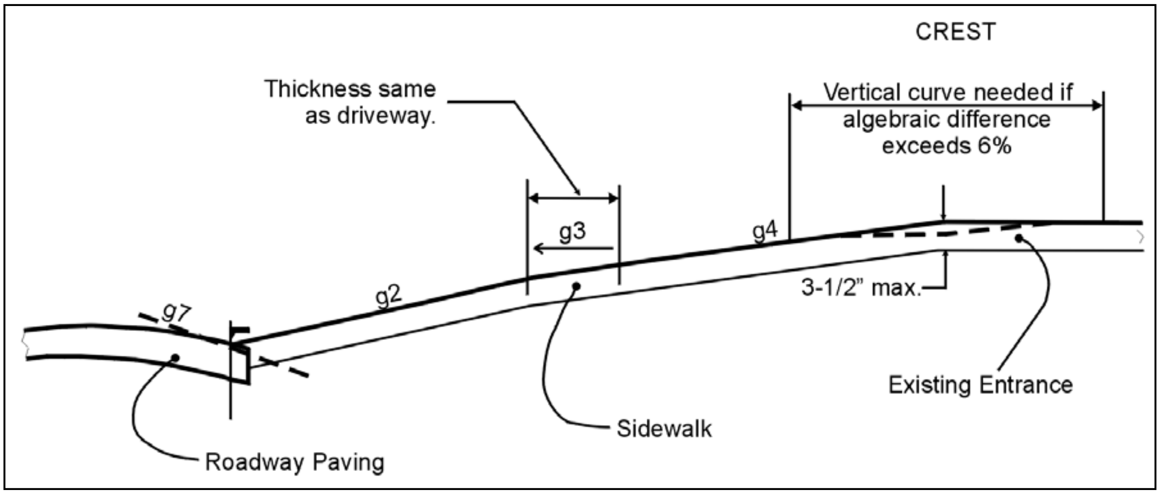

FIGURE 3. Graphic Explaining Commercial Driveway Grade Guidelines

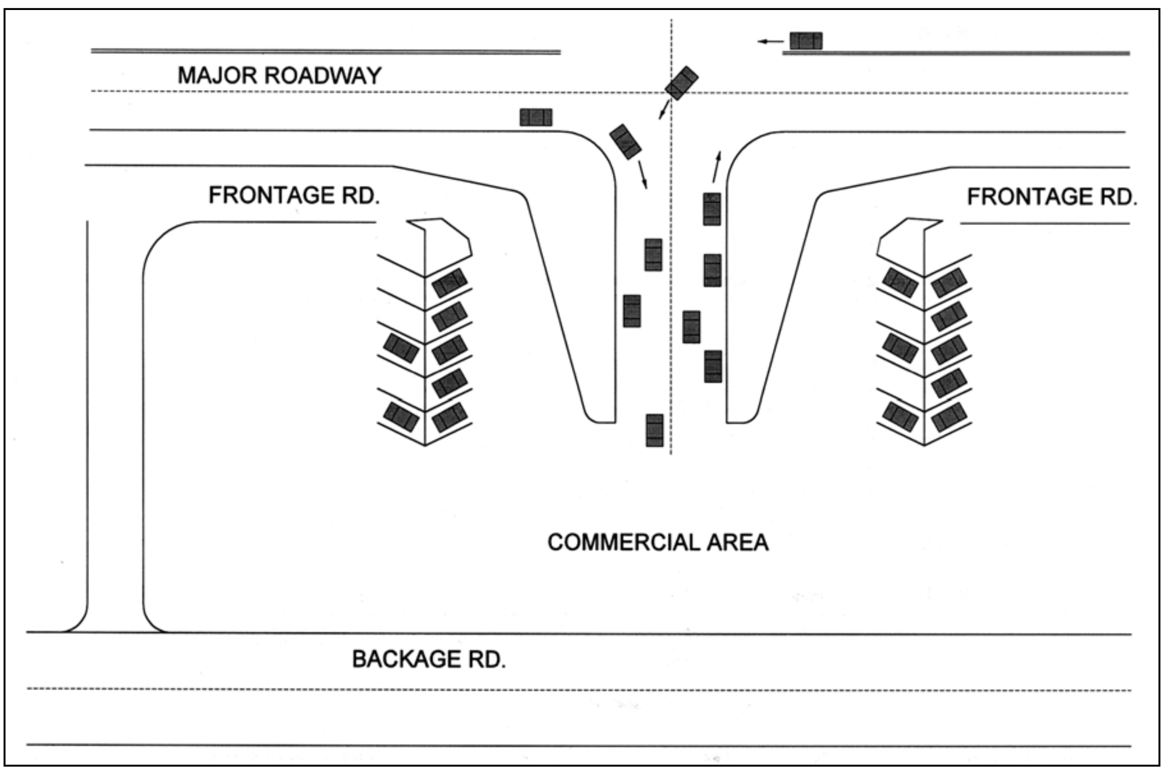

FIGURE 4. Graphic Explaining Internal Circulation and Adequate Driveway Throat Length

TABLE 1. Safety Impacts of Excessive Direct Access

Access Points per Mile	Undivided (Painted Centerline) Crash Rate (per MVMT)	TWLTL Crash Rate (per MVMT)	Raised Median Crash Rate (per MVMT)	Rate Reduction Raised Median vs. TWLTL
Less than 20	3.8	3.4	2.9	–0.5
20 to 40	7.3	5.9	5.1	–0.8
40 to 60	9.4	7.4	6.5	–0.9
Over 60	10.6	9.2	8.2	–1.0

Note: MVMT = million vehicle miles traveled; TWLTL = two-way left-turn lane.
Source: *National Cooperative Highway Research Program Report*.

TABLE 2. Relationships between Access Management Treatments and Pedestrian Safety

Roadway Type	Median	Mid-block Pedestrian Crash Rate	Intersection Pedestrian Crash Rate
Undivided four lane	None	6.69	2.32
Five lane (TWLTL)	Painted	6.66	2.49
Divided four lane	Raised	3.86	0.97

Note: TWLTL = two-way left-turn lane.
Source: Oregon State University, 1998.

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Alternative Dowel Bars

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ABSTRACT

Alternative dowel bars for joints have been undergoing investigation at Iowa State University (ISU). The alternatives include size, shape and material parameter changes from the conventional 1.5-in. diameter, 18-in. long steel dowels currently employed in joints of pavements, bridge approaches and other locations where load transfer is needed but longitudinal movement must be accommodated. Alternative materials from the conventional steel dowel have been investigated. Among these alternative materials, fiber reinforced polymer (FRP) has been give considerable attention. FRP is considered a high candidate to replace steel in those areas where corrosion is of concern. The FRP allows for the slip that is needed but provides for a non-corrosive dowel bar. In addition to alternative materials that have been investigated, alternative shapes have been tested at ISU. Elliptical, hollow, and other shapes have been tested in the Structural Engineering Laboratory. The tests include elemental behavioral parameter tests, and full-scale pavement slabs subjected to up to 10 million cycles of load. This paper will focus on the structural behavior of the dowel bars utilized in conventional joints of a small gap.

Key words: dowel bars—FRP—pavement—reinforcement—slabs

Proceedings of the 2003 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2003.

INTRODUCTION

Based upon a research survey conducted at Iowa State University sponsored by the Civil Engineering Research Foundation's (CERF) Highway Innovative Technology Evaluation Center (HITEC), over 18 million dowel bars are sold per year for use in the US highway systems (*1*). If approximately two-thirds of the US has a potential for corrosion via roadway chemicals, sea salts, or chemical concrete mixtures; then, a potential for an alternative corrosion-free or corrosion resistant dowel bar would affect the sales of about 12 million dowel bars per year. One of the likely candidates for an alternative material to stop the corrosion problem is that of a fiber-reinforced polymer, commonly known as FRP or GFRP.

Dowel bars are subject also to large cycles of fatigue loading as the bars transmit loads from one portion to another of a concrete pavement, bridge, or other structural components. As the continued cycling occurs, the bearing of the contact from the bar on the concrete can cause an "oblonging" of the hole surrounding the bar for a typical circular bar. Thus, a need also exists to reduce the bearing stresses between the dowel bar and the concrete.

The combination of the corrosion and bearing fatigue problems for dowel bars leads to the need to consider alternative shapes and materials for dowel bars. Research has been on-going at ISU on both the alternative shapes and the alternative materials for dowel bars. Several types of structural tests and analyses have been conducted at ISU. This short paper will provide a summary, some results and analyses, and some recommendations based upon the ISU structural work.

PARTIAL SUMMARY OF ISU WORK

The structural laboratory work at ISU has focused on many different potential dowel bars of various shapes and materials. The different types of dowel bars investigated include

- 1.5-in. ϕ standard epoxy coated,
- 1.5-in. ϕ stainless steel,
- 1.5-in. ϕ GFRP,
- 1.875-in. ϕ GFRP,
- 1.5-in. ϕ aluminum,
- 1.957-in. ϕ aluminum,
- 1.714-in. ϕ copper,
- 1.5-in. ϕ copper,
- 1.5-in. ϕ stainless steel,
- 1.5-in. ϕ hollow-filled,
- 1.75-in. ϕ GFRP,

- Aged GFRP,
- Special-sized shaved GFRP,
- Hollow-filled,
- 1.5-in. ϕ plain steel, and
- Several sizes of elliptical shaped

The types of structural laboratory tests conducted include

- full-scale pavement sections subjected to fatigue loading,
- Iosipescu elemental shear (static)
- AASHTO T-253 elemental shear (static and fatigue),
- Pull-out,
- Alkalinity aging, and
- Chemical properties

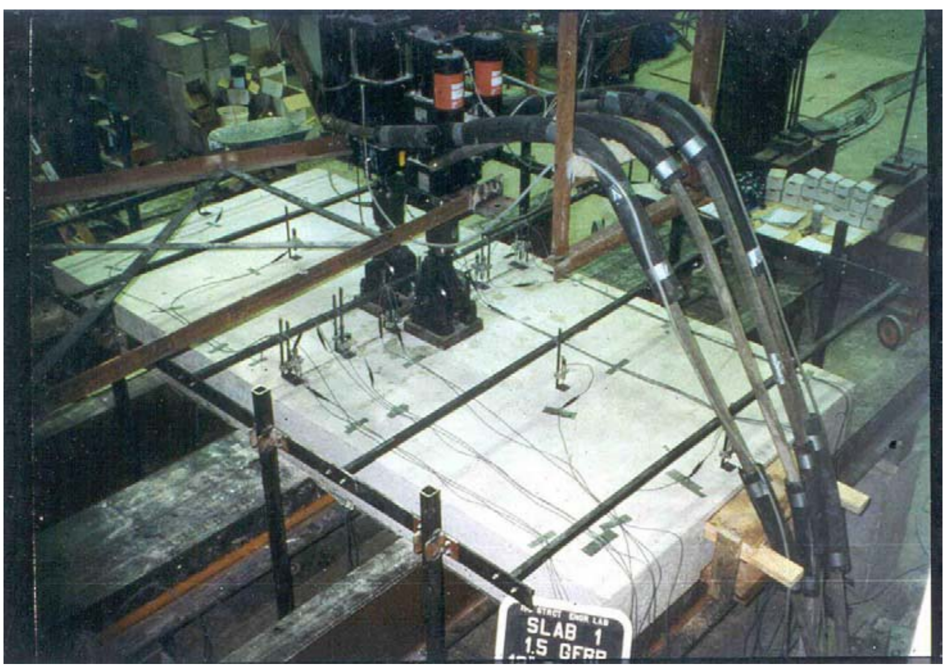

FIGURE 1. Full-Scale Test of Pavement Slab Containing Dowel Bars

All of the above-mentioned tests were conducted in the Structural Engineering Laboratory at ISU. The full-scale test is shown in Figure 1. The AASHTO test is shown in Figure 2. Since the FRP was a special idea of the author, a significant number of tests have focused on this application.

The latest work has focused on elliptically-shaped dowels. The elliptical shape has been used for steel and GFRP dowels for laboratory testing and in recent field applications. The field projects currently underway are directed by Dr. James Cable and the author. This paper, however, is focused only on the structural laboratory applications.

Most of the laboratory tests have been supported by the IDOT, the IHRB, and the various manufacturers of the alternative dowel bars and given in References (*2-6)*. Some of the significant results of the GFRP dowels have been summarized in the Papers (*7-9*) and in a newsletter Article (*10*).

ANALYSIS

The analysis of the alternative dowel bars has included several aspects, most important of which is a straight forward means of determining a mathematical means of strength and deflection of the dowels across a pavement joint. The following equations have been derived to give a relation of the deflection of the pavement joint as shown in Figure 3.

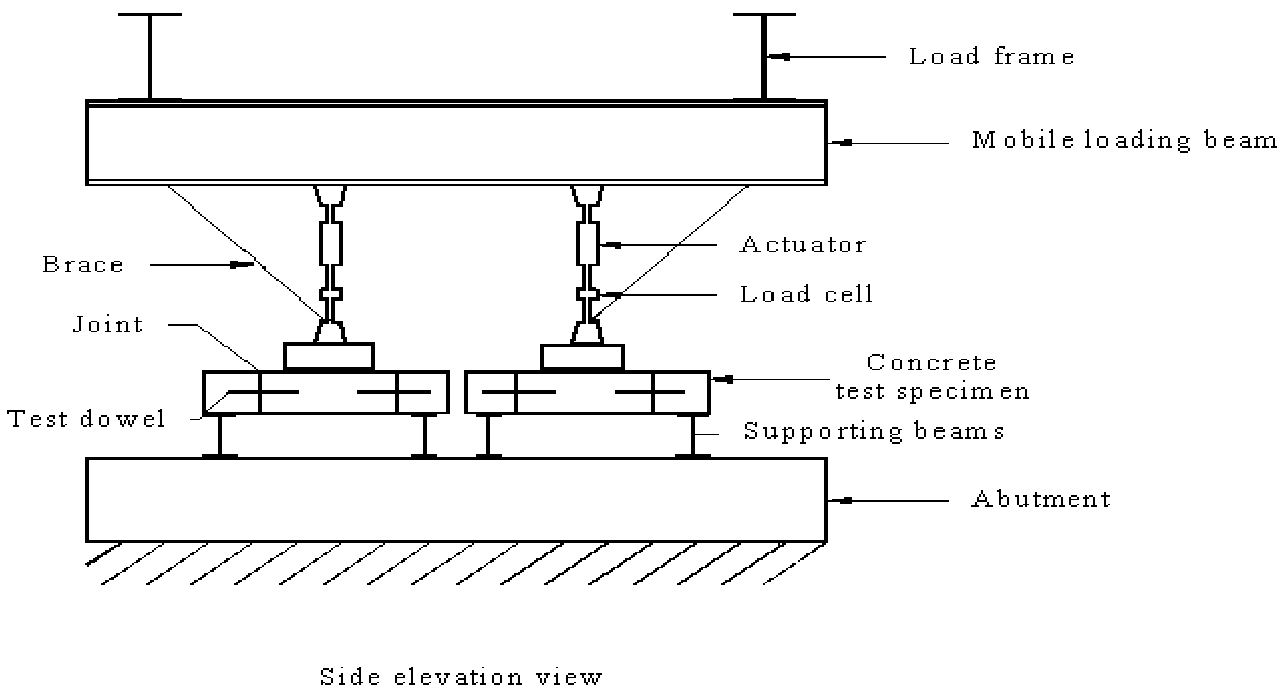

FIGURE 2. AASHTO Elemental Tests

The deflection at the face of the joint, Δ, was determined by Equation 1. The equation for shear deflection, δ, is shown in Equation 2 and was obtained from Young (*11*).

$$\Delta = 2y_o + z\left(\frac{dy_o}{dx}\right) + \delta + \frac{Pz^3}{12EI} \tag{1}$$

where,

$$\delta = \frac{\lambda Pz}{AG} \tag{2}$$

The modulus of dowel support, K_o, along with the bar properties are related as shown in Equations 3.

$$y_o = \frac{P}{4\beta^3 EI}(2 + \beta z) \tag{3}$$

where,

$\beta = \sqrt[4]{\frac{K_o b}{4EI}}$ = relative stiffness of the dowel bar encased in concrete (in^{-1})

By imputing various values of K_o into Equation 3, a K_o versus y_o graph can be created. Since Equation 2.9 is dependant on the bar shape and material properties, a K_o versus y_o graph must be created for each dowel bar of a different shape and/or material. Shown in Figure 4 is a sample K_o versus y_o graph for a 1.5-in. ϕ round epoxy coated steel dowel bar.

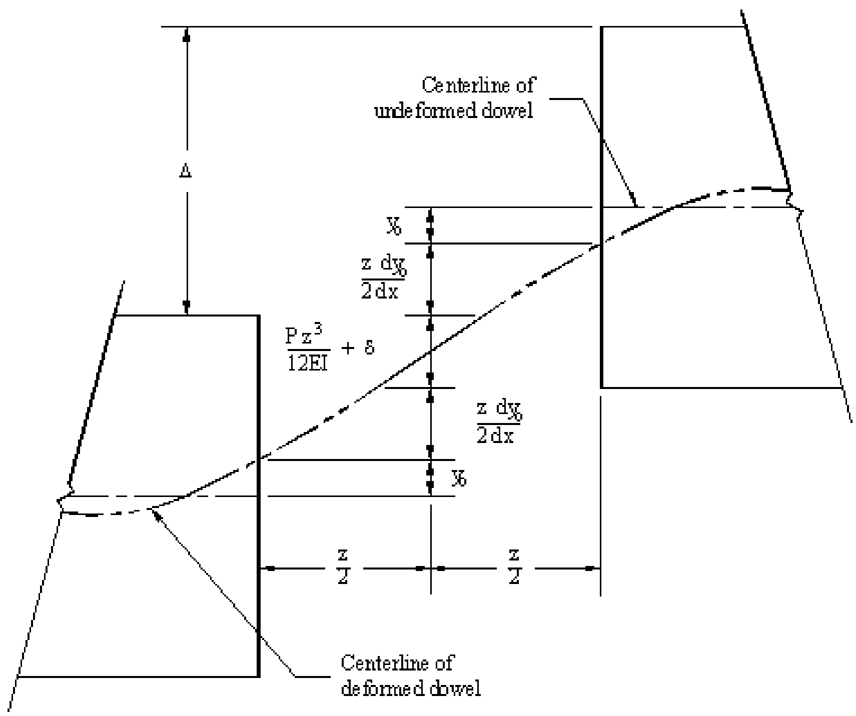

FIGURE 3. Deflection Relationships Across aJoint of Width Z.

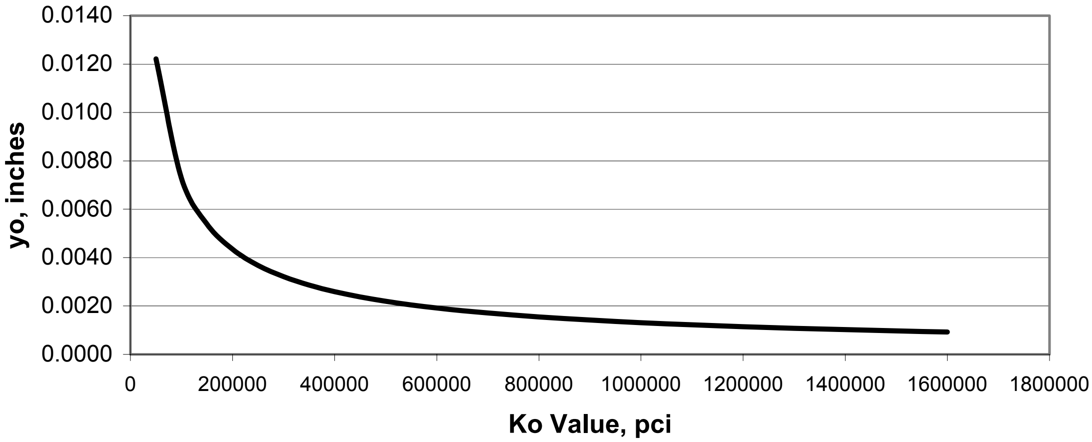

FIGURE 4. Relationship of Modulus of Dowel Support vs. Deflection for a 1.5" Steel Dowel Bar

Using the modulus of dowel support and the deflection at the face of the joint the concrete bearing stress can be calculated, as shown by Equation 4.

$$\sigma_o = K_o y_o = \frac{K_o P}{4\beta^3 EI}(2 + \beta z) \tag{4}$$

where in the above equations
E = modulus of elasticity of the beam (psi)

I = moment of inertia of the beam (in.^{4})

K_o= modulus of dowel support (pci)

B = dowel bar width (in.)
E = modulus of elasticity of the dowel bar (psi)
I = moment of inertia of the dowel bar (in.^{4})
P = load transferred through the dowel (lbs)
z = joint width (in.)
λ = form factor, equal to 10/9 for solid circular section
A = cross-sectional area of the dowel bar (in.^{2})
G = shear modulus (psi)

BRIEF RESULTS AND CONCLUSIONS OF SOME LABORATORY TESTS

The AASHTO tests were conducted after the Iosipescu tests of elemental specimens subjected to direct shear. During the sequence of conducting the AASHTO tests (Figure 2), several items were found to be wrong with that test procedure. Thus, recommendations are being put forth for work to be done to correct the deficiencies, such as possible rotation, uneven bearing, changes in load distribution, and issues of shear and moment across the interface joint. Also, issues w.r.t. the joint width, z, need to be addressed in a revised test procedure. Some of the highlights from the Reports (*2,3,4, and 5)* are as follows:

- The results of this research indicated that the elliptical dowel bars behaved as predicted. When comparing the 1-1/2 in. ϕ round epoxy coated steel dowel bars to the large elliptical steel dowel bars, the large elliptical steel dowel bars produce bearing stresses on the concrete that are greatly reduced while the increase in relative deflection is minimal.

- The large elliptical steel dowel bars have an increase in cross-sectional area of nearly 18% but provide a reduction in bearing stress of over 26%. In contrast, the 1-1/2 in. ϕ round epoxy coated steel dowel bars have a 44% increase in cross-sectional area over the smaller 1-1/4 in. ϕ round epoxy coated steel dowel bars yet only provide a 25% reduction in bearing stress.

- The round dowel bars did retain a slight advantage in the stiffness over elliptical dowel bars of a similar cross-sectional area due to their shape. However, this difference in stiffness is insignificant based on the small variance in the deflection of the slabs. The difference in magnitude of the deflections is so small that the dowel bars could be considered as having roughly the same deflection.

- This research has shown that the 1.5 in. ϕ round epoxy coated steel dowel bars have roughly same bearing stress as the medium elliptical dowel steel bars. This occurrence could be beneficial if the load transfer efficiency was determined.

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- Dowel bar spacing is a method to distribute load to the dowel bars. The smaller the spacing of the dowel bars the smaller the load on the dowel bars. A decrease in pavement thickness will lower the number of bars available for load transfer and a smaller spacing may be required.

- The 1.5-in. diameter GFRP dowels spaced at 12 in. on center were inadequate in transferring load.

- The 1.5-in. diameter GFRP dowels spaced at 6 in. on center were effective in transferring load over the design life of the pavement.

- The current design guideline for steel dowels cannot be applied to GFRP dowels.

- The 1.75-in. FC dowels spaced at 8 in. performed at least as well as 1.5-in. steel dowels at 12 in. for transferring static loads across the joint in the full-scale pavement test specimens. The performance of the 1.75-in. FC dowels spaced at 12 in. was similar to that of the 1.5-in. steel dowels spaced at 12 in. with any difference being attributed to dowel diameter.

- The load transfer efficiency of 1.75-in. FC dowels spaced at 8 in. for a full-scale pavement slab was nearly constant (approximately 44.5% load transfer) through two million applied load cycles with a maximum of 9,000 pounds.

- The load transfer efficiency of 1.5-in. steel dowels spaced at 12 in. for a full-scale pavement slab decreased (approximately from 43.5% to 41.0% load transfer) over the first two million cycles.

- The load transfer efficiency of 1.75-in. FC dowels spaced at 12 in. for a full-scale pavement slab decreased from an initial value of approximately 44% to a final value of approximately 41% after 10 million cycles.

ACKNOWLEDGEMENTS

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Reducing Crashes at Rural Thru-STOP Controlled Intersections

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Key words: crash reduction—intersections—safety

INTRODUCTION/PROBLEM STATEMENT

County engineers around the State of Minnesota identified the frequency and severity of right angle crashes at rural Thru-STOP controlled intersections as being a significant concern. They also suggested that it was commonly believed that the key contributing factor to these crashes were vehicles on the minor road approaches running through the STOP signs.

An informal survey of practice of county highway departments found that the typical approach to designing the traffic control at these rural intersections started with installing a 30 by 30 inch STOP sign. Then, if there were ever an intersection recognition or crash problem the county highway staff would consider a variety of alternatives contained in their traffic safety tool box; including:

- adding rumble strips or overhead flashers
- adding a supplemental plaque to the STOP sign (CROSS TRAFFIC DOES NOT STOP)
- adding either bigger or additional STOP signs

However, a series of research reports has suggested that these strategies have not been consistently effective at reducing crashes. Research conducted by Iowa State University (*1*) analyzed more than one-hundred locations in Iowa and concluded that intersections with rumble strips had a higher crash frequency than comparable intersections without rumble strips. A similar study of twenty-five intersections in Minnesota came to the same conclusion (*2*). A study by the University of Minnesota of red/yellow overhead flashers installed at rural Thru-STOP controlled intersections found that they not only didn't reduce crashes, but in a driving simulator environment they also appeared to confuse drivers into thinking that the intersection was actually controlled by an All-Way STOP (*3*). (Following the release of this study, MnDOT issued a technical memo to their District's requesting the removal of these devices.) The University of Arkansas found the CROSS TRAFFIC DOES NOT STOP plaque to be of limited effectiveness because of inconsistent usage of the plaque and the fact that most drivers do not understand the concept of just exactly what is cross traffic (*4*). And finally, prior to the current study conducted by Harder, Bloomfield and Chihak using the University of Minnesota's driving simulator there was virtually no research documenting the effectiveness of either larger or additional STOP signs. There is one additional key point, the basic approach used by the typical county engineer to address rural intersection safety appears to assume that the primary contributing factor is lack of recognition of the STOP sign by drivers on the minor road approaches (which results in the vehicles on the minor road running through the STOP signs). However, there is nothing in the literature to suggest that this is a valid assumption.

Therefore, the Minnesota Local Road Research Board adopted as an objective - the identification of new mitigation strategies for inclusion in the County Engineer's Safety Tool Box, based on addressing the root causes of rural intersection crashes.

APPROACH

The basic approach to addressing the traffic engineering issues associated with safety at rural intersections consisted of the following four key steps:

1. Describing the basic geometric characteristics of typical rural Thru-STOP intersections and then using MnDOT's crash records data base to identify the actual crash profile (crash frequency, severity, type, etc.) for a set of similar intersections and then identifying high frequency intersections.

2. Reviewing the actual police crash reports of a sample of the total crashes (i.e., all right angle crashes) in order to document the cause of the crash as noted by the investigating officer.

3. Randomly selecting a sample of the high crash frequency intersections, conducting a field review at the selected locations and documenting the basic roadway geometry and intersection area traffic control devices.

4. Identifying potential mitigation strategies that are directly linked to the documented crash causes and contributing factors.

Crash Analysis Results

The crash data analysis began by describing the geometric characteristics of the typical rural intersection in order to screen out those intersections with features that could result in baseline crash patterns that are not representative of the desired conditions. Conversations with a number of county engineers indicated that the typical features of their rural intersections include:

- 2-lane roadways only

- No medians or auxiliary turn lanes

- Four legs of approach

- Thru-STOP intersection control

The search of MnDOT's transportation information database found a total of 7,634 intersections along the State's system of trunk highways, of which 3,920 (51%) were classified as Rural and Through/STOP and 1,604 (21%) were found to meet all of the search criteria (see Figure 1). It should be noted that the study was limited to state highway intersections with county highways, county roads and other local roads (primarily township roads) because of the need to access MnDOT's roadway information data. Similar roadway and traffic control databases do not currently exist for county and local roads.

MnDOT's crash records database was then used to document the crash characteristics for the group of 1,604 similar intersections and for each intersection in the group over an almost three year study period. The key results of this analysis include the following:

- 721 (45%) intersections had no crashes.

- 883 (55%) intersections had at least one crash.

- There were a total of 2,296 crashes at the 1,604 intersections.

- Right angle crashes were the most common type of intersection crash (768 crashes/33%) and the most severe (62% of serious injuries and 71% of fatalities).

- This group of intersections averaged 0.6 crashes per intersection per year and approximately 0.2 right angle crashes per intersection per year.

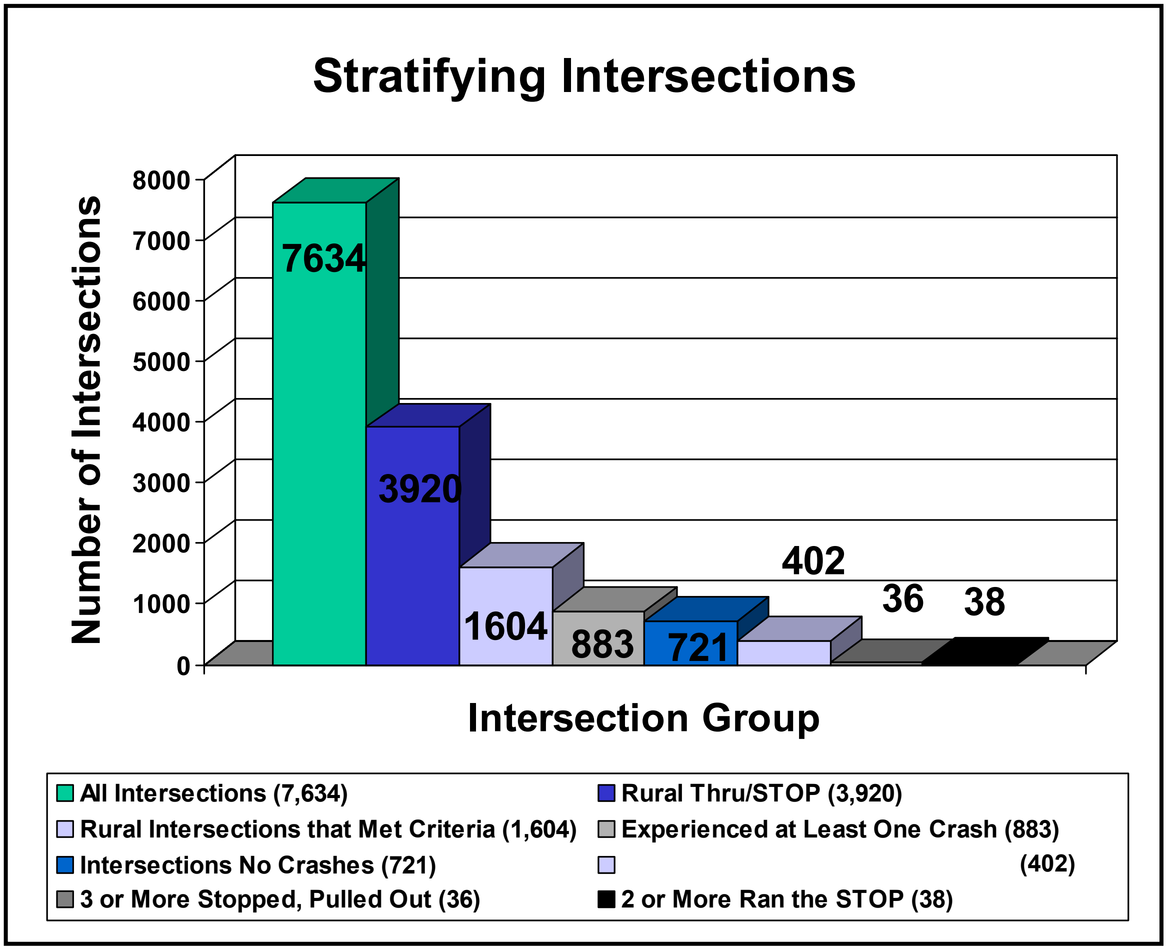

FIGURE 1: Intersection Stratification

The results of the review of MnDOT's crash records database confirms the county engineers concerns about right angle crashes at typical rural, Thru-STOP intersections. Right angle crashes are the most frequent type of crash at these intersections and the most severe. However, the crash records database could not provide detailed information relative to either the causes of the crashes or any background information about the drivers involved in these crashes.

As a result of the limitations of the crash records database, the actual police crash reports for the 768 right angle crashes were reviewed. These reports revealed the following information:

- 435 right angle crashes (57%) involved a vehicle that had stopped at the STOP sign and then Pulled Out (see Figure 2).

- 204 right angle crashes (26%) involved a vehicle that Ran Thru the STOP sign (see Figure 2).

- 129 right angle crashes (17%) could not be identified relative to vehicle actions (either vehicle action prior to the crash wasn't documented or there was conflicting information – Figure 2).

- Right angle crashes caused by vehicles Running Thru the STOP sign are more severe than STOP and Pull Out crashes (and much more severe than the average for all crashes in Minnesota over the same time period – Figure 3).

- A total of 156 intersections (10%) had Ran Thru the STOP crashes, of which 118 (7%) had only one crash and only 38 (2%) had two or more crashes during the study period. Only 1% of these intersections averaged more than one Ran Thru the STOP crash per year.

- A total of 251 intersections (16%) had STOP and Pull Out crashes, of which 162 (10%) had only one crash and only 36 (2%) had three or more STOP and Pull Out crashes during the study period. Only 10% of these intersections averaged more than one STOP and Pull Out crash per year.

- Most of the right angle crashes occurred during daylight conditions. However, the percentage of Ran Thru the STOP crashes that occurred after dark (18%) is approximately twice the rate for both STOP and Pull Out (8%) crashes and the rate for all crashes in Minnesota (10%).

- The age of drivers involved in both Ran Thru the STOP and Stop and Pull Out crashes was compared to the expected distribution based on Statewide crash totals (see Figure 4). The data indicates that drivers between the ages of 25 and 40 are significantly over represented in Ran Thru the STOP crashes and drivers less than 19 and over 85 are over represented in STOP and Pull Out crashes.

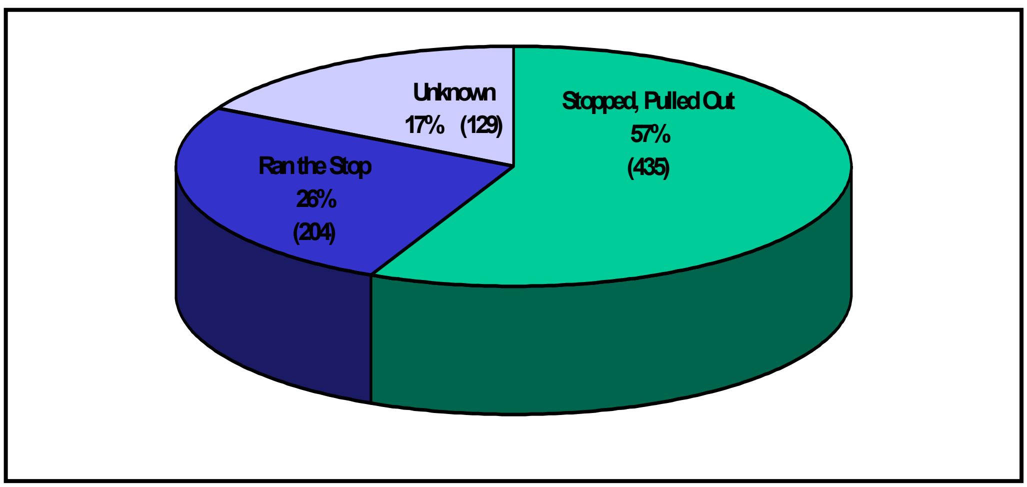

FIGURE 2: Distribution of Right Angle Crash Type at Study Intersections

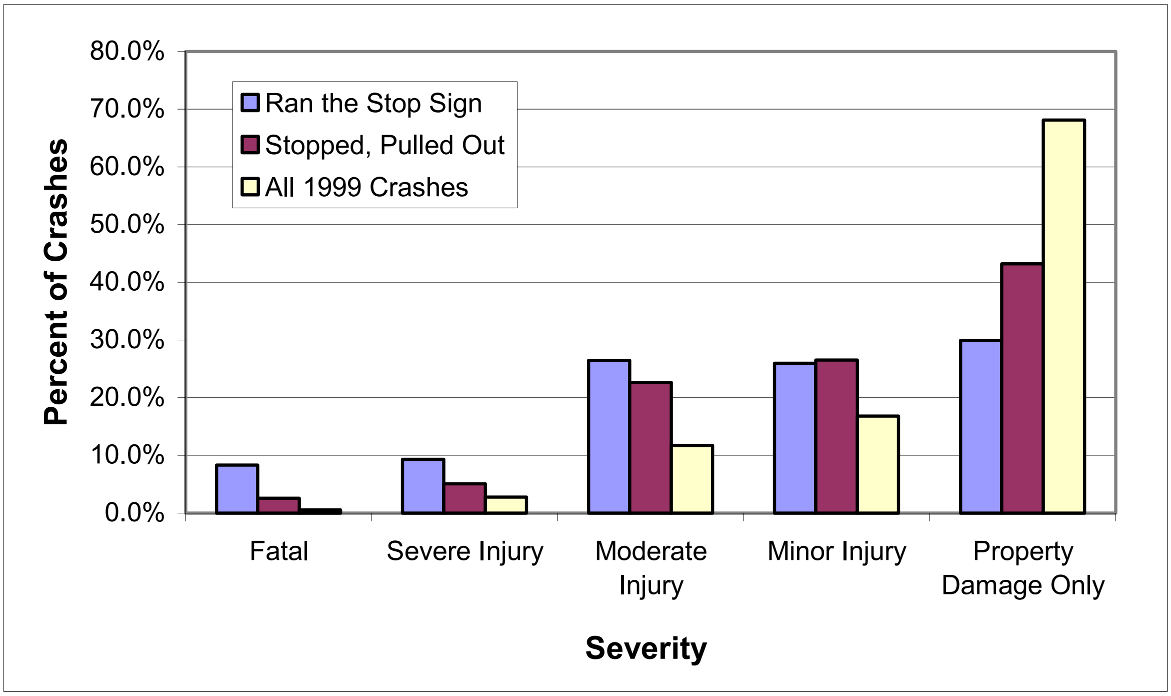

FIGURE 3: Right Angle Crash Severity

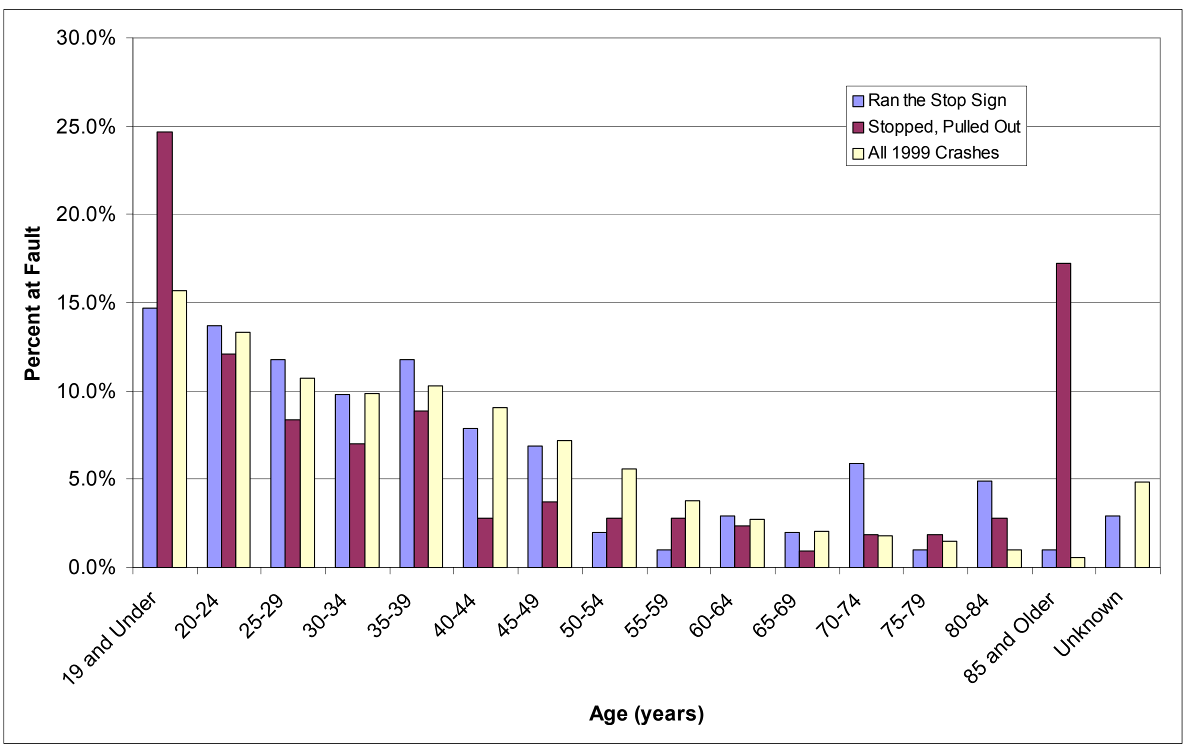

FIGURE 4: Age Distribution

Field Review of Selected Intersections

The final step in the analytical process involved randomly selecting a sample of ten intersections from each of three different categories for field review. The three categories of intersections identified for review include; intersections with multiple Ran Thru the STOP crashes, intersections with multiple Stop and Pull Out crashes and as a comparison intersections with no crashes during the study period. The objective of this effort was to determine if the basic design and/or traffic control features could be contributing to the higher than expected frequency of right angle crashes. The focus on intersections with multiple crashes was based on the expectation that intersections with only one crash during the study period were most likely examples of the random nature of crashes while intersections with multiple crashes would have a far higher probability of somehow being different than the other intersections in the group and that the difference was contributing to the higher than expected frequency of crashes.

The thirty intersections (10 per category) selected for the field review are identified in Table 1, along with the basic crash characteristics and the critical crash rate. The critical crash rate is a statistical quality control technique. Intersection crash rates higher than the critical rate are an indication that the crash frequency is statistically significantly higher than expected and that the difference is most likely due to conditions at the intersection (as opposed to the random nature of crashes). It should be noted that 19 of the 20 intersections selected for review have crash rates equal to or greater than the critical rate. This suggests that the use of multiple crashes as a selection criteria was a valid approach to identifying intersections where something was different.

The field review documented the following features:

- Signs – number, size, placement and condition of STOP and STOP AHEAD signs
- Intersection Sight Distance (from the minor road approaches)
- Sight Obstructions to Signs (along the minor road approaches)
- Presence of Other Devices (street lights, pavement markings, rumble strips)
- Proximity to Other Controlled Intersections (along the minor roads)
- Daily Traffic Volumes

The key observations derived from the field review follow. It should be noted that these observations are merely that - observations, and because of the limited sample size are not statistically significant.

Signs

- At intersections with crashes, the use of more and larger STOP signs appears to reduce the number of Ran the STOP crashes.

TABLE 1: Field Review Intersections

	Major Street	Minor Street	County	Number of "Stopped, Pulled Out" Crashes	Approach Volume	Total Crashes	Crash Rate	Critical Crash Rate	Severity Rate
Stopped, Pulled Out	USTH 12	CSAH 92	Hennepin	3	12,860	6	0.5	0.7	1
	MNTH 3	160th Street CSAH 46	Dakota	5	8,350	12	1.6	0.7	3.5
	MNTH 3	Rich Valley Blvd	Dakota	4	8,870	8	1.0	0.7	2.5
	MNTH 25	CSAH 37A	Wright	4	13,340	13	1.1	0.7	2.3
	MNTH 23	CSAH 18	Yellow Medicine	3	3,850	5	1.4	0.8	4
	MNTH 25	CR 106 & T225	Wright	3	13,080	9	0.8	0.7	2
	USTH 61	170th Street CSAH 4	Washington	4	10,250	10	1.1	0.7	2.7
	MNTH 7	CSAH 10	Carver	10	9,910	15	1.7	0.7	4.6
	MNTH 60	TH 57	Goodhue	3	4,070	7	1.9	0.8	5.1
	MNTH 23	CSAH 12	Kanabec	3	4,810	8	1.8	0.8	4.6
				Number of "Ran the Stop" Crashes					
Ran the Stop	MNTH 246	CSAH 1 and CR 81	Rice	2	3,920	8	2.2	0.8	6.4
	MNTH 57	CSAH 22	Dodge	2	1,540	2	1.4	0.9	3.6
	MNTH 95	CSAH 7 and CR 57	Mille Lacs	2	4,420	6	1.5	0.8	4.5
	MNTH 95	CSAH 7 and CR 57	Isanti	2	5,370	6	1.2	0.8	2.4
	MNTH 200	CSAH 4	Mahnomen	2	1,400	3	2.4	0.9	7.1
	USTH 71	CSAH 4	Renville	2	3,670	3	0.9	0.8	3.6
	USTH 61	190th Street	Washington	2	6,430	7	1.2	0.7	2.7
	USTH 212	CSAH 43	Carver	3	9,790	6	0.7	0.7	2
	MNTH 4	CR 186B	Stearns	2	1,750	6	3.8	0.9	11.3
	MNTH 23	CSAH 12	Kanabec	2	4,810	8	1.8	0.8	4.6
No Crashes	MNTH 104	CSAH 13 LT	Kandiyohi		730	0	0.0	0.9	0
	USTH 8	HAMLET AVE	Chisago		18,200	0	0.0	0.6	0
	MNTH 96	OAK GLEN TR	Washington		5,070	0	0.0	0.8	0
	MNTH 5	CSAH 50	Carver		4,240	0	0.0	0.8	0
	MNTH 50	JOAN AVE	Dakota		4,410	0	0.0	0.8	0
	MNTH 21	230TH ST C	Scott		4,210	0	0.0	0.8	0
	MNTH 210	CSAH 65	Otter Tail		1,340	0	0.0	0.9	0
	MNTH 25	CSAH 47	Morrison		1,740	0	0.0	0.9	0
	MNTH 4	CR 179	Stearns		1,350	0	0.0	0.9	0
	MNTH 30	CSAH 8	Olmsted		3,060	0	0.0	0.8	0

NOTE: Shaded rows indicate intersections with crash rates over the critical rate.

- The placement of STOP signs does not appear to be an issue. There was very little variation in the placement of the signs from intersection to intersection and the placements were all in substantial compliance with the guidelines in the MNMUTCD.

- The use of brighter retroreflective sheeting material appears to reduce the frequency of both total crashes and right angle crashes. The highest usage of diamond grade sheeting was at intersections with no crashes and the lowest usage was at intersections with multiple Ran the STOP crashes.

- STOP AHEAD signs were in place at all but one intersection. At intersections with crashes, it appears that the use of larger, brighter advance warning signs reduces the frequency of Ran Thru the STOP crashes.

Intersection Sight Distance

- Intersection sight distance does not appear to be related to the frequency of STOP and Pull Out crashes.

- Each category had about the same number of intersections (between 2 and 4) with less than adequate intersection sight distance (Assumed to be 10 seconds, consistent with the basic guidance in the AASHTO Green Book – 5).

- The category with multiple STOP and Pull Out crashes had the fewest number of intersections with less than adequate sight distance.

Sight Obstructions to STOP Signs

- Sight obstructions to STOP signs does not appear to be related to the frequency of Ran the STOP crashes.

- Intersections with no crashes had the lowest frequency of obscured signing (*1*). However, the intersections with multiple Ran the STOP crashes only had two instances of obscured signing.

Presence of Other Devices

- Intersections with crashes and street lights had a much lower frequency of both night time crashes and Ran the STOP crashes.

- Intersections with STOP AHEAD pavement markings had a lower frequency of Ran the STOP crashes.

- Intersections with rumble strips on the minor road approach had the same frequency of Ran the STOP crashes as intersections without rumble strips.

Proximity to Other Controlled Intersections

- Proximity to other controlled intersections (along the minor road) may be related to total crash frequency. All of the intersections with both multiple STOP and Pull Out and Ran Thru the STOP crashes were more than a mile away form the nearest controlled intersection, while only one-half of the intersections with no crashes were more than a mile from another controlled intersection.

Daily Traffic Volumes

- The intersections with multiple STOP and Pull Out crashes had the highest traffic volumes, with the average daily approach volume in the range of 9,000 vehicles per day. The other two intersection categories each had average approach volumes in the range of 4,500 vehicles per day.

During a review of this material with the advisory panel of county engineers, it was requested that the crash data for the sample of intersections with multiple crashes be analyzed to determine if unfamiliar motorists were contributing to the frequency of crashes. The analysis consisted of identifying the location of each crash and the home city of each of the "at fault" drivers and then documenting the distance between the two. The results of this analysis indicate that the average distance between home and crash site is less than 10 miles and 80% of the distances are less than 30 miles. This would suggest that unfamiliar drivers do not appear to be over represented in either STOP and Pull Out or Ran Thru the STOP crashes. This might also suggest that, in rural areas with relatively low traffic volumes, drivers who are in fact familiar with the road and know that traffic is fairly infrequent might not be paying sufficient attention.

POTENTIAL MITIGATION STRATIGIES

The results of the analysis of the crash data and the field review of the selected intersections suggest the need to develop two basic approaches to addressing the issue of right angle crashes at rural through/STOP intersections. The first would have to address the most prevalent type of crash, STOP and Pulled Out, where the primary contributing factor is gap selection. The second would then address the Ran the STOP types of crashes, where the primary contributing factor is intersection recognition. The results of the crash analysis also suggests that due to the very low frequency of occurrence of either type of crash (an average of less than .1 crashes per intersection per year), the most effective implementation would most likely involve a systematic approach instead of an approach focused on the very small number of locations with multiple crashes.

It appears that there are two types of strategies for addressing the issue of gap selection, static and dynamic. The Pennsylvania Department of Transportation has experimented with the use of pavement markings to identify unsafe gaps. Other possibilities include the use of sign posts or street light poles to identify the limits of unsafe gaps on the major street approaches to intersections. An example of how these devices could be deployed would consist of placing a series of street lights on the major street approaches, with the last street light at the limits of the unsafe gap - current research (*5*) suggests that a safe gap at a rural intersection would be in the range of 650 to 800 feet (approximately 8 to 10 seconds at a posted 55 mile per hour speed limit). A dynamic system could use either a optical detectors or radar to sense the location of vehicles approaching the intersection on the major street and then using some type of changeable message sign to inform the driver waiting at the STOP sign on the minor street. The University of Minnesota has just begun a research project that will look at developing and evaluating alternative dynamic technologies to assist with the gap selection process.

It appears that there are also static and dynamic strategies for addressing the issue of intersection recognition. The static strategies would increase the conspicuity of traffic control devices by making them larger, brighter or using additional signs and markings. The dynamic approach could use optical detectors or radar to sense the speed of vehicles on the minor street approaches and then use some type of changeable message sign to warn drivers on the major street to take action to avoid vehicles that appear to be going too fast to stop prior to entering the intersection. Virginia Tech has just begun a research project that will look at developing and evaluating alternative dynamic technologies to warn of STOP sign and traffic signal violators.

In either case, a complementary component to developing and deploying new or additional devices would include an educational program for drivers. Drivers would have to be taught what these devices mean and what they would need to do to effectively use the information in order to prevent crashes.

CONCLUSIONS

The review of MnDOT's crash records data base and police crash reports indicates the following:

- Almost one-half of the 1,604 rural Through/STOP intersections in the data base experienced no crashes during the 2+ year study period.

- This group of intersections averaged 0.6 crashes per intersection per year and approximately 0.2 right angle crashes.

- Right angle crashes account for more crashes than any other type.

- Right angle crashes are the most severe type of crash, accounting for nearly 71% of the fatal crashes.

- The most common type of right angle crash (approximately 60%) involves a vehicle Stopping and then Pulling Out into an unsafe gap. Approximately 25% of the right angle crashes involved a minor street vehicle Running through the STOP sign. (The remaining right angle crashes could not be classified.)

- The occurrence of STOP and Pull Out crashes is not wide spread. 251 intersections experienced one of these crashes (out of a total of 1,604 intersections over a 2+ year study period). Only 89 intersections experienced two or more STOP and Pull Out crashes.

- The occurrence of Ran the STOP crashes is not wide spread. 118 intersections experienced one of these crashes and only 38 intersections experienced two or more Ran the STOP crashes.

- Light condition at the intersection appears to be a contributing factor. Vehicles are running the STOP signs at intersections without street lights at twice the statewide average for all crashes.

- The age of the driver appears to be a contributing factor. Drivers under 20 and over 85 are over represented in STOP and Pull Out crashes and drivers between the ages of 25 and 40 are over represented in Ran the STOP crashes.

- Increasing the conspicuity of traffic control devices by using bigger, brighter or additional signs and markings appears to lower the frequency of Ran the STOP crashes.

- Rumble strips do not appear to be effective at reducing the frequency of Ran the STOP crashes (intersections with and without rumble strips had the same frequency of crashes).

- Intersection sight distance does not appear to be related to the frequency of gap selection related crashes.

- Proximity to other controlled intersections may be related to crash frequency.

These conclusions suggest that some of the old strategies in the Intersection Safety Tool Box should be discarded because they have no history of being consistently effective (rumble strips on the approach to intersections, red/yellow overhead flashers and the CROSS TRAFFIC DOES NOT STOP sign). In addition, these conclusions appear to support the development of two new types of mitigation strategies, the first focused on improving the ability of drivers on the minor street to identify safe gaps in traffic approaching on the major road and the second focused on addressing the issue of intersection recognition. These strategies would then be candidates for the Intersection Safety Tool box if and when they are able to demonstrate their effectiveness.

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Turning Students on to Transportation: A Pilot Program for Recruiting High School Students into Transportation Careers and Programs of Study

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ABSTRACT

Iowa is facing a looming crisis with its transportation workforce. As experienced, older workers retire, young people are not replacing them. To help address this problem, the Center for Transportation Research and Education (CTRE) at Iowa State University developed a pilot retention and recruitment program aimed at high school students. CTRE formed an advisory committee composed of people in education and transportation. With the guidance of this committee, CTRE carried out several recruitment activities during the 2002–2003 school year. Strategies included an initial interest survey, a "show and tell" session with a high tech snowplow, a booth at two high schools' career days, and a day-long field trip to a road construction site and college campus.

Key words: recruitment—students—transportation careers—workforce development

INTRODUCTION

Since 1980, the total number of vehicle miles traveled in Iowa has increased by 62 percent. Increased traffic means increased wear on roads, and two-thirds of the pavements on Iowa's primary road system are more than 30 years old. There is plenty of work to be done. But like the rest of the United States, Iowa is beginning to experience a shortage of individuals working in transportation-related careers. Recruiting young people to the industry is crucial.

Attracting young people to the field of transportation has become critical for maintaining and developing Iowa's transportation infrastructure. According to the National LTAP (Local Technical Assistance Program) Association, nearly half of the current transportation workforce may retire by 2010. Losing that expertise could be devastating.

Part of the mission of the Center for Transportation Research and Education (CTRE) is education and outreach, which includes research opportunities for ISU students and continuing education opportunities for professionals and technicians in the field through the Iowa Local Technical Assistance Program. Developing a recruitment and retention program for high school students to address the developing need for transportation workers seems like a natural fit.

A one-year pilot program, described below, is the first phase of a multi-phase, statewide recruitment and retention program. The overall goal is to increase enrollment in transportation-related programs at Iowa community colleges and universities by 10 percent in five years.

The goals of this pilot program, which was conducted with North High School in Des Moines, Iowa, and Ames High School in Ames, Iowa, during the 2002–2003 academic year, were to

- recruit Iowa high school students to participate in the pilot program
- increase awareness of professional and technical opportunities in transportation among high school students, their parents, and guidance counselors
- enroll, in a transportation-related program at Iowa State University (ISU) and Des Moines Area Community College (DMACC) in 2003, 1–2 Iowa high school students who likely would not have considered this course of study if not for the pilot program
- develop a promotion/recruitment tool that is easily replicated at other high schools and that will form the foundation for a future multi-phase, statewide recruitment and retention program

FORMATION OF ADVISORY COMMITTEE

The first step was the formation of an advisory committee. The primary role of the committee was to advise CTRE about the program's content and format. Committee members included the following:

- City public works staff: Jeff May, Knoxville, Iowa; Jon Hanson and Al Olson, Ankeny, Iowa
- High school guidance counselors: Liegh Lussie and Roxanne Kucharski, North High School, Des Moines, Iowa; Mike Wittmer, Ames High School, Ames, Iowa
- Iowa Department of Transportation (Iowa DOT) staff: Gerry Ambroson, Recruiting/Co-op Coordinator; Sandra Larson, Research and Technical Bureau Director
- Des Moines Area Community College (Boone campus) staff: Shelby Hildreth, Education Advisor; Renee White, Group Leader for Civil Engineering Technician and Land Surveying

- CTRE staff: Deborah Witt, Student Coordinator; Marcia Brink, Communications Manager; Michele Regenold, Program Supervisor; David Plazak, Associate Director for Policy; Duane Smith, Associate Director for Outreach

DESIGN OF PROGRAM

The advisory committee brainstormed possible activities to build Iowa high school students' interest in careers in transportation. The committee agreed to conduct the following activities:

- Distribute an interest survey and video about careers in transportation
- Provide show and tell sessions with the Iowa DOT's high-tech snowplow
- Operate a booth during career day
- Facilitate a day-long field trip to a construction site and college campus (ISU or DMACC)

Survey and Video

In November, the coordinator wrote and distributed a survey about careers in transportation to approximately 640 ninth through twelfth grade students in North High School's math and science classes. The purpose of the survey was to gauge students' potential interest in the following careers:

- transportation planning
- civil engineering
- civil engineering technician
- land surveying
- construction engineering
- general (non-college level careers)

Roughly 50–60 percent of the students surveyed expressed interest in these career fields.

After taking the survey, students viewed a brief video, *Careers in the Infrastructure*, provided by DMACC. The purpose of the video was to introduce civil engineering, civil engineering technology, construction engineering, construction technology, construction management and land surveying to high school students/teachers. The advisory committee felt that the video reinforced that civil engineers, land surveyors and technicians do many jobs, which are "invisible" to the public (i.e. water supply, sewage treatment, environmental protection, roads, buildings, etc.). The video also reiterates that there will be plenty of jobs in the foreseeable future.

Show and Tell with High-Tech Snowplow

In December, before the snow came, North High science and math students rotated outside for a brief presentation about the Iowa DOT's high-tech snowplow. Dennis Kroeger, a transportation research specialist at CTRE who's conducted research on the snowplow, gave the presentations. He and the snowplow driver answered students' many questions about winter maintenance, costs, and the jobs that went with operating and constructing the snowplow.

Career Day

In February, the booth at the North High and Ames High's career days was an opportunity to interact with students and talk with them about transportation careers. To entice students to stop, we had a big red helikite, which is part kite and part helium balloon, inflated and tied up to the basketball rim behind the booth. Survey equipment was set up for students to experiment with, and two laptops had presentations running, one about transportation careers and one about 3-D community planning software. We distributed flyers that briefly discussed half a dozen careers, salaries, and education requirements. Candy was also an enticement. Besides educating students about transportation-related careers, we wanted to persuade students to sign up for a day-long field trip. Sixteen students signed up for the ISU field trip, and eight signed up for the DMACC field trip.

FIGURE 1. Career Day at North High, Des Moines, Iowa (Student Peers through Land Surveying Equipment while DMACC's Shelby Hildreth Explains What It Does)

Field Trips

The field trips were the culminating events in our pilot recruitment program. The purpose of the field trips, which occurred in April, was to give students a taste of transportation careers by visiting a construction site and a taste of college life by visiting a campus.

FIGURE 2. Students Visited a Construction Site on the Interstate 235 Project in Des Moines, Iowa

Iowa State University Field Trip

Although 16 students originally signed up for this field trip, two students actually attended, one a senior and one a sophomore (both boys). CTRE staff escorted the students and began with a visit to a construction site on the Interstate 235 project in Des Moines. Wes Musgrove, assistant construction engineer for the Iowa DOT, gave the students a tour of a site that included bridge and embankment construction. Next, the students went to CTRE for a short lesson in global positioning systems (GPS) followed by a scavenger hunt for candy using handheld GPS equipment. The GPS lesson and scavenger hunt were presented by Jay Sraker and Steve Truby with ISU's Science, Engineering and Technology Extension office. The students also got a chance to talk to several ISU students working at CTRE. Lunch at ISU's Memorial Union was next. The Union is a busy place and gave the boys a nice feel for college life. After lunch, the boys were given a tour of central campus. The guide pointed out the buildings students most often use and gave them a bit of history about the university. The weather was perfect and showed off the campus well. Both boys seemed impressed with the whole experience.

FIGURE 3. Students Participated in a Scavenger Hunt for Candy Using Handheld GPS Devices

Des Moines Area Community College Field Trip

DMACC experienced much better turn out for its field trip. While eight students originally signed up during career day, 13 students went on this field trip. DMACC's field trip occurred two weeks after ISU's, so word of mouth may have contributed to the increased participation. DMACC staff escorted the North High students and took them for a similar Interstate 235 construction tour. Once students reached the DMACC campus in Boone, they were given a campus tour and then heard presentations about the two transportation-related programs, which are only available on the Boone campus, civil engineering technology and land surveying. Students visited DMACC's materials lab and participated in a couple of hands-on activities with materials. They had lunch brought in to the lab. They concluded their visit with a trip to the admissions office.

FIGURE 4. Students Mixed Materials During the DMACC Field Trip to Boone, Iowa

SUCCESSES AND CHALLENGES

Ultimately, this pilot program will have been successful if one or two North High School students attend either ISU or DMACC in fall 2003 with the intention of studying for a transportation-related career. Since only one of the two students who participated in the ISU field trip was a senior (the other was a sophomore), it's unlikely our recruiting efforts will impact ISU this year. We won't know until classes start at DMACC in August whether the program helped DMACC's recruiting efforts. Students who decide to attend DMACC often do so at the last minute, right before the start of classes.

One of the biggest challenges, we realized too late, was in keeping students' interest level up about participating in the ISU field trip. Six weeks elapsed between the time students signed up for the field trip during the career day and when they actually took the field trip. The North High guidance counselors tracked down the students who had signed up to give them permission slips for their parents to sign, but that wasn't enough to generate interest in more than three students (one was sick the day of the field trip) among the 16 who had originally signed up.

DMACC's field trip, on the other hand, had much better participation. It was scheduled two weeks after ISU's. While DMACC originally had eight students signed up, 13 went on the field trip. We believe this is due, at least in part, to the ISU field trip participants' "spreading the word" about their enjoyable experience.

PROGRAM'S FUTURE

We began this pilot program with the idea that it was the first phase of a multi-phase project that would involve more and more high schools. Now that we've completed the program, we realize how time and resource intensive this approach can be. Consequently, we've come up with a new plan for the next school year (2003–2004).

We'll be taking a two-pronged approach:

1. One part of the program will be to plan and host a day-long transportation career fair at Iowa State University during the late winter or spring and invite students from central Iowa schools and possibly statewide. Our early planning will consist of gauging interest among schools and, assuming sufficient interest, recruiting teachers, guidance counselors, and transportation industry representatives to form an event planning committee.
2. The other part of the program will be the development and distribution of a standardized presentation about careers in transportation to people working in the industry such as city public works directors, consulting engineers, snowplow drivers, motor grader operators, and city planners. These professionals could give the presentations to children during their local school's career day or during extra-curricular events. They can help generate enthusiasm about careers in transportation (and about the career fair) where they live and work. We have a good start on this because of the materials already created for the pilot program.

Potential Program/Activity Models

Many exciting programs exist to interest young people from grade school through college in careers in highway construction and transportation. Following are just a few to give readers an idea of the diversity of programs:

- The U.S. Department of Transportation's Garrett A. Morgan Technology and Transportation Futures Program sponsors a K–12 math, science, and technology literacy challenge to help teachers integrate transportation components into the curriculum.
- The AASHTO/Transportation and Civil Engineering (TRAC) Program includes a hands-on math and science education program, an essay contest about the role of transportation, and a MagLev design/build contest for junior and senior high students.
- The National Association of Women in Construction (NAWIC) provides a video about construction, a building design program for middle/junior high students, and a CAD design/drafting program for high school students.

Two models that we're particularly interested in relate to our proposed career fair:

1. One program was originally developed with the assistance of the Federal Highway Administration (FHWA) and is a good model for the overall structure of our career fair. The FHWA has sponsored construction career days in several states, beginning in Texas in 1999. State departments of transportation have helped fund and/or organize the events. Organizations such as Associated General Contractors have also contributed money, equipment, and volunteers to plan and staff the event.
2. Two other programs, developed by the American Society of Civil Engineers (ASCE) and the National Engineers Week (E-week), are useful models for a major event during the career fair. Both organizations conduct contests on building truss bridges. The purpose of the contest is to provide high school students with a realistic, engaging introduction to engineering. The contest gives students the hands-on opportunity to build a bridge spanning 12 inches that will hold 50 pounds using only Popsicle sticks and glue.

CONCLUSION

Even if the pilot program is unsuccessful in recruiting students to study transportation in fall 2003, we still regard it as a beneficial experience. The advisory committee came away with renewed enthusiasm for recruiting students into transportation-related careers, and the future of the program looks bright. Ultimately we hope the program will generate as much enthusiasm among Iowa students, parents, educators, and employers.

Exclusive Facilities for Trucks in Florida: An Investigation of the Potential for Reserved Truck Lanes and Truckways on the State Highway System

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ABSTRACT

The movement of freight by truck has grown tremendously in the United States. Exclusive highway facilities for trucks (EFTs) are often identified as a countermeasure to reduce congestion, enhance safety, and improve the flow of freight. The Florida Department of Transportation expressed interest in further investigation of the potential for EFTs in Florida and contracted with the Center for Urban Transportation Research (CUTR) to lead the research effort. This project focused on the feasibility of separating large trucks out of the traffic mix. Target areas of concern were instances where trucks had a significantly negative impact on safety and congestion. After conducting a thorough literature review, researchers identified national case studies and visited sites where special treatments had been implemented. Researchers obtained input from state and local transportation officials, highway safety professionals, planners, enforcement agencies, and truck driver public interest groups. CUTR developed a methodology to select sites in Florida that warranted further consideration for EFTs. Specifically, researchers constructed several GIS models to identify “hot spots” based on truck crashes, truck volume and percent, and level of service. Rural and urban locations were both considered, as each scenario presented a different set of challenges. CUTR visited sites in Florida and worked with local officials to document additional details about local streets and interstate highways. Lastly, CUTR assessed the feasibility of countermeasures for each site. Researchers determined that most of Florida’s Interstate system was suitable for EFT consideration; with the most appropriate areas having sufficient available right of way.

Key words: Florida highways—geographic information system traffic models—trucks—truck lanes—truck traffic

INTRODUCTION

Exclusive highway facilities for trucks (EFTs) are often identified as a countermeasure to improve the flow of freight, reduce congestion, and enhance safety. Preliminary observations suggested that there might be both urban and rural locations in Florida where reserved lanes for trucks or separate "truckways" should be considered. The Florida Department of Transportation (FDOT) contracted with the Center for Urban Transportation Research (CUTR) to lead the research effort.

The purpose of this research was to evaluate the potential for reserved truck lanes and truckways in Florida. CUTR examined previously completed scholarly research, as well as prior and ongoing applied projects, and instances where EFTs were considered but not implemented. CUTR also sought to find examples of EFTs, if they did indeed exist. The project specifically examined the current and future potential for reserved truck lanes and truckways on the State Highway System (SHS) and presents a methodology to allow others to evaluate this potential solution. The research examines conditions favorable to reserved truck lanes or truckways and evaluates specific potential applications on the SHS. Operational considerations and practices necessary to feasibly implement this potential solution are also presented.

LITERATURE REVIEW

A considerable body of prior research has discussed the use of exclusive highway facilities for trucks. For this project, information was culled from many diverse sources, including scholarly research projects, private proposals, policy papers, and applied projects. Some projects briefly mentioned the concept as a countermeasure to increase safety on congested highways, while others analyzed several configuration options to improve freight movement or extend roadway life. National, state, and local governments have investigated the potential for implementing special lanes for trucks. European nations have also considered the idea. Applications of truckways have been considered to improve conditions for short- and medium-range facilities such as bridges and port area roadways, as well as along entire corridors such as the proposed NAFTA Superhighway. In some cases, the option has been studied and rejected, while others have yet to reach a firm decision on a course of action.

The literature review revealed several different configuration options for exclusive truck facilities. They range from adding lanes in the median space of an existing highway to the construction of a separate, parallel roadway (*1, 2*). Some studies suggested acquiring additional right of way to add lanes (*2*), while others opt for an elevated structure built in the median (*3*). Truck lanes may be placed on the inside or outside of the roadway, and they may or may not involve a barrier to separate them. Some interior-lane options have been designed as three-lane, variable passing lane facilities (*2*). A few areas, Seattle, Washington for example, have even discussed allowing trucks to share the HOV and/or bus priority lanes (*4*). In any event, tolls may or may not be part of the plan.

Some considerations were found to be common among many projects. In most cases, three factors were measured to determine the feasibility of exclusive truck facilities: safety, operations, and roadway geometrics. Average annual daily traffic (AADT), percent of trucks in the traffic mix, level of service, and lane and shoulder width were usually among the more important items. Others involved available median width, vehicle characteristics, and roadway and vehicle design. Scholarly approaches often employed a reconfigured traffic model to project future volumes and economic feasibility (*5*).

No true exclusive highway facilities for trucks were found to exist. Several factors have steered local and state agencies away from implementing exclusive truck facilities, however the most common issue was the high construction costs. Cost estimates ranged from $4 to $8 million per mile (*2*), and high costs were attributed to right of way acquisition, required heavy-duty construction, and design type (with elevated structures costing the most). In addition, public acceptance of truck-related countermeasures has been mixed. Although public interest groups are generally in favor of making highways safer by removing trucks, they are usually reluctant to fund such projects with tax dollars. The trucking industry also has been skeptical of the benefits of reserved truck lanes, often pointing to a reluctance to pay tolls and the potential for low public opinion. Most agree that it is difficult to estimate the trucking industry's level of compliance if a special facility was in place (*2*).

It is important to note that a number of studies have evaluated restricting trucks from travel in certain lanes of the highway, and over half of the states impose some form of highway lane restriction on trucks. While these studies are significant, the subject area was considered beyond the scope of the project.

National Case Studies

Few truly exclusive facilities for trucks and/or heavy vehicles actually exist. Although the literature review revealed no long-range, truck-only highways, a few short-range, special-use facilities were found. The roadways are site-specific and usually serve a limited portion of traffic, such as port-related freight movement or international border crossings. However, in most cases, the implementations have had a significant impact on local truck traffic. CUTR visited six special truck facilities in Newark, New Jersey; Boston, Massachusetts; New Orleans, Louisiana; and Laredo, Texas, and met with agency officials responsible for planning, management, and operation. The purpose of these visits was to identify specific conditions that led to their construction and to document lessons learned during the implementation process. The project team gained significant insight into the planning and management of such facilities and was better able to refine the site selection criteria for potential applications in Florida.

RESEARCH METHODOLOGY

CUTR developed a methodology to select sites in Florida that warranted further consideration for truckways or reserved truck lanes. First, criteria to identify potential applications were documented. Important factors included truck crash rates, truck volumes, highway level of service, and percent of trucks in the traffic mix. To pinpoint specific priority areas, researchers constructed two geographic information systems (GIS) models. One model was used for long-range travel, while the other examined scenarios within major metropolitan areas. GIS methods lent themselves particularly well to this research project. The technology allowed researchers to handle large volumes of data, to integrate several data sets, and to weight each variable according to importance and relevance. Primary factors mentioned above could be weighted heavily, while secondary factors could be included at a low enough level to be relevant but not confound the outcomes.

CUTR relied exclusively on FDOT data for the creation of the GIS models. The results consisted of a series of maps that showed specific areas in Florida that warranted further consideration. The entire state, including close-ups of individual areas, was shown. Specifically, critical factors were scored on a scale of 1 to 9. The factors were combined and each ½-squared mile was given a final score. The map was configured to show only squares that received a score of 6 or higher. Individual crash locations were overlaid to show problem areas.

CUTR planned site visits to particular areas in Miami, Jacksonville, Orlando, Ft. Myers, and Tampa, as well as rural locations, and worked with state and local officials to document specific details about local streets and highways. Researchers also investigated the potential use of abandoned rail rights of way and other under-utilized corridors to remove truck traffic from local streets and urban neighborhoods.

Researchers assessed the feasibility of truck-only countermeasures for each site. CUTR developed a methodology to select sites that warranted further consideration for exclusive truck facilities. Specifically, researchers constructed several GIS suitability models to identify “hot spots” based on truck-related crashes; truck volume; percent of trucks; highway level of service; proximity to airports; proximity to seaports; and, proximity to other intermodal facilities. The process of creating and selecting the appropriate suitability model was iterative. Each of the variables was individually considered, and multiple combinations of the models were run.

A typical research approach is to test for conditions that have contributed to a known result. Since there were no conditions under which a long haul truckway has been constructed, the suitability model creation was both iterative and collaborative with FDOT systems planning staff. By using various combinations and weightings of factors, three models were developed and run for the State Highway System in order to identify the most suitable highways for exclusive truck facilities serving the following trip types: “Between Cities,” “Within Cities,” and “Regional Facilities.”

Between Cities Model

The objective of the Between Cities Model was to identify highway corridors that may be deemed suitable for an exclusive facility to move truck traffic from one city to another. Important factors in identifying these types of corridors are the percentage of trucks of total traffic, segments that have high volume of trucks and truck crashes, level of service and percent of trucks. It was determined that a highway’s proximity to a specific local truck traffic generator was far less important than the absolute demand for the movement of freight at a system level. This model attempts to identify the most basic movements of trucks in the state. Truck volume is highly weighted in this model with 75% of the model being attributed to truck volume. Level of service has the second highest weighting with 15%. Percent trucks and truck crash rate were both given a weight of 5%. Six potential corridors emerged from this model.

Within Cities Model

The design of the Within Cities Model attempted to identify those areas where additional truck capacity may be required in urban areas. These areas are sometimes characterized as those links needed in order to move freight the “last mile” to an intermodal facility or distribution center. In this model, proximity to airports with high levels of air cargo activity and seaports are highly valued. Truck mix is becomes more important than the absolute number of trucks as a measure of need.

The Within Cities Model identifies highway segments based on level of service, truck volume, percent trucks, truck crash rates, distance to truck terminals and transfer facilities, airports and seaports. In selecting the areas for further review derived from this model, routes were excluded if they were being addressed in the Between Cities Model. The project team focused on access to local intermodal facilities. Priority was given to those local corridors that connected major intermodal facilities with an emphasis on connectivity to the Interstate System. Three sites emerged for additional examination.

In an attempt to determine if the first two models would fail to capture facilities or needs of a regional nature, a third model was constructed. This Regional Model is a hybrid of the previous two models discussed. It builds off of the Within Cities Model, but gives higher values to some of the factors that are significant in the Between Cities Model; consequently, some of the variables from the Within Cities Model are given less weight. The results of the Regional Model identified no additional highway segments beyond those in the Within Cities Model. Although the scoring of specific highway segments varied, no new roadways emerged.

RESULTS

Potential Sites in Florida

Most of Florida's Interstate System emerged as suitable for consideration of exclusive truck facilities. The most obvious opportunities to create a truck-exclusive facility are where the need seems apparent and the right-of-way is available.

Between Cities Model

Miami to Titusville Portions of the I-95 corridor from Miami (the southern terminus of I-95) to around Titusville scored very high on the Between Cities Model. The highest scores in the 210-mile corridor were in the southern Broward County area. Interstate Route 95 in South Florida serves 4 major seaports in the region and provides a primary commuter route into and out of the major employment centers of Florida's Gold Coast.

With median constraints on the southern end of this corridor, it seems doubtful that an exclusive truck facility could be easily constructed. An alternative, that at first glance seems, to make sense is to try to route long haul trucks to Florida's Turnpike. A serious attempt to do this was conducted in the mid-1990s with little success; however, other potential opportunities do exist. One low cost potential is to make the existing HOV lanes available in the off-peak hours to trucks only. Another is a scheme that would involve operating I-95 and Florida's Turnpike as one facility on the northern part of the corridor providing exclusive, separated lanes for commercial traffic.

Daytona to Jacksonville The I-95 corridor from Daytona to Jacksonville, Florida generally scored high on the Between Cities Model. The highest scores in 89-mile the corridor were on I-295 near the I-10 interchange area. The corridor serves as a commuter route, is part of the intra-regional circulation network, and handles significant seaport, airport and intermodal facility traffic. Upon construction of the "eastern bypass," there seems to be a potential for a shift of significant truck traffic from existing I-295 to the east side of the urban area. This may be one of the only opportunities in the state where taking an existing mixed-use lane and converting it to a truck-only lane may be worth considering.

The additional through traffic capacity that will be available with the completion of the loop provides decision makers with a unique opportunity to provide an incentive for long distance trucks to use one side of the loop or the other. If it was deemed more appropriate that through truck traffic be on the west side of the loop, then an exclusive truck lane, signed and striped, could be instituted at a fairly low cost. The converse is not true given that the new facility is only a four-lane highway.

Naples to Ft. Myers The region served by this 36-mile corridor can be characterized as an area in transition. The traditional agricultural and mining uses to the east of the interstate are giving way to large-scale, low-density residential development. Uses of I-75 seemed to be a mix of interstate through traffic, localized commercial uses, commuter traffic and recreational travelers. The operating characteristics of agricultural and mining trucks are not as appropriate for a high-speed facility as are those for an "over the road" tractor and trailer combination. During times of citrus harvest, the increases in these kinds of vehicles affect the highway's performance. This traffic mix, along with high AADT on a four-lane highway causes this section to rank high among the corridors examined across the state.

The only apparent opportunity in the corridor from Naples to Ft. Myers is to widen I-75 to the "inside" and create exclusive truck lanes. Without the proposed widening now programmed for preliminary engineering, there seems to be sufficient median width (minimum of 80 feet) to consider a fully separated exclusive truck facility. Once the widening is completed, it is doubtful that the corridor will score as high on the GIS model. The remaining median width, after the widening, will still afford a future opportunity to provide exclusive lanes and, perhaps, even a separated facility.

Tampa through Orlando to Daytona Interstate 4 scored highest at its western most end (actually a portion I-275.) This 139-mile corridor changes character dramatically, heavy with commuter and recreational traffic for most of its length. It also serves as one of only a few through freeway routes in Orlando and Tampa. With the Port of Tampa on one end, massive distribution and significant manufacturing uses in Polk County, and the intense development of all kinds in the greater Orlando area, this corridor will continue to present challenges to transportation professionals.

Opportunities for facilitating easier truck movement on the Tampa end of the corridor will be discussed in a section addressing the "Within Cities" findings that follows. While the corridor is long and very complex, some opportunities may present themselves to help in the movement of trucks. The "take a lane" option does not seem feasible, and the median is not of an adequate and consistent width across the entire corridor to consider a simple solution. It is possible that a "total transportation corridor" could emerge as a viable future solution to the growing demands for this corridor and could include accommodation for an exclusive truck facility.

Venice to the Florida State Line Interstate Route 75 from Venice north to the Florida/Georgia State line was another long distance corridor that rated a high score. The longest of the corridors identified, it scored highest at three locations (Venice, I-4 and U.S. 27) along its 270 miles. Interstate Route 75 serves both a heavy demand for interstate through movement as well as handling significant commuter traffic around the Tampa and Ocala areas. Its interchanges with Interstate Route 10, U.S. Route 301, and Florida's Turnpike are all critical linkages for truck traffic.

Like most of the other corridors examined, the highway median is rapidly being consumed for "mixed use" lane capacity additions. Given that the northern section of I-75 may be able to be widened once more within the existing right of way, and the truck mix is in this area is one of the highest found in the study, the "last widening" should be considered for exclusive truck use.

Lake City to Jacksonville The Interstate Route 10 corridor from Lake City to Jacksonville (60 miles) was, overall, on the lower end of the highest scoring highways on the GIS Between Cities Model. Interstate Route 10 provides the primary east-west access across all of northern Florida. I-10's 369 miles connect Pensacola, Tallahassee, and Jacksonville with significant truck interchange points and links the ports of Pensacola, Panama City, and Jacksonville to rest of the state and to the states west of Florida. Throughout its length, the corridor has sufficient median width to accommodate even a separated facility within the existing right of way. Few highway overpasses exist that would require modification, and little vertical curvature exists throughout this portion of I-10.

Within Cities Model

The scores for the Within Cities Model were lower than the Between Cities scores; however, this is not to suggest the importance of the routes identified by this model are less critical than those identified in the Between Cities Model. The different variables used and their associated weightings account for these differences. As in the Between Cities Model, the Within Cities scores are a ranking of relativity, that is, the scores represent a highway or highway segment's position to all other highways on the State Highway System. Based on the model scores, the areas of Miami, Jacksonville, and Tampa were examined more closely for potential opportunities to enhance freight mobility through the use of exclusive truck facilities.

This model attempts to find areas of need to carry freight “the last mile.” While much attention is usually given to through and interstate movements of freight, a common critical constraint is moving from an intermodal transfer point to a higher level of the transportation system.

Miami This area includes parts of Miami and Fort Lauderdale. The presence of the ports and airports in the region contributes to the high scores in the area. Around the Miami International Airport, the highest scores occur on I-95 south of the Palmetto Parkway interchange south to the East-West Expressway. The intense distribution activity that has developed west of the Miami International Airport generates significant truck traffic. The ability of this traffic to move to and from the major sea- and airport facilities of Miami is impeded by the lack of any free flow east to west facility.

The concept of a truck tunnel in and out of the Port of Miami has been studied for some time and would alleviate some of the congestion depending on its western terminus. Although extremely expensive and not easily constructed, perhaps an elevated facility on either the east-west toll road SR 112 or SR 836 for use by automobiles with the existing at-grade lanes reserved for trucks is viable for, at least, study.

Tampa The difference in this look at Tampa is the relatively high scoring and length of the corridor leading out of the port area toward the interstate. These characteristics, combined with the examination of the I-75 corridor in the Between Cities Model, seem to indicate the need for more direct expressway access to the area around Tampa’s port. Currently, truck traffic moving to and from the port that is destined for all points other than west, must wind its way through the local system. Local studies of this condition are underway. Perhaps special accommodation for Port of Tampa truck traffic could be incorporated into any adopted design. This could potentially remove additional truck traffic from city streets.

Jacksonville The model for Jacksonville indicates that the northwest section of Interstate 295 scores very high with truck volume and percentage. The site-specific need that this model attempts to locate seems to be for the U.S. 1 area from the port activity along Tallyrand Avenue to I-95. The opportunities outlined in the Between Cities discussion of the Jacksonville area would seem to have little potential impact on what appears to be a local access issue. This would be required before any recommendation could be made for this area, particularly given that the model used in this study only dealt with state highways. The nature of the Tallyrand access area requires detail for the local street system.

CONCLUSIONS

Most of Florida’s Interstate System emerged as the most suitable highways for consideration of exclusive truck facilities. The most obvious opportunities to create a truck-exclusive facility are where the need seems apparent and the right of way exists to create new lanes for a facility as opposed to “taking” a lane from existing users. An ideal separated facility would provide for ease of passing and adequate shoulders for disabled trucks. This kind of a facility, if it were to be constructed in the median, would most appropriately be situated in areas where interchanges are far enough apart to avoid the long weave sections that would be required for entering and exiting trucks and require approximately 60-feet of right of way. This “separate facility” type seems to fit only the Interstate 10 corridor west of Interstate 295. Although the interchange spacing seems appropriate on Interstate 75 north of Tampa, long sections of the northern part of the corridor have insufficient median.

Although many agencies have and are studying exclusive roadways for trucks, the only facility close to a true truckway is the 33.5-mile, “dual-dual” section of the New Jersey Turnpike. Although there are sections of Florida’s Interstate System that rival the highest traffic sections of the New Jersey Turnpike, the percent of trucks in these areas is lower than the 15 percent average that New Jersey reports; however, with the continued growth in all traffic, and the demand for truck movement not appearing to cease any time soon, the traffic profiles will approach those of New Jersey. From public policy and public perception standpoints, it may more advisable to create traffic separation by excluding trucks from "express lanes." The precedent for truck lane restrictions is already set. This approach also advantages both constituencies, while avoiding the perception that heavy public investment is being made only for one industry.

A system-wide approach to looking at this issue may present some additional opportunities not specifically addressed in the methodology employed in this study. Without the benefit of detailed origin and destination information for commercial traffic, it is difficult to understand how much of the demand for truck capacity on a particular route is a function of the fact that an interstate exists to facilitate movement. The most efficient way to serve the distribution of traffic, or most commodities requiring a fixed infrastructure, is by way of a grid. It may be prudent to give consideration to creating a system of "truck-friendly" highways to make any desired movement more efficient. The system could rely on existing state highways and minimize the need for new construction on new location. Future improvements to all of these facilities could be made with major truck movements in mind. The "truck grid" or backbone could evolve over time within the context of a plan to provide maximum connectivity and alternatives to the congested urban sections of the interstate system.

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Foamed Asphalt Stabilized Reclaimed Asphalt Pavement: A Promising Technology for Mid-Western Roads

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ABSTRACT

Recycling of the materials obtained from the milling of asphalt pavements, known as RAP (Reclaimed Asphalt Pavement), involves mixing RAP with asphalt cement/emulsion and aggregates in definite proportions to produce a new asphalt concrete mix or cold-in place recycled mixture. However, in many cases, the RAP is unusable because it is not uniform (i.e. it may originate from different sources) or the underlying pavement does not provide adequate structural support. One solution to this inadequate support problem is construction of a base with full depth reclamation (FDR) materials stabilized with foamed asphalt. The process is also suitable for moist materials since the moisture is needed to accomplish base compaction. A research project was initiated at Kansas State University to estimate the structural contribution of the foamed asphalt stabilized bases in a typical pavement structure. Four pavement test sections, three with foamed asphalt stabilized bases and one with conventional crushed stone base, were constructed at the Civil Infrastructure Systems Laboratory (CISL). Falling Weight Deflectometer (FWD) tests were conducted before accelerating loading of these test sections. The layer moduli were backcalculated from the FWD deflection data and the structural layer coefficients were estimated following 1993 AASHTO Design Guide and other methodologies. The results show that the estimated structural layer coefficient of the foamed asphalt stabilized FDR base materials is 0.18. This indicates the promise of this recycled base material in pavement construction.

Key words: asphalt pavement—cold-in-place—foamed asphalt

INTRODUCTION

Cold-in-place recycling is gaining recognition and popularity worldwide as a cost effective method of rehabilitating distressed asphalt pavements (*1*). In-place recycling requires the use of specially designed recycling machines with a mixing chamber. While the milling operation is taking place in the front part of the machine, the milled material passes through a mixing chamber where it is mixed with the stabilizing agent (lime, fly ash, bitumen emulsion, foamed bitumen or cement slurry). The mixture is then placed on the milled pavement and compacted. The process is carried out in a single-pass operation. The use of foamed bitumen as a stabilizing agent is not a new idea. Csanyi (1956) investigated the possibility of using the foamed asphalt as a binder for soil stabilization (*2*). Figure 1 shows the schematic of the asphalt foaming process. Foaming of the asphalt reduces its viscosity considerably and has shown to increase adhesion properties making it well suited for mixing with cold and moist aggregates. No chemical reaction is involved, only the physical properties of the asphalt are temporarily altered. When the cold water comes into contact with the hot asphalt, it turns into steam and then gets trapped in the asphalt as thousands of tiny bubbles. After a few minutes, the asphalt will regain its original properties once the steam evaporates.

The first reported use of foamed asphalt dates back to 1957 on an Iowa county road. Several other field applications were also reported including projects in Arizona (1960) and in Nipawin, Canada (1960-1962). The original process consisted of injecting high-pressure steam, at controlled pressure and temperature, into heated penetration-grade asphalt cement. This required special equipment on the job site, such as, a boiler and was not very practical. In 1968, Mobil Oil Australia modified the original process by adding cold water rather than steam, into a stream of hot asphalt in a low-pressure system (*4*). This made the process much more practical and economical. The foam was created within an expansion chamber after which it was dispersed through a series of nozzles, onto the aggregate mass. However, the nozzles were prone to blockage, and the manufacturer could not control the foam characteristics. Recently, Wirtgen GmbH of Germany, Soter of Canada, and CMI of Oklahoma City have developed new equipment for producing foamed asphalt.

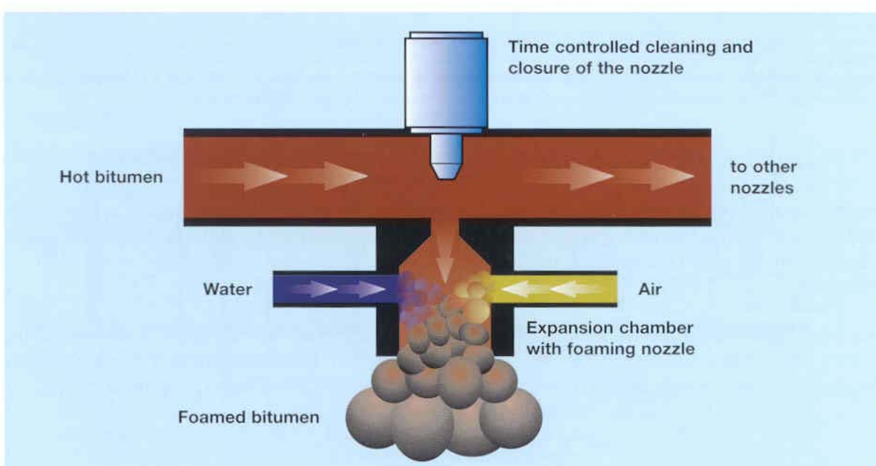

FIGURE 1. Schematic of the Foamed Asphalt Production (*3*)

Stabilization of RAP material with foamed asphalt has been tried in the United States and abroad. Roberts et al. (*5*) compared the performance of stabilized RAP material with foamed asphalt with those treated with cut-back asphalt and asphalt emulsion in the laboratory. Macaronne (*6*), Lancaster (*7*) and Ramanujam (*8*) reported successful stabilization with foamed asphalt of RAP material, both in-plant and in-place, in Australia. Van der Walt (*9*) have reported the use of foamed asphalt to stabilize RAP material in South Africa. Van Wijk (*10*) reported the successful use of foamed asphalt stabilization of the RAP material on two road sections in Indiana. The RAP material was obtained by milling only the top five inches of a distressed asphalt pavement and was, therefore, not contaminated with aggregates or soil.

In full-depth reclamation (FDR) of asphalt pavements, the salvaged material would contain not only RAP, but also aggregates from the granular base and in some case, soil from the subgrade. Thus it would be useful to determine if the foamed-asphalt stabilization is effective for the FDR materials, and what are the structural properties of this new material as bases in a pavement system.

OBJECTIVES

The goal of this research was to evaluate the structural performance of foamed asphalt stabilized base layers obtained from full-depth reclamation of an existing asphalt pavement. The objective was accomplished by doing structural testing on the flexible pavements with foamed asphalt stabilized bases.

MIX DESIGN

The laboratory mix design for the foamed asphalt stabilized base materials was done at the IADOT Central Materials Laboratory in Ames, Iowa. Sample aggregates, reclaimed asphalt pavement (RAP), soil, and the PG binder to be used in this project were shipped to IADOT to develop the mixture design. A Wirtgen Foamed Bitumen Laboratory Plant (WLB 10) was used in the mix design process. The optimum water content for foaming was found to be at 3% water injection rate at a binder temperature of 160˚C (320˚F). The added asphalt content at which the soaked indirect tensile strength of the mixture was maximum was taken as the design asphalt content. For this mixture, the design asphalt content was found to be 3%. Details of the mixture design can be found elsewhere (*11*).

TEST SECTION LAYOUT AND CONSTRUCTION

The test bed at CISL consists of two six feet deep pits, the North Pit (approx. 15 x 20 feet square) and the South pit (approx. 20 x 20 feet square). Four pavement sections were constructed in the pits, two in the North pit ((FDR-6 & FDR-9) and two in the South pit (FDR-12 & AGG-9). Figure 2 shows the schematic of the pavement cross sections. The existing subgrade material was silty clay. After removal and drying, the subgrade soil was recompacted in the pit to a density greater than 90% of the maximum dry density (MDD). Lane AGG-9 was constructed with a nine-inch granular base. The material used in this base is classified as an AB-3 by the current KDOT specifications and consists of crushed limestone materials. The material has an MDD of 128 pcf at optimum moisture content of 10%. The granular base was compacted in three lifts, each having a thickness of three inches. Compaction was done using the vibratory plate compactor and very high densities were obtained.

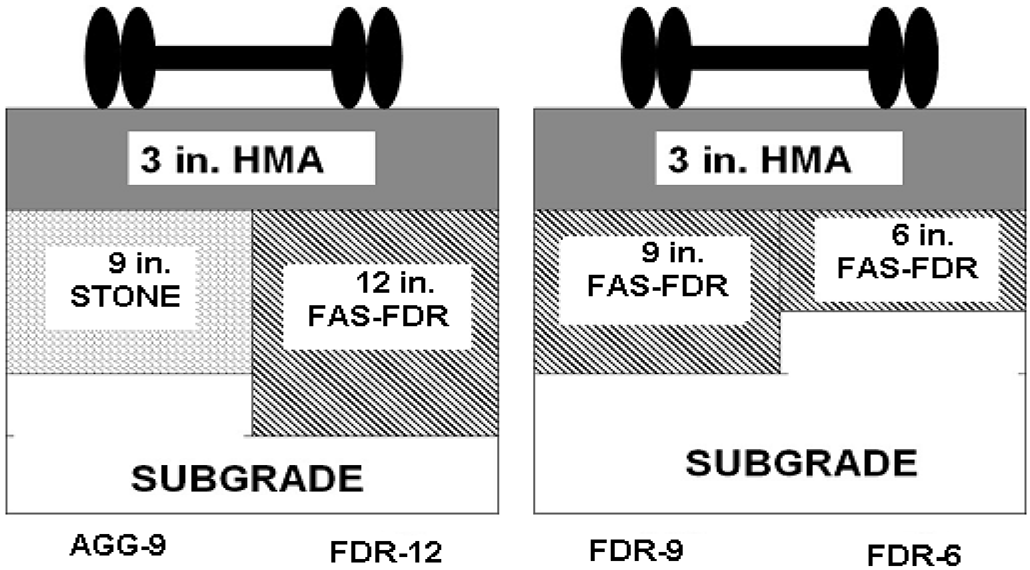

FIGURE 2. Cross Section of the Pavement Sections

Production of Foamed Asphalt Stabilized Material

The foamed asphalt stabilized base material was produced on the grounds of CISL in a portable plant of Wirtgen GmbH. of Germany. The plant consisted of a two-bin aggregate blending system and a chamber for mixing foamed asphalt with the full depth reclamation material blend. The RAP, aggregate and soil were stockpiled at the site. The soil and the aggregate were preblended, and then that mixture and RAP were fed with a front-end loader. The whole process was carefully controlled with a control panel on the plant. The produced material was collected on a dump truck and stockpiled for later use.

Construction of Foamed Asphalt Stabilized Base

The stockpiled stabilized material was transferred into the pit in the CISL with a bucket. Enough material was transferred at a time so that, after compaction, a three-inch layer of compacted FAS-FDR material will result. The material was raked to have a plane surface, and then compaction was done with a jumping jack-type compactor. The in-place density was monitored with a nuclear gage and was found to be satisfactory.

Construction of the Asphalt Concrete Surface Layer

The 3-inch asphalt layer above the base was placed in one lift on all lanes in two pits. The compaction was done with a steel-wheeled vibratory roller. The asphalt layer consisted of a ½ in. (12.5 mm) nominal maximum size Superpave mixture. The combined aggregate gradation (dry) of this mixture, designated as SM-12.5B in Kansas, passes below the maximum density line in the sand sizes.

Falling Weight Deflectometer (FWD) Testing

FWD testing was performed by KDOT personnel on two separate dates before loading began on these sections. The tests were performed at six stations on each test lane as shown in Figure 3.

For stations 1, 2 and 3 the geophones were oriented toward the East. For stations 4, 5 and 6 the geophones were oriented toward the West. Stations 3 and 4 were at the same location, in the center of the lane, but the geophones were directed to the East for station 3 and to the West for station 4. The FWD testing sequence consisted of three drops at 6,000 lbs load level followed by five drops at 9,000 lbs load level. The seven geophones were placed at 0, 8, 12, 18, 24, 36 and 60 inches from the center of the FWD loading plate. The deflections recorded for the last drop at 6,000-lb load level and the last two drops at the 9,000 lb-load level were used to backcalculate the elastic moduli of the pavement layers.

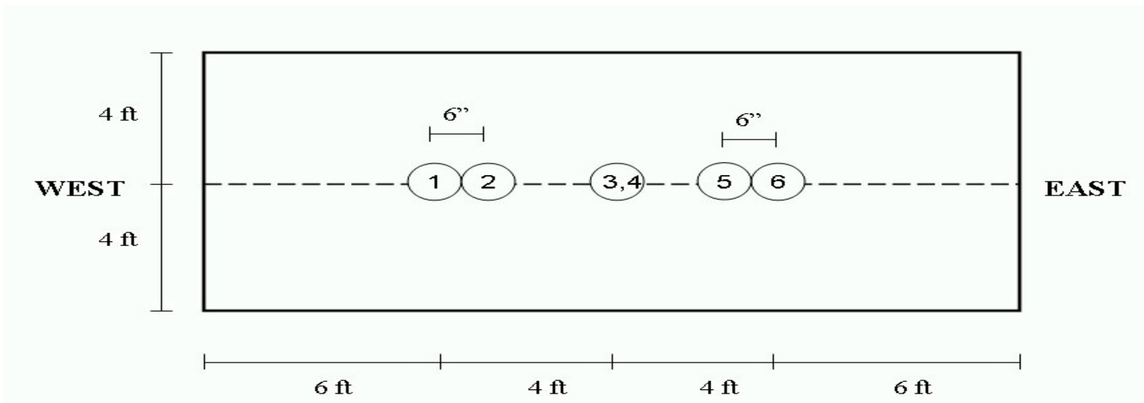

FIGURE 3. Location of the FWD Test Stations

Backcalculation of Layer Moduli from the FWD Deflections

The backcalculation analysis was performed using MODULUS 4.0 backcalculation program. The backcalculated asphalt layer moduli were not corrected to the standard temperature of 68°F since the temperature at the bottom of the asphalt layer varied only between 67°F and 72° F during FWD tests. The average values of the backcalculated moduli, backcalculated at the 9,000-lbs load level, for the six FWD stations are plotted for each pavement layer in Figure 4. Figure 4 indicates that the backcalculated asphalt layer moduli had large variabilities for the six FWD test stations. Moduli are also quite different for the four pavement sections, despite the fact that the same HMA mix was used in paving. This large variation can be attributed to the fact that the asphalt layer thickness of the constructed pavements varied along the pavement sections. Some variabilities between the test drops were also observed. Figure 4 also indicates that the backcalculated modulus for the foamed asphalt stabilized FDR material is always higher that the backcalculated moduli for the AB-3 granular base material. This indicates that the stiffer foamed asphalt stabilized FDR material base may assure a better protection to the soil subgrade than the conventional AB-3 granular base. The average backcalculated subgrade soil modulus is close to 15,000 psi for lanes FDR-6, FDR-9 and AGG-9. Higher values of around 20,000 psi were obtained for the lane FDR-12.

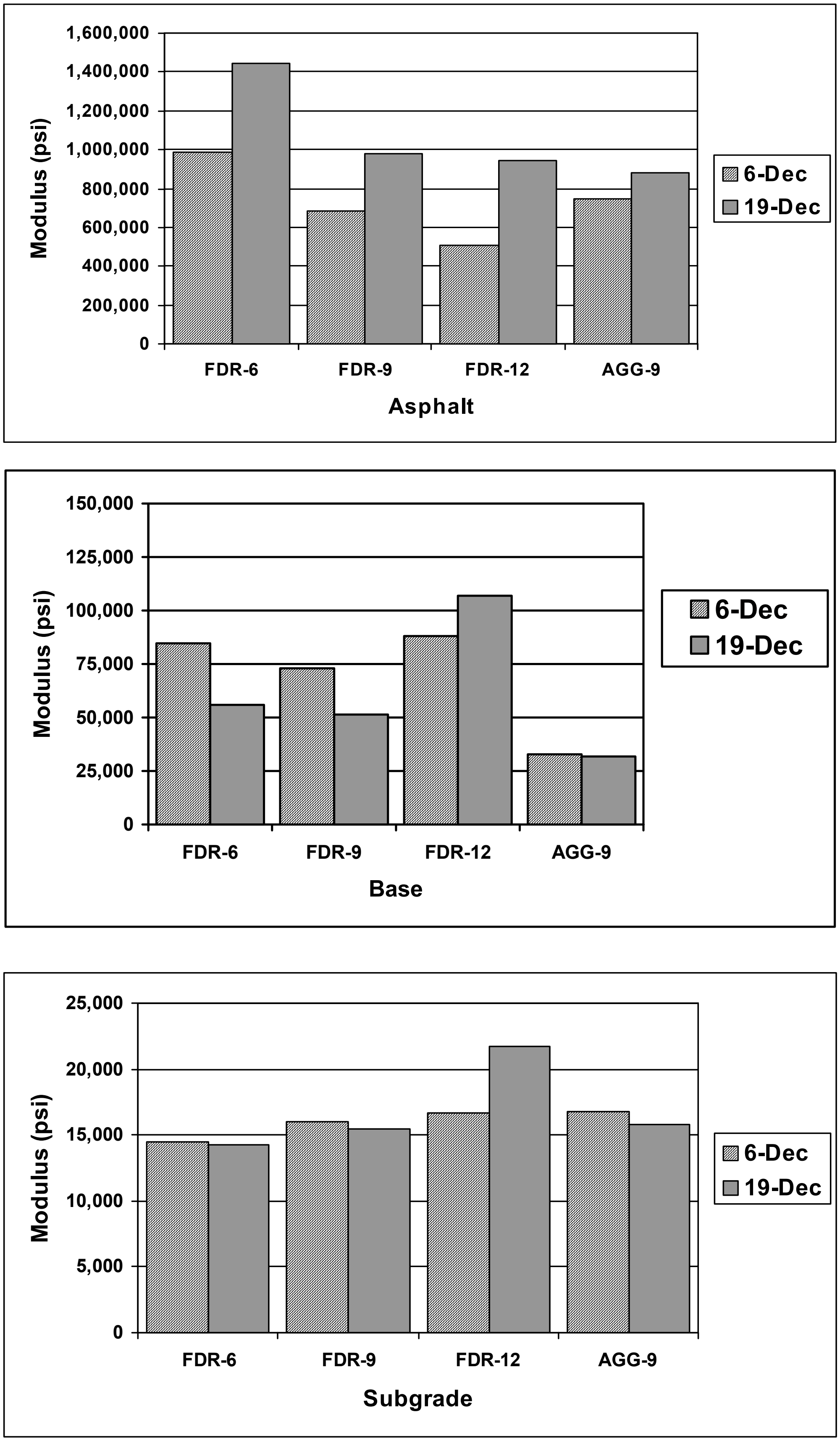

FIGURE 4. Backcalculated Layer Moduli

BASE LAYER STRUCTURAL PERFORMANCE EVALUATION

Currently, there is no standard method for the determination of layer coefficients. Several methods have been used by different investigators to determine layer coefficients for certain paving materials (*12*, *13*, *14*). In this study, the AASHTO Design Method (*15*) and the Equal Mechanistic Approach were followed to determine the structural layer coefficient of the foamed asphalt stabilized FDR materials for base. In both approaches, backcalculated layer moduli values were used to determine the layer coefficient values.

AASHTO provides the following general equation for Structural Number (SN) reflecting relative structural contribution (using coefficients (a_i) and thickness (D_i)) and assuming no effect of drainage:

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 + a_4 D_4 \quad (1)$$

Because the pavements structures in this study had only two layers on top of the subgrade soil, the Structural Number can be computed as:

$$SN = a_1 D_1 + a_2 D_2 \quad (2)$$

Assuming that the asphalt concrete layer had a thickness of h1= 3 inches, and a typical structural layer coefficient for the asphalt concrete would be 0.42 (i.e. a1=0.42), the structural layer coefficient for the base layer material can be computed as:

$$a_2 = [SN_{eff} – 0.42 * 3.0] / D_2 \quad (3)$$

where

a_2 - the structural layer coefficient for the base layer material;
SN_{eff} - the effective structural number; and
D_2 - the thickness of the base layer, in inches

The effective structural number (SN_{eff}) can be computed from the following equation given in the 1993 AASHTO Design Guide (*15*):

$$SN_{eff} = 0.0045 * D * E_p^{1/3} \quad (4)$$

where

D = total thickness of all pavement layers above subgrade (inch), and
E_p = effective modulus of the pavement layers above subgrade (psi)

In equation (4), E_p is determined after computing the backcalculated subgrade modulus (M_r) value. The AASHTO algorithm for determining M_r suggests that M_r be calculated from a single deflection measurement at a distance sufficiently large enough so that the point falls outside the stress bulb at the subgrade-pavement interface and the measured deflection is solely due to the subgrade deformation. The following equation is used to calculate the M_r value:

$$M_r = (0.24 P) / (d_r)*r \quad (5)$$

where

M_r = backcalculated subgrade resilient modulus;

P = applied load;
d_r = deflection at a distance r from the center of the load; and
r = distance from the center of the load.

To use a particular sensor deflection for estimating the subgrade resilient modulus, the sensor location must be far enough so that it corresponds to the deflection of the subgrade only, but also be close enough so that it is not too small to be measured accurately. AASHTO further suggests that the minimum distance be determined by the radius of the stress bulb (a_e) at the subgrade-pavement interface. This is accomplished by choosing the 3rd or 4th sensor arbitrarily and checking whether it falls outside a radial distance of $0.7a_e$ from the center of the load or not.

The calculated M_r value is used to calculate the equivalent pavement modulus, E_p that satisfies the equation:

$$d_0 = 1.5 * p * a * \left\{ \frac{1}{M_r * [1+(D/a*(E_p/M_r)^{1/3})^2]^{0.5}} + \frac{1 - 1/\sqrt{[1+(D/a)^2]}}{E_p} \right\} \quad (6)$$

where
d_0 = the temperature corrected (68˚F) central deflection, in inches;
p = load pressure, in psi;
a = load plate radius, in inches; and
M_r = subgrade resilient modulus, in psi.

The deflections used in the calculations for the base layer structural coefficients are those measured corresponding to the last drop at the 9,000 lbs load level, on December 6 and 19, 2001, before any loading was applied. The estimated structural layer coefficients of the base layer material for all FWD test stations were computed and then averaged for each lane. The average value of the layer coefficient for the foamed asphalt stabilized FDR base material is: FDR-6 – 0.16; FDR-9 – 0.15; and FDR-12 – 0.217. The overall average computed for the three lanes with foamed asphalt stabilized FDR material, $a_2 = 0.1756$.

The average value of the computed structural layer coefficient for the AB-3 granular base is 0.137, almost same as the value of 0.14 used by KDOT for the AB-3 granular base for the structural design of flexible pavements in Kansas. It is then reasonable to apply a simple linear correction to estimate the structural layer coefficient for the foamed asphalt stabilized FDR base material:

$$a_2 = 0.1756 \times (0.14 / 0.137) = 0.179 \quad (5)$$

Thus, the Falling Weight Deflectometer tests performed on the constructed pavements resulted in recommended structural layer coefficient for the foamed asphalt stabilized FDR base material of **0.18**.

CONCLUSIONS

Based on this study following conclusions can be made:

1. The foamed asphalt stabilized FDR material is a uniform material that can be placed and compacted easily, and can be efficiently used as base material in flexible pavements.

2. The effective structural number computed from the FWD deflections measured on the as-constructed pavements suggested a structural layer coefficient of 0.18 for the foamed asphalt stabilized FDR base material.

ACKNOWLEDGMENTS

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Cost Comparison of Treatments Used to Maintain or Upgrade Aggregate Roads

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ABSTRACT

This paper describes an investigation that will give Minnesota counties, cities, and townships information to make informed decisions on when it may be economically advantageous to upgrade and pave aggregate roads. The investigation will also provide resources for local governments to explain to the public why certain maintenance or construction techniques and policy decisions are made.

The research effort is based on the spending used to maintain low-volume roads found in the annual reports of certain counties in Minnesota. The reviewed activities include maintenance grading, regraveling, dust control/stabilization, reconstruction/regrading, paving, and associated maintenance activities.

The expected end product is a set of relationships that can be modified to address local conditions, which will include a cumulative maintenance cost per mile. These relationships are expected to show how the maintenance costs of aggregate roads, lightly surfaced roads, and hot-mix asphalt roads vary with the traffic, age, and type of surface. This relationship will also be used as a tool to assist in decisions about whether or not to upgrade an aggregate road to a bound surface.

Key words: gravel roads—low-volume roads

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INTRODUCTION

Counties, cities, and townships are often faced with the decision on how to best approach the maintenance of aggregate roads and when to upgrade them. Currently most of the information available for decision making on the costs, standards, and performance of different options is not specific to the upper Midwest. For example, Australian road agencies use economic evaluation of gravel roads to assist in their decision to upgrade a low-volume road (*1*). This study examines maintenance costs for various types of road surfaces found in Minnesota and identifies possible threshold values for upgrading low-volume roads.

This paper is a progress report of the research on the cost of maintaining low-volume roads in Minnesota. The goals of the research are to provide the tools needed to make decisions about upgrading low-volume roads. The research will use information from certain counties in Minnesota to estimate the costs to construct, maintain, and rehabilitate low-volume roads within the state of Minnesota.

OBJECTIVE

This project has the objective of identifying the methods and costs of maintaining and upgrading an aggregate road. The costs are from the viewpoint of a public maintenance entity using its own forces.

As an example, initially a road has an aggregate surface with a low-traffic volume. As traffic increases, so do the routine maintenance costs (*2*). At some point, it may be advantageous to improve the road by paving it, which would reduce the routine maintenance costs. Figure1 shows a case where a gravel road is maintained regularly by regraveling (shown as the steps in the graph); if the traffic on this road increases with time, the maintenance costs will also increase with time. To view the effect of upgrading the road to a hard surfaced road, a rehabilitation option is shown with the initial rehabilitation expense in gray. In this case the rehabilitation reduces the annual routine maintenance costs, shown by the solid bold line. The research goal is to determine at what point it is cost effective to upgrade the road so the initial rehabilitation cost, shown in gray, is less than the savings, shown in the hatched area, which then justifies an upgrade in the road surface. It is understood that the time value of money will be considered in this analysis. A review of current maintenance costs will provide a method to identify the threshold where a change in the surface would be beneficial, as in Figure 1.

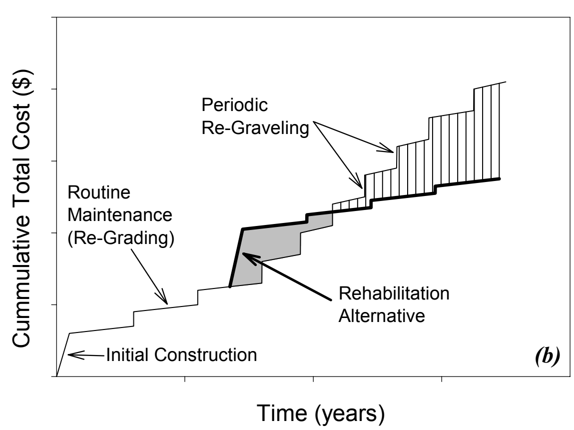

FIGURE 1. Cumulative Maintenance Cost vs. Time for a Specific Road

Cost comparisons will include the following types of roads found in Minnesota: hot mix asphalt (HMA), lightly surfaced roads, portland cement concrete (PCC), stabilizers/dust control products, and natural surfacing aggregate. Research team members' experiences with national/international practices in maintaining and upgrading low-volume roads will be recommended, if applicable. The authors intend to develop a process that will give counties, cities, and townships information to make informed decisions on the type of upgrade and time it may be economically advantageous to upgrade an aggregate road. Included will be methods used to upgrade a road with information on when each treatment would be appropriate.

DATA COLLECTION

The initial data review phase was done by a visit to Waseca and Olmsted Counties. During the initial visit, Waseca County provided an annual report that included a detailed summary their of maintenance costs by route.

The Minnesota Department of Transportation (MnDOT) State Aid Office had paper reports from 1997 to 2001 for some of the Minnesota counties. Of those reports, 40 percent provided information similar to the information found in Waseca County. Some reports had even more detail. The maintenance costs in the annual reports are grouped by funding source:

- County State Aid Highways (CSAH)
- County Roads (funded entirely by county funds)
- Municipal Roads

This study used CSAH and County Road information because most aggregate roads in the county system would fit one of these categories. For each road the maintenance costs were split into five main categories (see Table 1).

TABLE 1. Categories of Maintenance Activities

Routine Maintenance	**Repairs and Replacements**
Smoothing Surface*	Reshaping*
Minor Surface Repair*	Resurfacing**
Cleaning Culverts & Ditches	Culverts, Bridges, Guardrails
Brush & Weed Control	Washouts
Snow & Ice Removal	
Traffic Services & Signs	
Betterments	**Special Work**
New Culverts, Rails, or Tiling	Dust Treatments*
Cuts & Fills	Mud Jacking & Frost Boils*
Seeding & Sodding	**Special Agreements**
Bituminous Treatments***	

* Costs related to routine maintenance of road surface.
** Costs related to periodic maintenance of road surface.
*** Cost can be for routine or periodic maintenance of the road surface.

Some of the cost categories are affected by the choice of road surface and some are not. In the research we are only interested in costs affected by choice of surfaces. Some costs (like snow and ice removal) may be partly affected by the surface. For simplicity, it is assumed that costs such as snow and ice removal are not influenced by the surface type.

The other source of data was a set of county traffic maps that are used to determine the individual road cost (cost/mile/vehicle/day). The maps are prepared and provided to the counties once every four years by the state of Minnesota. The average daily traffic (ADT) on the maps is based on segments that have uniform traffic volumes. This does not necessarily coincide with changes in pavement type, thus making analysis difficult.

DATA REVIEW AND FINDINGS

Waseca County Data Analysis

The initial review of Waseca County data provided us with a snapshot of the kind of information that could be expected to be used in this study. The initial analysis was performed based on specific roads that did not change surface type within the reviewed time. The roads selected for review are shown in Table 2, and their cumulative maintenance costs are shown in Figure 2.

As seen in Figure 2, the conventional wisdom is that the maintenance cost of gravel roads increases with traffic was correct in this case. County Road 26, a high-volume gravel road, has the greatest total maintenance cost compared to the other roads over the same time.

Results of Initial Data Analysis of Other Counties

The initial review of four other counties provided an average total maintenance cost/mile as shown in Table 3 and Figures 3 and 4. The rest of this paper reviews the results of the analysis of the four counties. The final report for this project will include a more extensive review of more counties.

A review of Figures 3 and 4 shows the maintenance costs/mile for County D are much lower than the three other counties. Based on the data available at this time in the research, there is no explanation why the maintenance costs/mile are much less than the other counties. There possibly may have been recording errors when the cost reports were done. The cause of the low maintenance cost will be investigated in the next phase of the research when the counties are interviewed.

TABLE 2. Waseca County Roads Reviewed

Road	Length of Road	Surface	ADT	Classification
County Road 16	2.6 miles	Bituminous	225	Low-volume bituminous
County Road 7	4.1 miles	Bituminous	1200	High-volume bituminous
County Road 71	2.0 miles	Gravel	60	Low-volume gravel
County Road 26	5.6 miles	Gravel	130	High-volume gravel
County Road 27	2.4 miles	Concrete	800	Low-volume concrete

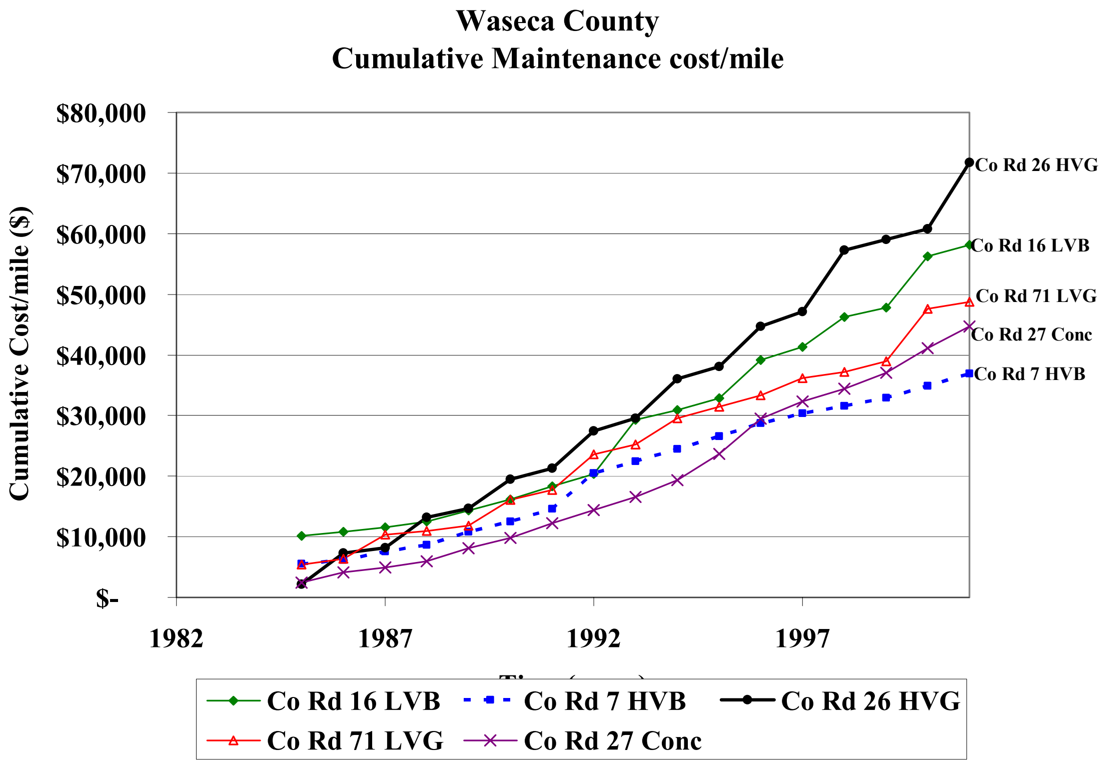

FIGURE 2. Waseca County Cumulative Total Costs/Mile vs. Years, Based on ADT

TABLE 3. Maintenance Cost/Mile by Road Type in the Reviewed Counties

County	Road Type	Miles	Total Maintenance Cost/Mile	Total Cost/Mile of Activities Influenced by Surface Type
A	Gravel	313	$3,250	$1,863
	Bituminous	189	$2,437	$638
B	Gravel	228	$2,526	$1,456
	Bituminous	442	$2,853	$1,320
C	Gravel	297	$3,413	$2,004
	Bituminous	426	$3,699	$2,105
D	Gravel	64	$395	$273
	Bituminous	198	$540	$210

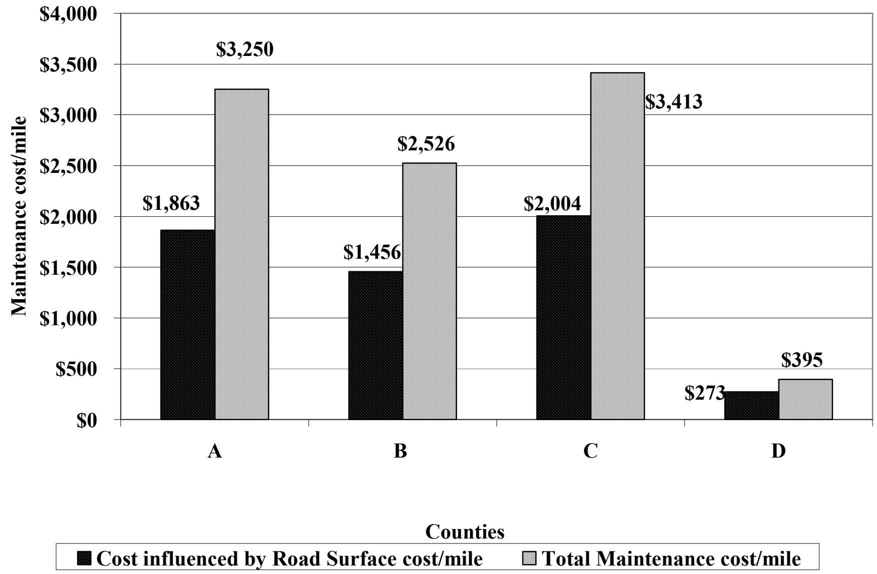

FIGURE 3. Average Maintenance Cost/Mile for Gravel Roads in Reviewed Counties

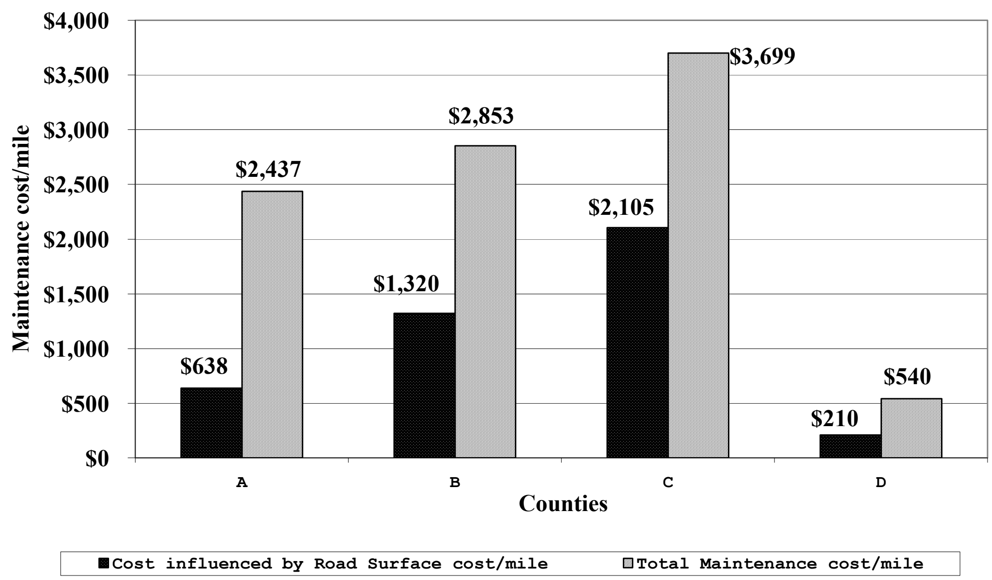

FIGURE 4. Average Maintenance Cost/Mile for Bituminous Roads in Reviewed Counties

Figure 5 shows the total maintenance cost/mile for gravel roads ranges from $1,380 to $5,452 per mile and from $1,785 to $6,055 per mile for bituminous roads, depending on the year. This cost variation could be caused by other maintenance activities not directly influenced by the road surface, such as brush and weed control; this variation is being investigated further. A review of the maintenance activities that influence the annual cost the most was performed and is shown in Figures 6 and 7.

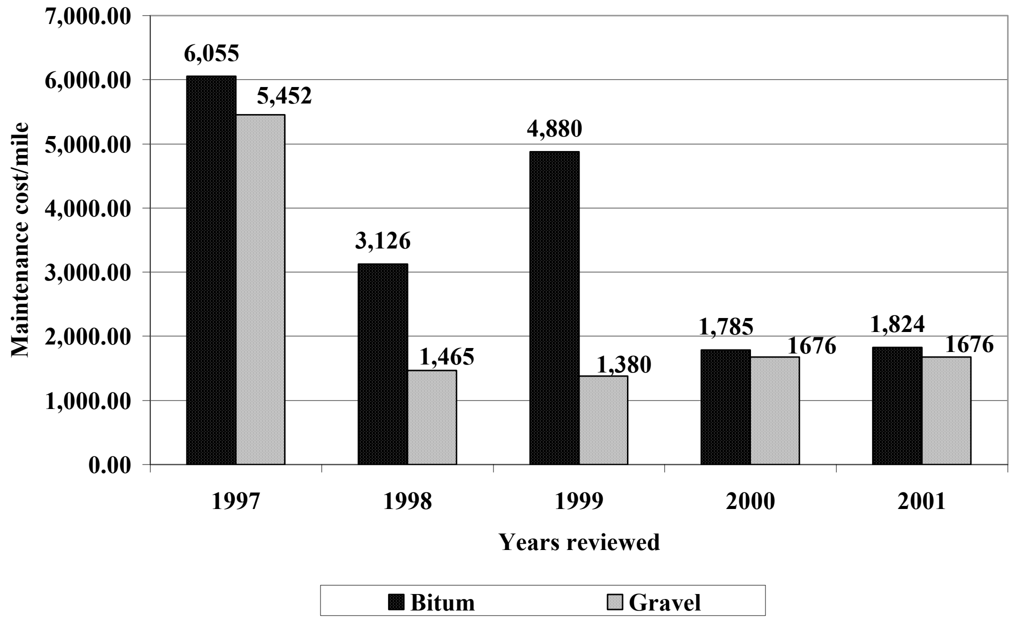

FIGURE 5. Total Average Maintenance Cost/Mile for Years from 1997 to 2001

Figure 6 shows, for gravel roads, resurfacing is the greatest portion of the total maintenance cost/mile at 43 percent, followed by smoothing surface at 17 percent and snow and ice removal at 11 percent. A combination of several other maintenance activities accounts for 24 percent of the total maintenance cost/mile.

Figure 7 shows there are five maintenance categories instead of four that represent more than 10 percent of the total maintenance cost/mile. The most influential costs/mile are snow and ice removal at 21 percent, minor surface repair at 17 percent, resurfacing at 15 percent, bituminous treatment at 12 percent, and other maintenance activities at 33 percent.

Notice the costs not related to the type of pavement surface (labeled other maintenance activities) exceed 20 percent of the total maintenance cost/mile. Another observation made from these two pie charts is that snow and ice removal is a greater cost/mile for hard surface roads than for gravel roads. This higher cost might be explained by the greater amount of time spent clearing snow and ice from bituminous roads compared to gravel roads and the use of sand and salt on bituminous roads compared to not using those materials on gravel roads. This finding brings up the point that when a road is upgraded to a hard surfaced road the cost of snow and ice removal increases and should be considered in the decision making process.

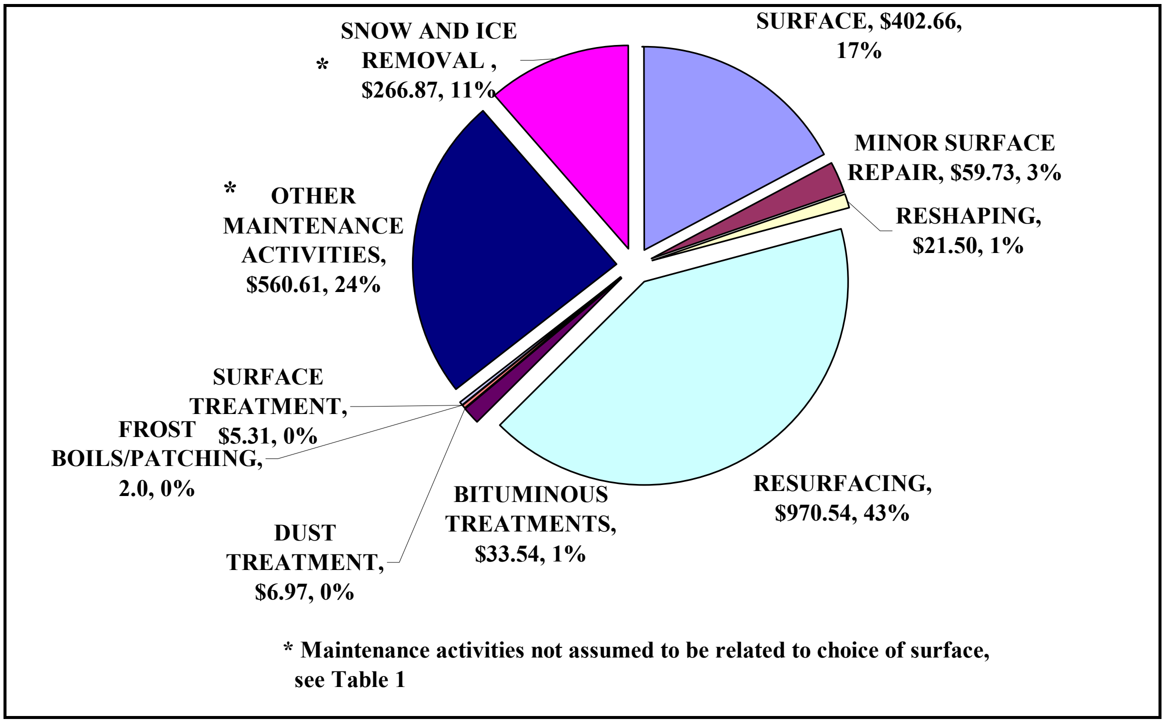

FIGURE 6. Average Cost/Mile for Gravel Road Maintenance Activities

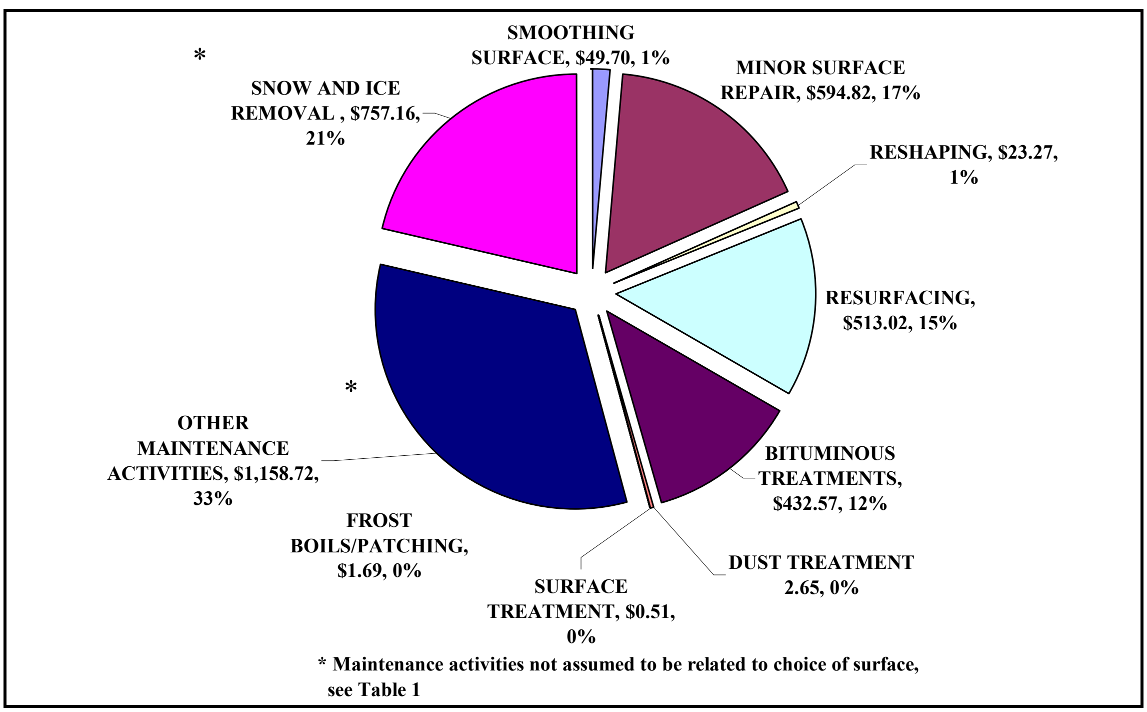

FIGURE 7. Average Cost/Mile for Bituminous Road Maintenance Activities

Results with Traffic

With the use of traffic maps, the variable of the ADT on each segment of road was added to the data set. The ADTs were grouped in the categories show in Table 4 to identify the relationship between traffic level and maintenance costs.

TABLE 4. Average Daily Traffic Range Groups

0–49	125–149	301–999
50–74	150–199	1,000 and up
75–99	200–249	
100–124	250–300	

From Figure 8 it is evident the maintenance cost/mile of gravel roads is greater than the maintenance cost/mile of bituminous roads when the ADT is above 100. Also, the maintenance cost/mile of gravel roads increases considerably when the ADT is greater than 200. The bar chart in Figure 8 provides valuable information on when to upgrade a gravel road to a hard surfaced road based on traffic range and total maintenance cost/mile.

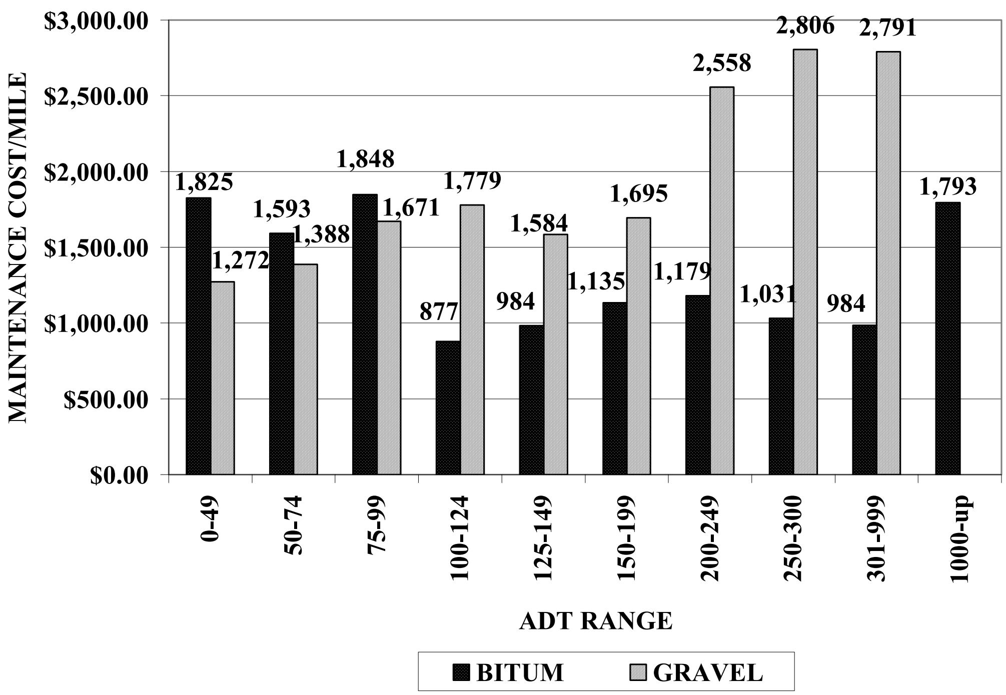

FIGURE 8. Average Maintenance Cost/Mile vs. ADT Range (surface type relevant costs only)

CONCLUSIONS

The study at this point has provided information to give rough averages and ranges for the total maintenance cost/mile of low-volume roads. During the research it was found that the data collected might not be comparable unless it is known how the counties classify their costs. The main questions to be asked now are follows:

- Do counties have a standard practice in classifying cost?
- Can the results be generalized and a database created?

In order to obtain answers to these questions, the researchers will conduct interviews with county engineers and continue the investigation about whether or not to upgrade a gravel road. The final result of the study will be a decision aid for those making decisions about upgrading gravel roads.

ACKNOWLEDGMENTS

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The research project advisory committee includes the following members:

David Fricke, Minnesota Association of Townships
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Keith Kile, Birch Lake Township Supervisor
Joel Ulring, St. Louis County Engineer
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US Experience with Centerline Rumble Strips on Two-Lane Roads: Pattern Research and North American Usage

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ABSTRACT

Shoulder rumble strips are widely used throughout the United States (US), including Kansas. However, Kansas has several miles of two-lane highway with no shoulder. These highways have a number of single vehicle run-of-the-road crashes and crashes from cars going across the centerline and colliding with on-coming vehicles. Some US states have been experimenting with centerline rumble strips. The Kansas Department of Transportation (KDOT) wishes to install and evaluate several miles of centerline rumble strips (CLRS) and contracted with Kansas State University (KSU) to survey other states and summarize their experience, select the best rumble strip pattern and to develop a research design to evaluate KDOT field test installations. The paper will summarize the findings from a nationwide survey. It will describe the study to select the best rumble strip pattern. The field installation and research design to evaluate driver reaction and motorist acceptance will also be presented.

Key words: centerline rumble strips—mitigating cross over crashes—rumble strips

INTRODUCTION

Modeled after shoulder rumble strips, centerline rumble strips are placed between opposing lanes of traffic to alert drivers that they have crossed over into the path of oncoming traffic. The purpose of this research is to determine which rumble strip pattern will be most effective at accomplishing this task, determine driver reaction and motorist acceptance and, long term, determine whether these centerline rumble strips will indeed reduce the number of crossover accidents in the locations where they are installed.

The purpose of rumble strips is to provide motorists with an audible and tactile warning that their vehicle is approaching a decision point of critical importance to safety or that their vehicle has partially or completely left the road or their lane. Rumble strips can be installed either on the traveled surface of the roadway or the roadway shoulder. Rumble strips placed on the traveled surface are warning devices intended to alert drivers to the possible need to take some action (*1*).

CURRENT STATUS

The use of rumble strips on shoulder highways to warn drivers that they are leaving (or have left) the traveled surface is becoming increasingly popular. Today shoulder rumble strips are being used on rural and urban highways throughout the United States (US) as a method to reduce drift-off-the-roadway accidents, and their effectiveness has produced as high as a 60 percent reduction for these types of accidents in some installations (*2*).

The study covered in this paper is still in progress and the authors are currently updating a survey of North American states and provinces but have not synthesized the results as the data collection phase is still in progress. Also, the Kansas test sections, originally scheduled to be completed in 2001, have only recently been completed (July 2003). Thus, this paper can cover only the testing and analysis of the Kansas test patterns, a brief summary of a published report of a Colorado DOT study, controversy over CLRS in Colorado, their status in Canada and methodology to be used to evaluate the recently completed Kansas test sections.

INITIAL NORTH AMERICAN SURVEY

Initially, a phone survey was conducted in the Fall of 1999 of the DOT of various states with centerline rumble strips (CLRS) in place. The states involved in this survey were Colorado, Arizona, California, Pennsylvania, Oregon, and Washington. Its purpose was to accumulate and analyze data regarding the types and dimensions of centerline rumble strips being installed in these locations and any problems or concerns they raised. This was followed by a more formal survey that was sent to all 50 states and all Canadian provinces. This survey was written to address the following questions:

- Are centerline rumble strips in use?
- How were they constructed (milled or rolled)?
- What are their dimensions? (width, length, depth)
- What pattern type was chosen?
- Are they located in all zones or only in double yellow ‘no passing’ zones?

- How long have they been in use?
- Has any data been gathered?
- What type of research was conducted on that data?
- What were the results?

This survey produced 23 responses. Florida, Michigan, South Dakota, New Hampshire, Virginia, North Carolina, Missouri, Illinois, New York, Indiana, Texas, Wisconsin, Utah, and Nova Scotia, Canada, all were either considering installations or asked for additional information and results. California, Oregon, Massachusetts, Washington, Arizona, Colorado, Connecticut, Pennsylvania, and Alberta, Canada, responded that they had centerline rumbles strips installed at various locations. This information can be seen in Table 1.

TABLE 1. Various Other States' Milled Centerline Rumble Strips

State	Width	Length	Depth	Spacing Between Strips	All Zones or No Pass Only	Comments
California	6.5"	16"	0.5"	Continuous 24"	No Pass Only	Used with raised thermoplastic striping and reflectors
Washington	6.5"	16"	0.5"	Continuous 12"	No Pass Only	Markings installed over strips
	6.5"	16"	0.5"	Continuous 24"	No Pass Only	Markings installed over strips
Oregon	7"	16"	0.63"	Continuous 12"	No Pass Only	Used with 4' median
Arizona	6.5"	12"	0.5"	Continuous 12"	All Zones	Markings installed over strips
	6.5"	8"	0.5"	Continuous 12"	All Zones	Narrower to reduce residential noise
	6.5"	5"	0.5"	Continuous 12"	All Zones	Narrower to reduce residential noise
Massachusetts	6.5"	18"	0.5"	Continuous 12"	No Pass Only	Markings installed over strips
Pennsylvania	6.5"	30"	0.5"	Alternating 24 & 48"	No Pass Only	Across centerlines - 12' lanes
	6.5"	16" each	0.5"	Alternating 24 & 48"	No Pass Only	Outside centerlines - 12' lanes
	6.5"	16"	0.5"	Alternating 24 & 48"	No Pass Only	Between centerlines - 12' lanes
	6.5"	18"	0.5"	Alternating 24 & 48"	No Pass Only	Across centerlines - 11' lanes
	6.5"	10" each	0.5"	Alternating 24 & 48"	No Pass Only	Outside centerlines - 11' lanes
	6.5"	12"	0.5"	Alternating 24 & 48"	No Pass Only	Between centerlines - 11' lanes
Colorado	6.5"	12"	0.5"	Continuous 12"	All Zones	Markings installed over strips
Connecticut	6.5"	16"	0.5"	Continuous 12"	No Pass Only	Markings installed over strips
Alberta, Canada	6.5"	12"	0.5"	Continuous 12"	All Zones	Markings installed over strips

Note:
Width - represents dimension parallel to travel surface
Length - represents dimension perpendicular to travel surface

Since the above survey, the Colorado DOT published a report on 17 miles of CLRS that were installed in 1996 on a winding, two-lane mountain highway for evaluation. (*3*) The report stated:

"Comparison of traffic records for similar 44-month periods before and after the installation showed the following:

- Head-on accidents decreased from 18 to 14.
- Sideswipe from opposite directions decreased from 24 to 18.

- Average daily traffic (ADT) increased from 4007 in 1992 to 5661 in 1999.

- Average ADT for the 44-month period before construction was 4628, for the same time span and the same months after construction it was 5463."

Also, positive comments were received from the public, and during the four-year evaluation, no adverse pavement deterioration was noted.

The report went on to "highly recommend" using CLRS in areas where that "have a history" of head-on and sideswipe accidents.

This Colorado report is not without controversy. A study of the same data by a University graduate student came to a different conclusion. (*Davis, V., unpublished data*). Mr. Davis concluded, "Because the data fails to show a statistically significant reduction in crossover accidents for the two time periods future installation of centerline rumble strips based on this data would not be justified."

Also, various bicycle advocates in Colorado are opposed to CLRS on two-lane, mountain roads with no shoulders. According to Bicycle Colorado (2002) (*info@bicyclecolo.org, unpublished data*) they cite the Davis study and claim 400 letters opposed the planned milling of CLRS on two Canyon roads.

Other states are going forward with studies of CLRS. New York, for example, plans "Centerline Audible Roadway Delineator" test installations in each of the state's regions. (*Bray J., unpublished data*) It should be pointed out that New York, as in most states, will use them only in no passing zones, i.e., "where opposing traffic is separated by a double yellow line."

In Canada, the only province using any CLRS is Alberta. In a recent Transportation Association of Canada (TAC), Synthesis of Practice Report (2001). (*4*) The report summarized the testing in Alberta as follows:

"Testing in Alberta has found that many motorists encroach on the centreline of the road and this has resulted in complaints from nearby residents of the excessive noise. Testing of various depths of milled-in rumble strips, retaining the same length and spacing of strips, using three different vehicle types (tractor trailer, pick-up truck, and motorcycle) was completed. For centerline rumble strips, the testing led to the recommendation that rumble strips be at least 8 mm (0.2 inches) deep and 300 mm (7.5 inches) wide. Motorcycles encountered no adverse handling conditions when riding on or over the rumble strips except for braking, which was not an issue since it was unlikely that deceleration would occur entirely within the rumble strip zone. Where significant heavy vehicle use was encountered, a 500 mm (12.05 inches) wide rumble strip created more significant noise and vibration in the cab of a tractor-trailer."

The report recommended the following design dimensions. (*4*)

- The following design dimensions for continuous milled-in centerline rumble strips are appropriate:

- Strip Shape	Rounded
- Strip Width	300 mm (7.5 inches), within painted lines
- Spacing Between Strips	300 mm (7.5 inches)
- Strip Depth	8 ±2 mm (0.2 ± .05 inches)
- Strip Length	175 ±25 mm (4.4 ± .6 inches)

KANSAS PATTERN TESTS

After compiling and analyzing the results of these surveys, it became apparent that there was no standard in the types and dimensions of rumble strips being used and tested. Members of the KSU research team then drafted a proposal for centerline rumble strip testing in the state of Kansas. This proposal called for the evaluation of three different patterns (continuous 12 inches on center, continuous 24 inches on center, and alternating 12 & 24 inches on center) consisting of four different widths each (5, 8, 12, and 16 inches), for a total of 12 test patterns (see Figures 1, 2 and 3). Decibel (dB) and steering wheel vibration (g) levels would then be recorded at the driver's position during a series of tests at various speeds utilizing multiple vehicle types. This testing would attempt to validate an optimum pattern for centerline rumble strip installations in the state of Kansas.

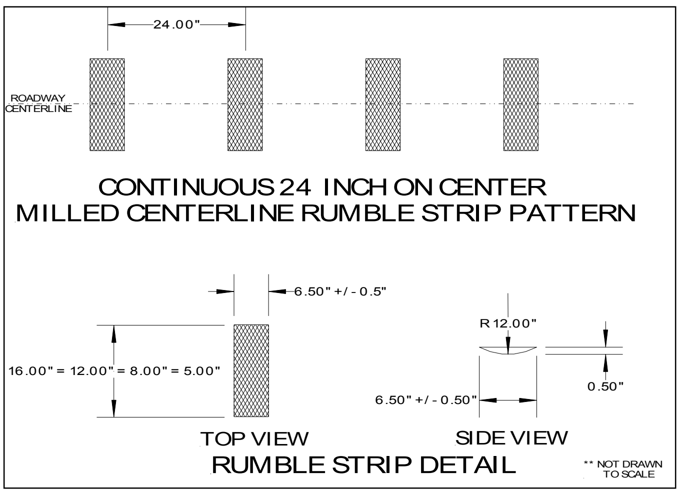

FIGURE 1. Kansas Blueprint of Continuous 12 Inches on Center Pattern

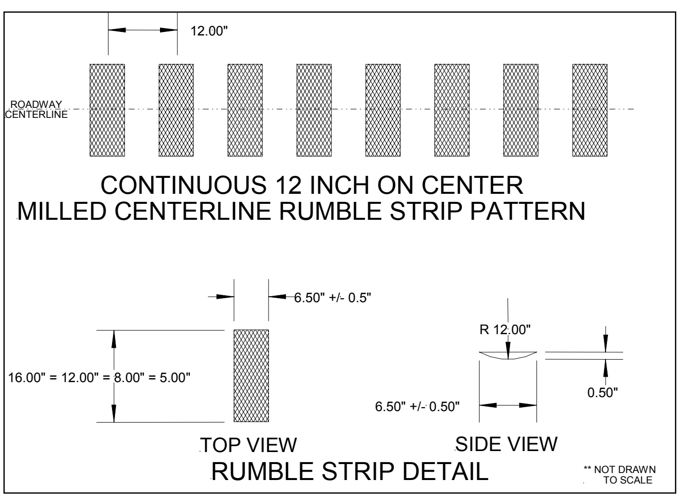

FIGURE 2. Kansas Blueprint of Continuous 24 Inches on Center Pattern

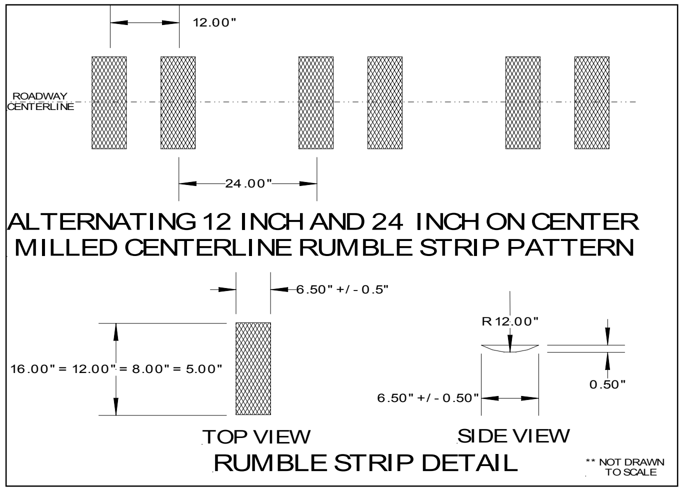

FIGURE 3. Kansas Blueprint of Alternating 12 & 24 Inches on Center Pattern

The Kansas centerline rumble strip test patterns were installed in May 2000 on the southbound lane of Interstate 135 approximately 8 miles south of Salina, Kansas (see Figure 4). The rumble strips were installed in such a way that the general driving public would not contact them under normal driving circumstances.

FIGURE 4 . Dustrol Inc. Milling Machine

Each test pattern section is approximately 1/4 mile in length with 200 feet between test sections. The depth of cut was 0.5 inch on all patterns.

The vehicle tests were conducted using seven vehicles. The seven vehicles consisted of: two large trucks (a 1996 International Harvester 4900 DT 466 dump truck and a 1995 Ford L8000 dump truck), a full-size pickup truck (1991 Chevrolet 2500), a full-size passenger car (1993 Pontiac Bonneville), a compact passenger car (1994 Ford Escort Wagon), a minivan (1995 Ford Aerostar), and a sport utility vehicle (1997 Jeep Cherokee). The vehicles negotiated the rumble strips in such a manner that the driver's side wheels made contact with the rumble strips.

Interior Noise Level And Steering Wheel Vibration Testing

Testing at this site would consist of both interior noise level testing and steering wheel vibration testing. These are tested because sound and touch are the two senses that the rumble strips alert when the driver's visual senses become impaired (falling asleep, becoming distracted, etc.).

Interior noise level testing was conducted by measuring the noise levels generated by the rumble strips as the vehicles passed over each test section. The data was recorded using a Quest Technologies Model Q-300 dosimeter, with a remote microphone clipped to the driver's collar just below the right ear (see Figure 5). This meter operates at 32 samples per second, and displays the highest decibel reading taken during any one-second period. While the tests were conducted, the climate control system, radio, and any other noise-producing sources were turned off, and the windows were rolled up, to eliminate as much background noise as possible. A video camera was used to record the noise levels on the dosimeter as the vehicle passed over the test strips. This data was then transcribed from the videotape, analyzed to locate the proper test strip intervals, and then entered into Microsoft Excel for evaluation. Each vehicle negotiated the rumble strips at 60 mph. This speed was chosen because it is the current speed limit on many of the rural two-lane highways in Kansas.

FIGURE 5. Quest Technologies Q-300 Noise Dosimeter and External Microphone

Results Of Noise Tests

The decibel level average and standard deviation for each vehicle over each test section at 60 mph and 30 mph were calculated. The data was then analyzed for trends. Looked at were trends by pattern type and by rumble strip length. The results showed a trend in pattern type at both 60 mph and 30 mph. Among all of the vehicles tested, the continuous 12 inches on center pattern produced the highest average decibel levels, followed by the alternating 12 & 24 inches on center pattern and finally the continuous 24 inches on center pattern produced the lowest average decibel levels. Further analysis shows that over a given distance, the continuous 12 inches on center pattern has the greatest number of rumble strip indentations, followed by the alternating 12 & 24 inches on center pattern, and finally the continuous 24 inches on center pattern, which has the fewest indentations. Thus, it can be theorized that patterns with higher densities of rumble strip indentations produce higher average decibel levels. As for trends in decibel levels due to rumble strip length, it does appear that the longer rumble strips do generally produce higher average decibel levels, but there is no consistency among the longer lengths. This could be explained as a result of the vehicle tires not remaining in full contact with the shorter rumble strip patterns.

Steering Wheel Vibration Tests

Steering wheel vibration testing was conducted by measuring the vibration levels in the steering wheel of each vehicle that was generated by the rumble strips as the vehicles passed over each test section at 60 mph. The data was recorded using a MicroDAQ Model SA-600 accelerometer, which was firmly attached to the steering wheel of the vehicle by duct tape (see Figure 6). This accelerometer simultaneously samples and internally records the peak acceleration levels on all three axes (X, Y, and Z) at a rate of 4 readings per second. The accelerometer was controlled by MicroDAQ proprietary software by a laptop computer via the serial port. This data stored after each vehicle trial was then downloaded directly to Microsoft Excel for analysis. During testing, the drivers were instructed to maintain as minimal contact with the steering wheel as safely possible, so that the dampening effects caused by touching the steering wheel would be minimized.

FIGURE 6. MicroDAQ SA-600 3-Axis Accelerometer

The alternating 12 & 24 inches on center pattern produced the highest average vibration levels in four of the six remaining vehicles (the 1996 IH 4900 DT 466 Dump Truck removed from analysis) and the second highest average levels in the other two. Conversely, the continuous 24 inches on center pattern had none of the highest vibration levels, and only produced the second highest in two of the six. Thus, the highest overall vibration was produced by the alternating 12 & 24 inches on center pattern, followed by the continuous 12 inches on center pattern, and lowest were produced by the continuous 24 inches on center pattern.

Patterns Selected

Based on the results of the tests conducted, two patterns were chosen for further testing in an actual highway setting, pattern 4 (continuous 12 inches on center, 12 inches long) and pattern 6 (alternating 12 & 24 inches on center, 12 inches long) (see Figures 1 and 2). A section of each pattern was recently installed on a two-lane Kansas highway. Further testing will be conducted throughout the summer and fall of 2003.

FIELD ANALYSIS PLAN

The Kansas Department of Transportation (KDOT) recently installed approximately 50 miles of these two patterns of centerline rumble strips on a two-lane rural highway in June 2003. The Kansas State University (KSU) research team well evaluate motorist reaction, public opinion, study such things as vehicle positioning before and after, and observe drivers actions.

The KSU research team will observe and videotape vehicle positioning as the vehicles travel over tangent and curved sections of the highway where the CLRS have been constructed and compare them with similar sections without CLRS. The KSU research team will set up an interview station and hand out a stamped, self-address questionnaire to approximately 500 drivers. The questionnaire is shown on the next page (see Figure 7).

1. How often do you travel this section of highway? _ daily
_ 2-3 times per week _ weekly _ monthly _ seldom
2. Type of vehicle: _ passenger car _ van _ SUV _ large truck
_ pickup _ motorcycle _ RV _ other (specify) ___________________
3. Did your tires make contact with the centerline rumble strips?
_ continuous pattern _ alternating pattern _ both patterns _ neither
****** IF YOU SELECTED "neither" ON #3, PLEASE SKIP QUESTIONS 4 - 6 ******
4. Which patterns do you feel were adequately loud to gain your attention?
_ continuous pattern _ alternating pattern _ both patterns _ neither
5. Which patterns do you feel adequately vibrated the steering wheel?
_ continuous pattern _ alternating pattern _ both patterns _ neither
6. Overall which patterns of rumble strips would you recommend be installed?
_ continuous pattern _ alternating pattern _ both patterns _ neither
7. Have you ever fallen asleep or dozed off while driving a vehicle? _ no
_ yes, once or twice _ yes, infrequently _ yes, frequently
If "yes", what woke you up? _________________________________
8. Do you think centerline rumble strips will reduce accidents? _ yes _ no
COMMENTS____
NAME/ADDRESS____

FIGURE 7. Questions for Drivers During Field Analysis

CONCLUSIONS

CLRS appear to have the potential of reducing risk of certain types of accidents on two-lane, rural roads. However, their actual benefit has yet to be proven and they are not without controversy. Several states are considering their use but are being cautious and waiting for the results of additional studies.

ACKNOWLEDGEMENTS

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Utilization of LiDAR Technology for Highway Inventory

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ABSTRACT

Collection of roadway inventory data is a basic task for most highway agencies. Maintaining up-to-date information about roadways is essential for design, planning, maintenance, and rehabilitation purposes. For inventory, highway agencies typically rely on field data collection, which is time consuming and subject to limitations such as adverse weather conditions. Recent availability of new and emerging technologies has led to experimentation with these technologies for collection of roadway inventory. This paper is focused on the use of Light Detection And Ranging (LiDAR) technology for collection of roadway inventory. LiDAR uses the same principle as RADAR for collection of information. The LiDAR instrument transmits light beams towards a target and a receiver collects some of the reflected light. The time for the light to travel to the target and back to the receiver is used to determine the distance to the target. Placement of the LiDAR equipment onboard an airplane and directing the light beams at the surface of the Earth allows collection of the Earth's surface profile. The resulting data can be used in obtaining information on certain roadway inventory elements. This paper describes experimentation with LiDAR data collected for Iowa 1 highway, passing through Linn and Johnson Counties, Iowa. Aerial imagery and LiDAR data for the study corridor were merged and analyzed to extract information on highway grade, side slope, and contours as well as stopping and passing sight distances. The results were verified in the field for accuracy by comparing information obtained from LiDAR to the ground truth. These comparisons showed that the information obtained from the LiDAR data closely matched conditions in the field. Overall, this research indicates that where available, LiDAR data can be effectively utilized to obtain certain elements of the roadway inventory. LiDAR is a relatively new and costly source of transportation data. Therefore, additional applications useful to transportation agencies must be developed to justify investment in LiDAR data collection.

Key words: data collection—infrastructure—inventory—lidar—sight distance

INTRODUCTION

All transportation agencies maintain some type of roadway inventory, which is used for a variety of purposes. Maintaining up-to-date information about roadways is essential for design, planning, maintenance, and rehabilitation purposes. Inventory data are collected in different ways depending on the inventory element; the typical methods are field surveying using electronic distance measurement and photogrammetry. Although accurate, these methods are time consuming and labor-intensive. With the availability of new technologies, there is potential for finding more efficient methods of collecting roadway inventory data.

This study focused on evaluating the applicability of Light Detection And Ranging (LiDAR) technology in collection of certain roadway inventory data. LiDAR is a technology that can be used to collect information about a surface by sending and measuring the return time of thousands of light beams per second, which are directed at and then reflected from the surface. Measuring the return time of the light beams allows calculation of distance between the LiDAR instrument and the target. By collecting a great number of reflected light beams, the surface profile of the target can be obtained in the form of a "digital signature." Collection of information on the surface profile of Earth in the form of elevation data is one of several LiDAR applications. The main objective of this study was to utilize LiDAR data to obtain information on certain roadway elements. These elements included: 1) stopping sight distance; 2) passing sight distance; 3) side slope; 4) highway grade; and 5) contours.

A literature review focused on highway inventory, LiDAR technology, and the five inventory elements mentioned above follows this introduction. A section explaining the research methodology and the data used in this research follows the literature review. The next section presents the analysis of the LiDAR data and field validation of the results. The paper concludes with the authors conclusions based on the research results and acknowledgment of research sponsorship.

LITERATURE REVIEW

Highway Inventory

Highway management is a process that deals with several highway-related activities involving planning, design, construction, and operation maintenance (*1*). These activities are associated with maintaining, rehabilitating, and reconstructing/replacing highway assets in an efficient manner. These activities require accurate and up-to-date inventory data on the various roadway elements. Common inventory data include roadway geometry, signs, signals, pavement markings, pavement quality, roadside objects, bridges, and driveways. Roadway inventory data are distinct from other type of data handled by highway agencies in that 1) they are collected on each roadway or a large sample of roadways, rather than being collected for specific projects; 2) they pertain to the roadway and the right-of-way, but not to the surrounding buildings and areas; and 3) they pertain primarily to describing the identity, function, and physical features of the roadway and right-of-way (*2*).

The typical method for collecting and processing many geometric-related inventory data is a labor-intensive. A person drives along the roadway and takes notes regarding the current situation. This person might need to take measurements along the roadside using, for example, a total station to get readings necessary to find the grade or side slopes of the road. The person also checks the safety of the roadway and the availability of safe sight distances. The necessary information and measurements are recorded in a field book and later transferred to the roadway inventory database (*2*). Though most of this information could be available in the original design plans, roadway elements like sight distance might change with time due to construction and vegetation growth. Thus, there is need to update inventory information regularly.

LiDAR TECHNOLOGY

LiDAR technology utilizes the Global Positioning System (GPS), precision inertial navigation systems, laser-range finders, and high speed computing for data collection. LiDAR systems on airborne platforms (e.g., an airplane or helicopter) usually measure the distance between an object the laser beam hits and the airborne platform carrying the system (*3*). Airborne laser mapping instruments are active sensor systems, as opposed to passive imagery such as cameras (*4*). With LIDAR, it is possible to obtain elevation information on large tracts in relatively short time; elevation data obtained with LiDAR can be up to 6-inch accurate. LiDAR system uses the speed of light to determine distance by measuring the time it takes for a light pulse to reflect back from a target to a detector. A laser emitter can send about 5,000 pulses per second, but due to the high speed of light a detector can sense the reflected pulse before the next one is sent (*5*). Following a data collection flight, the data-tapes are transferred to a ground-based computer where a display of recorded data is immediately available (*6*). LiDAR systems produce data that can be used in digital elevation models (DEM). The high density of elevation points provides the possibility to create high-resolution DEM models (*5*). LiDAR has been effectively used in several applications including highway location and design (*7*) and highway safety (*8*).

HIGHWAY ELEMENTS

Sight Distance

According to the AASHTO's "A policy on geometric design of highways and streets" (*9*) (hereafter referred to as the Green Book), sight distance is "the length of the roadway ahead that is visible to the driver. The available sight distance on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path." Adequate sight distance is needed so drivers can avoid striking objects, or when using the opposing lane to pass a slower moving vehicle, and can comfortably merge with cross traffic at an intersection (*10*). Two aspects of sight distance are discussed below: sight distance needed for stopping and the sight distance needed for passing on two-lane undivided highways.

Stopping site distance (SSD) is the distance a vehicle needs to come to a complete stop before hitting an obstruction that a driver sights. SSD should be available for drivers on each lane of the roadway regardless of the type of highway. SSD is the sum of two distances: 1) the distance traversed by the vehicle form the instant the driver sights an object necessitating a stop to the instant brakes are applied, and 2) the distance needed to bring the vehicle to a complete stop after application of the breaks. The computed distances (d) for wet pavements and for various speeds are developed from the following equation:

$d = 1.47Vt + 1.075\ (V^2/a)$

where
t = brake reaction time (assumed 2.5 sec)

V = design speed, mph
a = deceleration rate, ft / s^2.

Passing sight distance (PSD) is the distance necessary for a driver on a two-lane highway to see ahead and overtake, pass, and return to the travel lane without interfering with another vehicle. Passing sight distance is considerably longer than stopping sight distance. If passing is to be accomplished safely, the Green Book suggests that the passing driver should be able to see a sufficient distance ahead, clear of traffic, to complete the passing maneuver without cutting off the passed vehicle before meeting an opposing vehicle that appears during the maneuver. When appropriate, the driver can return to the right lane without completing the pass if opposing traffic is too close when the maneuver is only partially completed.

Side Slope

Highway side slope is an important component of the roadway geometry and safety. It's the section of ground that intersects the shoulder and slopes up or down for a certain distance. Side slopes should be designed to ensure roadway stability and to provide a reasonable opportunity for recovery of an out-of-control-vehicle. Providing a flatter slope between the shoulder edge and the ditch bottom, locating the ditch a little farther from the roadway, or even enclosing short sections of drainage facilities will enhance the safety of the roadside, often at a small increase in cost.

Grade

Grading is an important aspect of cross section design. Highway grade differs depending on the terrain and the topography of the ground. The degree of slope affects the appearance, safety and maintainability of the roadside. When designing a highway, designers choose the grade of the road in accordance with the geometric design standards for the type of road (11). The topography of the land traversed has an impact on the grade chosen for that highway. Maximum and minimum grades depend on the design speed of the highway and on the type of terrain.

Contours

Understanding the ground's contour map is a key element to designing highways. A good contouring plan allows the designers to better understand the topography and geometry of the site and help engineers make a better decision on where exactly to locate the route and what difficulties the might be encountered due to rough topography of an area. It also helps in increasing the likelihood of achieving natural looking slopes and drainage facilities (12). During the initial stages of route planning and with the need for enough ground information designers needs to go back and can consult with the existing records of contour maps that are probably the most important source of preliminary information.

RESEARCH METHODOLOGY AND DATA CHARACTERISTICS

Figure 1 presents the methodology adopted in this research. LiDAR data were merged with geocoded aerial imagery in a GIS to obtain a composite database. This composite database was then analyzed to extract pertinent information. The extracted information was compared to “ground truth” for validation.

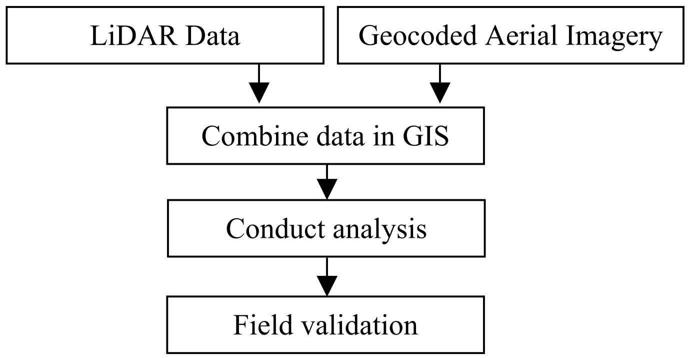

FIGURE 1. Research Methodology

Site Description

The study corridor, passing through Linn and Johnson Counties, Iowa, consisted of the Northern section of IA 1 Solon Bypass (Figure 1). Part of the study corridor is rural in nature; cornfields are viewed on both sides of the road. Some parts have clusters of trees and shrubbery while others have scattered acreages along the route. A small section of the highway is urbanized and passes through the Village of Solon. The corridor receives significant traffic during morning and evening due to its proximity to the University of Iowa.

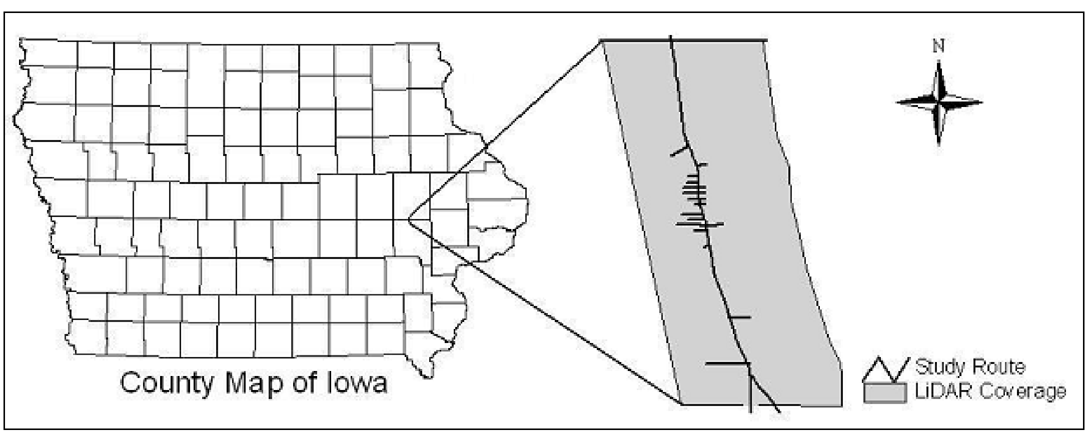

FIGURE 2. Study Corridor

Data

The aerial imagery and LiDAR data for the study corridor were collected on different days. The LiDAR data consisted of both the First-Return and the Last-Return. The former constitutes x, y, z readings for points that LiDAR hits first (e.g. tree canopy), while the latter constitutes last hits of the LiDAR (e.g., returns that penetrate through tree canopy). Aerial images were in Geo-tiff format with a spatial resolution of 2 meters while the LiDAR data were in ASCII comma delimited text file format with an accuracy of 6 inches. The LiDAR point Shapefiles were converted to 3-dimensional Shapefiles by incorporating height information in ArcView and finally to Triangular Irregular Networks (TIN). TIN is a vector data model that uses triangular elements to store surface information. Considerable time was needed for these conversions due to the high density of the LiDAR data.

Data Merging and Compilation

The study area consisted of 12 aerial images and 69 LiDAR bounds. Because all the images combined with all LiDAR bounds would create a large ArcView file that required significant time to retrieve, the aerial images were not joined in one project. Twelve smaller projects were created based on each aerial image. Each project was then overlaid on the appropriate LiDAR bounds.

To superimpose the 69 bounds of LiDAR data correctly on their 12 corresponding aerial images in ArcView, all LiDAR bounds in the text format were converted to Shapefiles and a trial and error procedure was used. The LiDAR bounds were overlaid on the image to see which files corresponded to the opened image.

DATA ANALYSIS AND VALIDATION

The five elements of interest consisted of: 1) stopping sight distance; 2) passing sight distance; 3) side slope; 4) grade; and 5) contour generation. Each element was analyzed separately using the same methodology and dataset. A description of the analysis of each element follows.

Stopping Sight Distance

Given a TIN, ArcView GIS software has a tool to identify any object obstructing the line of sight of an observer. A line of sight is a 3D graphic that is drawn from an observation point to a target on an active TIN in ArcView. The line of sight not only indicates that the target is visible; it also shows which parts of the terrain along its length lie within the observer's field of view (13). With this line of sight tool, it was possible to identify the sections of the route where a driver's line of sight was obstructed. The approximately 15-mile highway stretch under study was divided into 200 ft increments. The stopping sight distance was tested for adequacy at each station along the route. Using the line of sight tool, the line would start at the station and it would be extended along the road for the adequate sight distance needed. The height of the observer's eye and the object were obtained from the Green Book. The height of the driver's eye is estimated to be 3.5 ft and the height of the object to be seen by the driver is 2 ft, equivalent to the tail light height of a passenger car.

Using ArcView, a line of sight between the station and the expected location of the object was drawn. Green line segments indicated visible terrain while red line segments indicated terrain that was not visible. If the target was visible, a blue point (dot) was placed on the line of sight at the obstruction point. Following this method, stopping sight distance was obtained along the whole stretch of IA-1 and the blue points placed on the map would indicate an obstruction. Since the LiDAR data and the aerial images were obtained at different times, it was difficult to identify, after drawing the line of sight, whether the obstacle appearing was an object or it was a vehicle on the road. For this reason all the blue points that appeared on the road were eliminated and the focus was on blue points generated off the road. Figure 4 illustrates the blue points appearing due to an obstruction.

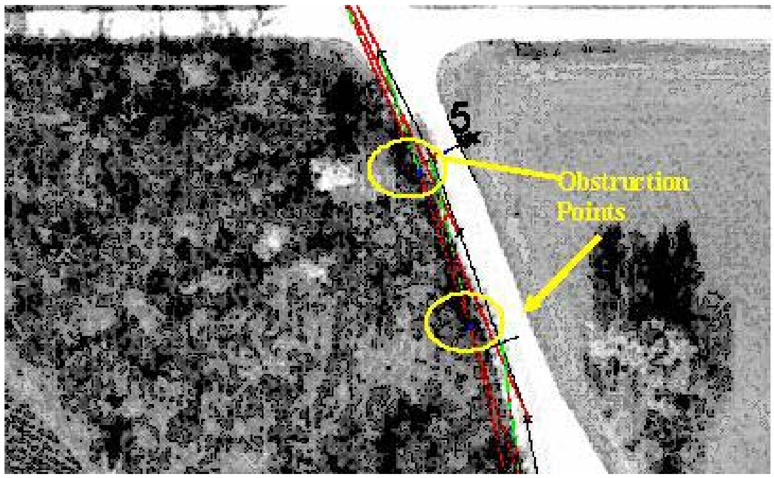

FIGURE 4. Identifying Stopping Sight Distance Obstructions

The analysis of the passing sight distance was similar to the analysis of the stopping sight distance in that the line of sight tool was used. The passing sight distance involved testing the ability of the driver to clearly view a conflicting vehicle on the opposing lane while attempting to overtake a slower vehicle. The line of sight is reciprocal i.e., the driver on the opposing lane should be able to see the vehicle maneuvering to overtake the slower vehicle on the opposite lane. It's very important for both drivers to have a clear view of the opposing lane while going through this process. The passing sight distance is a function of time, acceleration, speed and the difference in speed between the passing vehicle and the passed vehicle. The heights of the observer's eye and the object were obtained from the Green Book For the same reason mentioned in the stopping sight distance analysis, all obstructions appearing on the roadway were eliminated and only the points outside the roadway were taken into consideration.

Side Slope Analysis

In this analysis, side slopes were measured at stations located every 1,000 ft along both sides of the highway. At every station, the elevation of the point of intersection between the outer edge of the shoulder and the ground was found. A horizontal distance of 15 ft was measured from that point and the elevation at that point was measured. Using the identifying tool on the ArcView and with the availability of the TIN files, it was possible to find the elevation on any point on the map.

Grade

LiDAR data are a source of ground elevations and therefore highway grade can be calculated by taking into consideration elevations at different locations along the length of the highway. To calculate grade on IA1, a 500-ft increment was measured along the shoulder of the roadway in each project and the elevation found at both ends of the segment and finally grade calculated by taking the difference between elevations at the two ends of the segment. The segment was considered at the shoulder rather than at the centerline to make field validation easy. Working in the center of the roadway would have involved a high level of risk during the field validation.

Contours

The last highway element of interest was contour generation. Utilizing the Contour tool in ArcView, contour lines at 1.5 ft interval were generated using the available TIN themes.

FIELD VALIDATION AND RESULTS

Approach for Choosing the Field Validation Points

After the analysis was conducted, a set of points was selected for field validation. With the large number of points available, a careful selection of the points was needed. For each inventory element, 10 points were selected for field inspection. The method used to select the points for the side slope, grade, and contours concentrated on choosing points with the highest values in their category to overcome any imprecision caused by the LiDAR inaccuracy that can reach up to 6 inches. The objective was to compare the relative difference between the values obtained by ArcView GIS and between the values obtained in the field rather than checking the exact elevation values. The research team adopted this method due to the difficulty in finding a benchmark with a known elevation along the highway in which all the measurements can refer to.

To validate the results of the highway elements under study, a visit was made to the study area. A team of three persons was equipped with surveying instruments along with a digital camcorder. All five elements were investigated and digital photos were taken to compare and confirm the results obtained from the field. Information about the selected testing points was acquired from the GIS maps and tables were prepared to ensure an easy and quick entry of the collected data.

Sight Distance

Locating the segments of the road that showed an obstruction of the line of sight was the first step in the examination procedure. The station where the line of sight with an obstruction was generated was identified in the field. The team traversed the section starting from the station and driving either the stopping sight or the passing sight distance. The distance was measured using the vehicle’s odometer. The digital camcorder was used while traveling the section to help in recording the field conditions and obtaining still images of the obstruction. Two quick on-spot checks were made to verify the passing possibility for drivers traversing that section. The first check was to look for the correct signage in case of a no passing zone. The second check was to examine the road markings and see whether they comply with the results that were obtained earlier. The main purpose was to check whether the obstructions identified through ArcView existed in the field or not. The data obtained from the field confirmed the presence of all the obstructions detected by the line of sight analysis for both stopping and passing sight distances. Results indicated that 100% of the potential and actual obstructions were correctly identified by the line of sight analysis. Table 2 presents the results of the sight distance validation.

Side Slope

Validation of the side slope along the highway was accomplished by using basic surveying. A Total Station was the main instrument used for finding the elevation. The point to be checked was first located using the method mentioned earlier and then the elevation was found at the point where the shoulder intersects the ground and slopes down, the other reading was taken at a distance of 15 ft away. The field results complied with the results obtained from the GIS maps; Table 3 presents the results of the side slope validation.

Grade

Finding the highway grade was done using a Total Station and surveying. The grade was found along the shoulder of the highway and the sections checked in 500 ft segments. Table 3 presents the results of the side slope validation.

TABLE 2. Summary of Results for Sight Distance Analysis.

Section	GIS Results from LiDAR data		Field validation results	
	Sight distance not obstructed	Sight distance obstructed	Sight distance not obstructed	Sight distance obstructed
1		X		X
2		X		X
3		X		X
4		X		X
5		X		X
6		X		X
7		X		X
8		X		X
9		X		X
10		X		X

TABLE 3. Summary of Results for Side Slope Analysis

Point	Difference in elevation	
	Field difference (ft)	LiDAR difference (ft)
1	3.43	3.12
2	4.25	3.60
3	2.6	3.00
4	5.73	3.3
5	5.11	3.43

TABLE 4. Summary of Results for Grade Analysis

Point	Difference in elevation	
	Field difference (ft)	LiDAR difference (ft)
1	25.15	25.61
2	17.77	19.87
3	4.25	4.41

Contours

The main objective was to find the relative difference between the contour elevation on the GIS map and in the field. The points to be checked were placed on the contour map and then located in the field where the elevation was measured with reference to a fixed point chosen in the area and was assumed to be of zero elevation. The difference was then calculated and compared to the LiDAR analysis. Validation results are presented in Table 5.

TABLE 5. Validation of Contours

Point	Difference in elevation	
	Field difference (ft)	LiDAR difference (ft)
1	6.26	6.6
2	-6.26	-6.2
3	2.83	2.60

CONCLUSIONS AND RECOMMENDATIONS

LiDAR technology offers the potential to collect some roadway inventory elements that are difficult to collect by traditional inventory data collection methods. In this study LiDAR data were utilized to obtain specific roadway inventory elements without physically going to the field (except the airplane flight). As an active remote sensing system, LiDAR does not require the extensive use of labor in surveying big tracts of land. LiDAR is a safer way for data collection since data collectors are not exposed to roadway traffic hazards.

Aerial imagery and LiDAR data for the study corridor were collected and analyzed for extraction of highway stopping sight distance, passing sight distance, grade, side slope, and contours. Field validation was undertaken to check the accuracy of the extracted elements. Results from the analysis of the LiDAR data were close to the ground truth. Based on the results, the authors conclude that LiDAR data can be effectively utilized to collect information on stopping sight distance, passing sight distance, grade, sideslope, and contours.

Future recommendations include obtaining LiDAR data and aerial images simultaneously; this can help analysts detect false obstructions such as on-road vehicles and exclude them from analysis. Conducting cost analysis would help agencies understand the real cost of using LiDAR technology. LiDAR is a relatively new and costly source for obtaining transportation data and the cost of collection might not be justified based on the collection of only five inventory elements. Other applications useful to transportation agencies must be developed to off-set the cost of LiDAR data collection.

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Implications of Wetting-Drying Cycles for the Performance of Pavements Systems

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ABSTRACT

Any construction activity involving compacted fills requires an understanding of unsaturated soil behavior. Unsaturated soils comprise three phases: soil solids and pores filled with water and air, whereas in saturated soils the pores are completely filled with water only. In over half of the world the water table is at a considerable depth, which means that the construction activity occurs within unsaturated soils. In such situations the vast majority of geotechnical problems arise from ground movements caused by wetting and/or drying of the unsaturated soil under load. Construction of highways invariably involves unsaturated compacted geomaterials. Elements of pavement system including foundations and subbase are constructed under unsaturated conditions, which are subject to naturally occurring wetting-drying cycles and/or variation of moisture content due to drainage deficiencies. In this paper, key aspects of wetting-drying cycles and implications of such cycles for the performance of pavement systems are analyzed and discussed.

Key words: pavement system performance—-unsaturated compacted geomaterials—wetting-drying cycles

Note: Preparation of this paper was still in progress at the time of publication; final results will be presented at the symposium.

Effect of Curling on As-Constructed and Early Life Smoothness of PCC Pavements

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ABSTRACT

Pavement smoothness is a key factor in the performance and economics of a pavement facility. This paper presents the results of a study of the effect of curling on as-constructed and early life smoothness of Portland cement concrete pavements (PCCP) in Kansas. Six test sections on three newly built PCCP projects on I-70 and I-135 were selected. Material properties of different layers were collected. As-constructed and periodic (at 4-month intervals) profile measurements were taken by a South Dakota-type profiler on each wheel path of both driving and passing lanes. International Roughness Index (IRI) was used as the smoothness statistic.

A digital method was developed to separate curling from the measured profiles using Fast Fourier Transform (FFT). IRI values were calculated for the curled profile as well as for the profiles without curling. The contribution of curling to the measured smoothness was found to be significant. A set of models was developed to describe curling and early life smoothness in terms of different construction, geometric and climatic variables. The as-constructed and early life curling were found to be significantly affected by the PCCP slab thickness and the stabilized base stiffness. If curling could be minimized then as-constructed smoothness, in terms of IRI, becomes function of the PCCP slab thickness, 28-day compressive strength of concrete, change in plasticity index of subgrade soil after lime treatment, and strength of the base layer. Several recommendations were developed to minimize the effect of curling on as-constructed smoothness.

Key words: pavement curling—pavement materials—portland cement concrete

INTRODUCTION

Smoothness of a newly constructed Portland Cement Concrete Pavement (PCCP) is now a major concern in the highway industry. State highway agencies have recognized pavement smoothness as an important measure of pavement performance. Pavement smoothness is mostly controlled by the longitudinal profile of the road. Smoothness is an important indicator of riding comfort and safety. Several factors contribute to the pavement roughness: built-in construction irregularities, traffic loading, environmental effects, and construction materials (*1*). Construction irregularities can cause variations in the pavement profile from the design profile. Environmental effects such as temperature and moisture gradient across the thickness of slab can cause curling, which in turn, affects the smoothness of the pavement (*2*).

The American Concrete Institute (*3*) defines curling as "the distortion of any originally essentially linear or planar member into a curved shape such as the warping of a slab due to creep or to differences in temperature or moisture content in the zones adjacent to its opposite faces". A concrete slab tends to curl when it is subjected to a temperature and/or moisture gradient across the thickness of the slab. Curling induces stresses in the slab as the pavement is restrained by its weight and the reaction from the subgrade. The thermally induced stress caused by such interaction can be a significant factor in contributing to early pavement cracking (*4*). This may be critical, particularly within a few hours after placement, since concrete in the early stage of hydration may have insufficient strength to prevent cracking. Ytterberg (*2*) reported that curling is caused by drying shrinkage and by moisture and temperature gradients across the thickness of the slabs. Negative drying shrinkage and moisture gradients are usual in slabs on grade and they cause upward curling. Negative moisture gradients and upward curling are increased if the slab is made from high shrinkage concrete, or if the slab is exposed to low humidity air, or if the subgrade or sub-base under the hardened slab has a high moisture content. The most common positive temperature gradient with its downward edge curling is that caused by heat from the sun on the upper slab surface (*2)*. Upward edge curling is caused by negative moisture gradients, can be increased by cold slab surface temperatures or by hotter slab bottom temperatures.

Tremper and Spellman (*5*) made displacement profilograms of a number of highway pavements. They found that upward curling was the dominant condition. The upper portion of a pavement slab is nearly always drier than the bottom part, and that upward curling due to a moisture difference may be offset by wholly or partially in the afternoon by a higher temperature at the top than the bottom. Temperature rise due to solar radiation was not thought to be high enough to produce downward curling in the daytime. At night, the upward curling increased and reached to maximum (*5*).

OBJECTIVES

The objectives of this study was to evaluate and quantify the effect of curling on as-constructed and short-term smoothness of PCC pavements, and to identity the factors that affect curling and roughness, so that the occurrence of curling could be minimized through modifications to the design and/or construction practices.

TEST SECTIONS

Profile data was collected on six (*6*) PCCP sections in Kansas built in the Summer and Fall of 2000. All sections are jointed plain concrete pavements with 5-meter joint spacing and doweled joints, and are located on Interstate routes 70 and 135. All sections have 100 mm stabilized drainable subbase, known as bound drainable base (BDB) in Kansas, and 150 mm lime-treated subgrade. A drainable base is defined as the one with a minimum of 303 m/day (1000 ft/day) permeability. Most of the subgrade materials are fine and plastic. The effect of lime-treatment was variable as indicated by the before- and after-lime treatment plasticity index values shown in Table 1. Subbase stabilization was done with cement and cement-fly ash binder. The 28-day compressive strength of BDB materials was quite variable and varied from 0.90 MPa to 4.44 MPa (shown in Table 1).

TABLE 1. Base and Subgrade Properties

Section	Plasticity Index (%)			Subgrade Material Passing 75-micron Sieve (%)	BDB Compressive Strength (MPa)
	Before Lime Treatment	After Lime Treatment	Change		
PTS-1	22.5	20.0	2.5	96	0.90
PTS-2	20.5	18.5	2.0	96	N/A
STS-1	20.0	N/A	-	90	1.93
STS-2	23.0	N/A	-	90	2.24
TTS-1	26.5	19.5	7.0	97	4.44
TTS-2	20.0	21.5	0.5	97	4.11

Two of the I-70 sections, located near Paxico (PTS-1 and PTS-2), consist of 320 mm concrete slab while the other two on I-70 in Topeka (TTS-1 and TTS-2) have 280 mm slabs. The two test sections on I-135 in Salina (STS-1 and STS-2) have 290 mm concrete slabs. Both I-70 and I-135 are 4-lane divided highways. All test sections consist of 32 continuous slabs (i.e. 160 m long) and are located in one direction. Two different compositions of aggregates were used in the concrete as shown in Table 2. Sixty percent fine and 40% coarse aggregates were used for concrete on the Paxico test section. All other sections had 45% coarse aggregate and 55% fine. Concrete on all sections were air-entrained. The water-cement ratio varies from 0.45 on the Salina test section to 0.49 on the Paxico test section. The Paxico section had the highest 3-day modulus of rupture values (4.1 MPa) of concrete but the lowest 28-day core compressive strength (31.7 MPa).

TABLE 2. Concrete Mix Design Properties

Section	% Aggregate in Mix		% Air	Water-Cement Ratio	Cement Content (kg/m^3)	28-day Core Compressive Strength (MPa)	3-day Modulus of Rupture (MPa)
	Coarse	Fine					
PTS-1	40	60	6.5	0.49	330	31.7	4.10
PTS-2	40	60	6.5	0.49	330	31.7	4.10
STS-1	45	55	7.0	0.45	325	44.5	3.30
STS-2	45	55	7.0	0.45	325	44.5	3.30
TTS-1	45	55	7.5	0.47	335	41.4	3.92
TTS-2	45	55	7.5	0.47	335	41.4	3.92

DATA COLLECTION

Data was collected in different phases of construction. Data collected can be divided into three categories: a) Inventory data; b) climatic data; and c) profile data. Inventory data includes layer and material properties data, such as, Plasticity Index (PI) of subgrade soil (before and after lime treatment), compressive strength of concrete used for the BDB layer, concrete properties etc. as shown in Tables 1 and 2. Inventory data also include road structure and geometry data as well as traffic data in terms cumulative 80-kN (18 kip) Equivalent Single Axle Loads (ESAL). Climatic data includes temperature data of pavement top and bottom during the day of construction (thermocouples were used for this purpose) as well as mean monthly precipitation data and air temperature data.

After construction, profile data was collected periodically as shown in Table 3. As–constructed data was collected two to three weeks after construction before opening the sections to traffic. After the sections were opened to traffic, profile data was collected up to 24 months at approximately every four-month intervals. Profile measurements were done on both wheel paths of both lanes (driving and passing) using a South Dakota-type Profiler and three replicate runs were made. The South Dakota-type profiler used in this study was an International Cybernetics Corporation (ICC) profiler with laser sensors. Profile data was collected at about 75 mm (3 inch) intervals with the profiler operating at highway speed.

TABLE 3. Dates of Profile Data Collection

Pavement Age During Profiling	Date of Data Collection	Season
As-constructed	October 2000	Fall
4-Month	February 2001	Spring
8-Month	July 2001	Summer
12-Month	November 2001	Winter
16-Month	February , 2002	Spring
20-Month	June 2002	Summer
24-Month	November 2002	Winter

DATA ANALYSIS

International Roughness Index (IRI) was used as a summary statistic. IRI values were computed from the profile data using the RoadRuf software developed by the University of Michigan Transportation Research Institute (*6*). As-constructed IRI values for all sections are shown in Figure 1. It is to be noted that these IRI values represent the average IRI of both wheel paths and lanes with three replicate runs on each wheel path. The figure shows that the Paxico test sections had the lowest as-constructed IRI. The other sections had somewhat similar as-constructed IRI values.

Figure 2 shows the variation of the IRI values with respect to time. Almost all sections exhibit definite patterns and some of the variations could be attributed to the seasonal changes. IRI values for section TTS-1 were the highest for all cases. Topeka test sections (TTS-1 & 2) have higher grade than any other sections. It is to be noted here that, 8-month data for PTS-1 and PTS-2 were not available as those sections were used as work-zone during that time period.

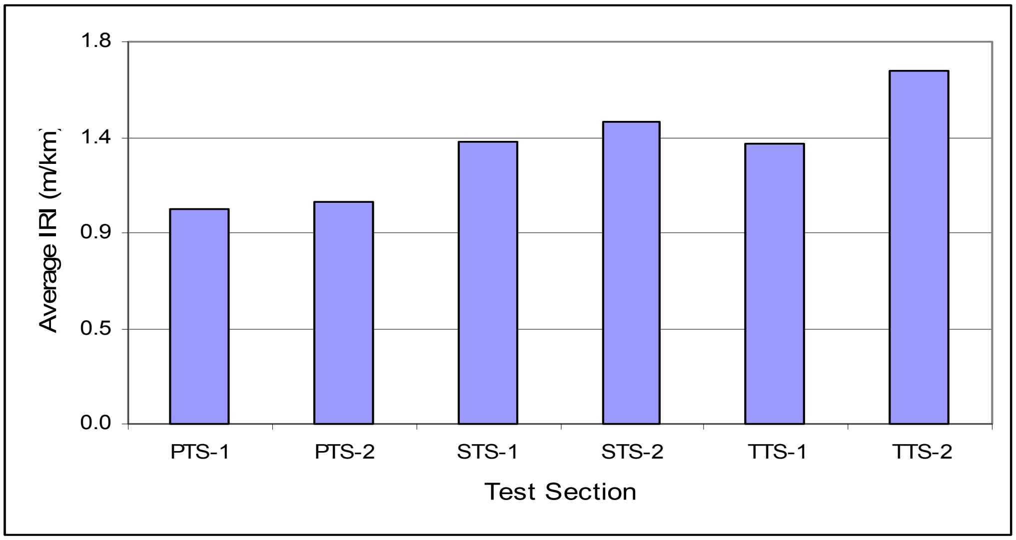


FIGURE 1. As-Constructed IRI Values of Different Sections

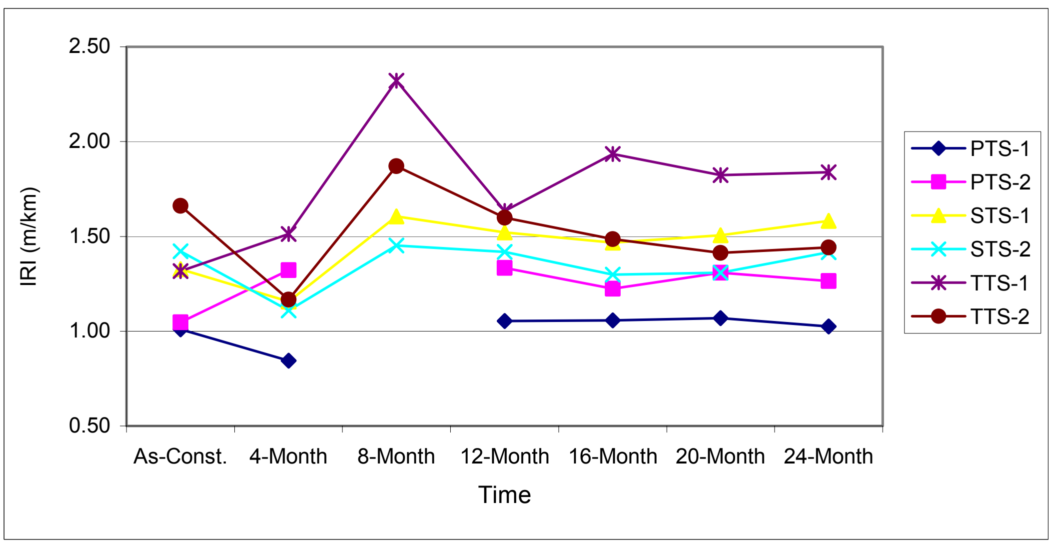
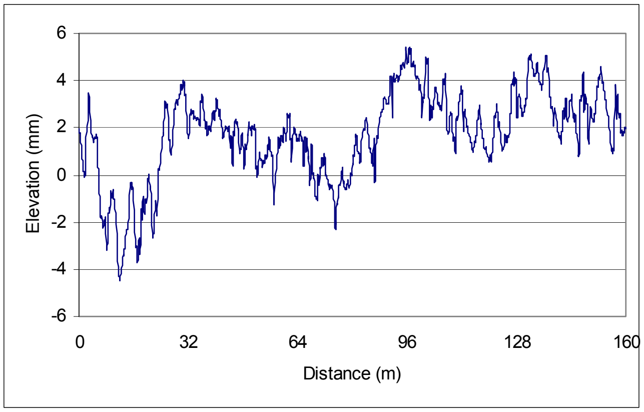


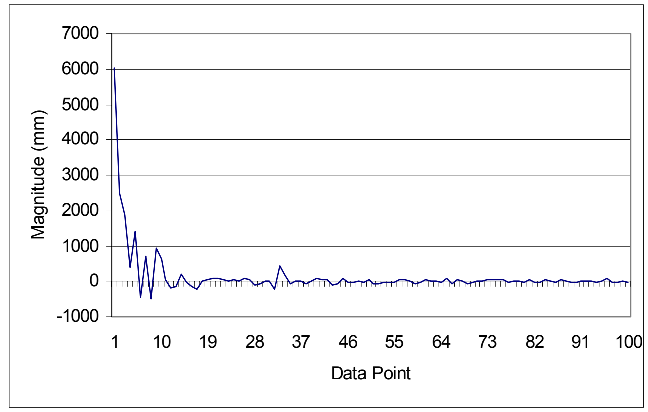
FIGURE 2. Variation of IRI with respect to Time

DIGITAL SEPARATION OF CURLING

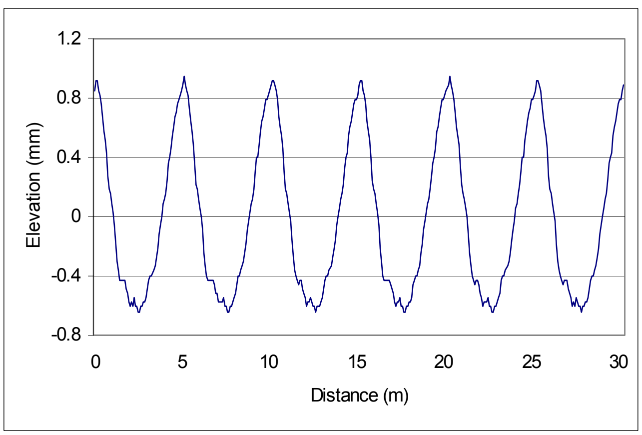
Thus far no universally accepted method has been proposed to identify and separate curling from the profile data. Byrum (*7*) has proposed an empirical method to quantify the effect of as-constructed curling from the roughness data obtained by a K.J. Law 690 DNC high-speed profiler in the LTPP program. Curling was separated digitally based on the Fast Fourier Transform (FFT) of the pavement profile obtained with the South Dakota-type profiler. MATLAB software was used for this purpose. It was assumed that curling is uniform for all slabs. An example of separation of curling from the profile data is presented here for the right wheel path of the driving lane on STS-2.



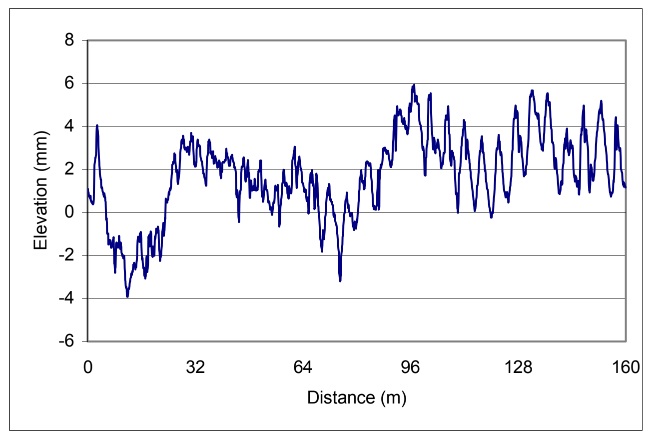
(a) Elevation Plot



(b) FFT of Elevation



(c) Elevation of Curled Profile



(d) Elevation of Profile Without Curling

FIGURE 3. Example of Digital Separation of Curling

Uniform curling was detected using elevation data from the profile of this section. The elevation plot of this section (160 m long) is shown in Figure 3(a). The FFT of these elevation data is shown in Figure 3(b). The figure shows spikes at every 32 data-point intervals. These spikes are the Fourier components of the fundamental frequency. The first distinguishable spike in Figure 3(b) (at 33 data points) represents a wavelength that is most likely caused by curling and also shows the distinct harmonics associated with it. Thus these spikes represent a wavelength of one slab length [5 m = 16.404 ft = (No. of data points (2,100) × sample interval (3 in.))/(33-1)) where 33 is the Fourier coefficient and "1" is the origin indexing used by MATLAB] and all of its harmonics. Then these spikes were separated from the other FFT coefficients. The inverse FFT of these separated spikes resulted in the separated curled profile of the section as shown in Figure 3(c). It shows the profile of six (6) curled slabs. The subtraction of the separated curled profile from the actual elevation profile resulted in the profile due mainly to the construction process (Figure 3(d)). IRI values were calculated for the profiles with and without curling. The IRI values for the original profile and the profile without curling were found to be 1.47 m/km and 1.18 m/km, respectively. The IRI value for the profile data from the separated curled portion (shown in Figure 3(c)) was 0.66 m/km. For this particular section, the contribution of curling was approximately 20% of the total roughness. It is to be noted that the IRI's calculated for the separated curling profile and profile without curling will not add up algebraically equal to the actual profile IRI since the IRI calculation algorithm is nonlinear. This technique was not found to be applicable to some sections. It is possible that some curled slabs do not curl uniformly enough to produce distinguished spikes in the Fourier components at wavelengths that are multiples of the slab length. This is especially true during early life of some concrete pavements.

CONTRIBUTION OF CURLING

Table 4 shows the contributions of curling to roughness for all test sections in the form of percent reduction of IRI values, obtained after subtracting curled profiles from the original profiles. The effect of curling on as-constructed smoothness was the highest for the Topeka test sections. The contribution of curling to roughness was the lowest 16 months after construction. After 20 months, on STS-1 section, curling contributed as much as 39% to the total roughness. It also shows that contributions of curling to roughness during this measurement period were much higher than other measurement periods. Since these measurements were taken during summer time, the contribution of curling to the measured roughness appears to be the highest for the summer months. This is quite plausible given the temperature gradients developed in the PCCP slabs during hot summer days and relatively cooler nights in Kansas.

TABLE 4. Decrease in IRI Values After Separation of Curling for Different Time Period

Section	Reduction in Average IRI Values (%)						
	As-Constructed	4-Month	8-Month	12-Month	16-Month	20-Month	24-Month
PTS-1	5.5	*	#	5.3	2.4	23.8	3.7
PTS-2	4.5	6.4	#	8.1	0.7	26.7	4.7
STS-1	3.7	*	9.5	5.1	6.7	38.9	7.0
STS-2	7.8	*	6.7	10.5	4.0	34.1	7.8
TTS-1	16.5	7.2	2.8	4.7	0.5	11.3	12.3
TTS-2	12.7	*	30.5	5.5	2.32	14.8	15.8

* Digital separation technique didn't work
# Used as work-zone

MULTIPLE REGRESSION ANALYSIS

Multiple regression analysis was used to find the functional relationship between IRI values for curled profiles and profiles without curling at different time periods and significant factors that influence IRI. The general form of the model is:

IRI=a+bX1+cX2+…
(1)

where
X1, X2, .. are the independent variables;
a is the intercept; and
b, c, …. are the correlation coefficients.

Construction, geometric, climatic and traffic data were used as independent variables. Models were selected based on a number of statistical information such as R^2 value, t-test statistic, as well as engineering judgment. Separate models were developed for each time period for the curled profile and the profile without curling. The models for the IRI from the original profiles have been discussed elsewhere (*8*). Table 5 lists the significant independent variables, parameter coefficients, and statistical information for the modes obtained by a statistical software SYSTAT (*9*). No feasible models were obtained for the 24-month measurements.

TABLE 5. Models Derived for the Study

Without Curling			Curling		
Variable	Parameter Estimate	R^2 Value	Variable	Parameter Estimate	R^2 Value
As-constructed IRI					
Intercept	-3.750	0.892	Intercept	-0.400	0.640
THICK	0.053		THICK	0.001	
BDBSR	0.787		BDBSR	0.240	
4-Month IRI					
Intercept	-1.600	0.763	Intercept	-0.452	0.708
THICK	0.009		CSTR	0.021	
CHGPI	-0.011		GRADE	0.006	
BDBSR	0.022				
8-Month IRI					
Intercept	-7.200	0.737	Intercept	2.223	0.639
THICK	0.030		GRADE	0.047	
GRADE	0.292		PASS200	-0.042	
12-Month IRI					
Intercept	0.563	0.607	Intercept	4.566	0.813
CSTR	0.018		THICK	0.002	
GRADE	0.167		GRADE	0.024	
			PASS200	-0.052	
16-Month					
Intercept	0.404	0.756	Intercept	-1.303	0.837
THICK	0.036		CSTR	0.036	
BDBSR	0.432		BDBSR	0.014	
GRADE	0.103		GRADE	0.041	
PASS200	0.012				
20-Month					
Intercept	-0.150	0.732	Intercept	4.934	0.745
THICK	0.006		THICK	0.004	
GRADE	0.238		GRADE	0.022	
			PASS200	-0.061	

THIK: Thickness of concrete slab (in mm);

BDBSR: Compressive strength of BDB layer (in MPa);

CHGPI: Change in Plasticity Index due to subgrade stabilization (in %);

GRADE: Vertical grade (in %);

CSTR: 28-day core compressive strength of concrete (in MPa)

PASS200: Subgrade materials passing 75-micron sieve (in %);

These models show that IRI values from profiles without curling, and curled profiles are influenced by: the thickness of the concrete slab, compressive strength of the BDB layer, 28-day core compressive strength of concrete, change in Plasticity Index of subgrade soil due to lime treatment, vertical grade of the road, and % subgrade materials passing 75-micron sieve. Positive parameter estimates denote that with increase in the value of the parameter, the IRI value will increase. Higher IRI would result from a concrete mixture whose 28-day compressive strength is higher. This is to be noted that such a mixture usually will have a lower water-cement ratio, and will be somewhat difficult to handle. The higher the thickness of the slab, the higher the IRI value. Similar observations have been made by Perera and Kohn (*10*) in their analysis of Long Term Pavement Performance (LTPP) profile data. The strength of the stabilized base also affects the as-built curling. The models show that higher base strength results in higher curling. It can be assumed that if the base is very stiff it will be somewhat "unyielding," and the profile of the base would significantly affect curling of the concrete slab.

CONCLUSIONS

Based on this study following conclusions can be made:

1. Contribution of curling to the roughness of newly built concrete was found to be significant.

2. IRI values calculated form profiles without curling are affected by compressive strengths of concrete and BDB layers, change in the Plasticity Index values of subgrade due to soil treatment and vertical grade of the road. Higher IRI would result from stronger concrete and stiffer base. This is also true for higher slab thickness.

3. As-constructed and early life curling of PCC pavements are affected by the slab thickness, stiff base, stronger concrete, and vertical grade. Increase in values of these parameters will result in increase in curling resulting in higher values.

4. Traffic does not have any effect on curling.

ACKNOWLEDGMENTS

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Training—the Key to Technology Implementation

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ABSTRACT

During the past decade, research has made great strides in providing new materials, methods and equipment for improving maintenance of transportation facilities. Topping the list of accomplishments is the way governmental agencies are approaching snow and ice control operations. The 1988 to 1993 Strategic Highway Research Program began the process with nearly 20 million dollars being spent in a maintenance operations research program. The International Technology Scanning Tour program followed in 1994 with a winter maintenance operations scan of Japan, Germany, and Austria, followed by a 1998 scan of Switzerland, France, Norway and Sweden and finally the latest winter operations and intelligent transportation systems applications scan in 2002 revisited Japan.

The American Association of State Highway and Transportation Officials established a pooled fund study to provide the necessary financial support to develop a national computer-based, anti-icing and road weather information system training program for state and local governments. Nearly all of the snow-belt states and the American Public Works Association (APWA) and the National Association of County Engineers (NACE) contributed to this pooled fund. The computer-based training (CBT) program developed to meet this need is fundamentally a menu-driven, hyper-linked, interactive, content manager. The user once logged in, can work through this stand alone training from beginning to end, like a book, returning to the menu at intervals, as desired, to select another path. The content is photographs, illustrations, text, video, charts, animation, interaction, narration and other means of communication. There are opportunities at various points to access the progress the user is making educationally, including quizzes, scenario-based problem cases, and exercises. The training can be individually administered or used in a group setting and can be the foundation for a certification program.

Two versions of the CBT program, one generic and the other customized, will be completed by the time this paper is presented. APWA and NACE selected the generic version while nearly all the states desired the customized version specifically tailored to the methods and chemicals used in their snow and ice control operations. Feedback from the state department of transportation maintenance personnel and trainers who are preparing their customization needs is the product exceeded their expectations. The CBT was easily installed on their computers and will fit well into their training program. The CBT will work well in either the group or individual training mode.

Key words: anti-icing—computer-based training—road weather information system—snow and ice control—technology transfer

BACKGROUND

During the past decade, research has made great strides in providing new materials, methods and equipment for improving maintenance of transportation facilities. Topping the list of accomplishments is the way governmental agencies are approaching snow and ice control operations. The 1988 to 1993 Strategic Highway Research Program (SHRP) began the process with nearly 20 million dollars being spent in a maintenance operations research program. The International Technology Scanning Tour program followed in 1994 with a winter maintenance operations scan of Japan, Germany and Austria, followed by a 1998 scan of Switzerland, France, Norway, and Sweden and finally the latest winter operations and ITS applications scan in 2002 revisited Japan.

This tremendous influx of new research knowledge and technological advances brings a societal obligation for government to increase the efficiency and effectiveness of private and public winter maintenance of transportation facilities. Environment Canada's recent declaration that chloride based chemicals should now be considered CEPA Toxic adds to this sense of urgency for the snow and ice community to focus on the proper handling, storage and application of commonly used anti-icing and de-icing chemicals.

Training for supervisors and field operators in understanding the new processes and equipment used in these proactive snow and ice control techniques has been slow in developing. Lack of effective and scientifically based training has hampered progress in the implementation of anti-icing (AI) and road weather information system (RWIS) technologies from the SHRP and International Scanning Tours.

The American Association of State Highway and Transportation Officials (AASHTO), recognizing these educational needs established a pooled fund study to provide the necessary financial support to develop a national computer-based, AI/RWIS training program for state and local governments. Nearly all of the snow-belt states and the American Public Works Association (APWA) and the National Association of County Engineers (NACE) contributed to this pooled fund.

The computer-based training (CBT) program developed to meet this need is fundamentally a menu-driven, hyper-linked, interactive, content manager. The user once logged in, can work through this stand alone training from beginning to end, like a book, returning to the menu at intervals, as desired, to select another path. The content is photographs, illustrations, text, video, charts, animation, interaction, narration and other means of communication. There are opportunities at various points to access the progress the user is making educationally, including quizzes, scenario-based problem cases, and exercises. The training can be individually administered or used in a group setting and can be the foundation for a certification program.

PROJECT DEVELOPMENT

The need for the development of an interactive computer-based, stand-alone, training program was identified during the AASHTO/Federal Highway Administration SHRP Implementation Program by the Lead States Team for the implementation of advanced anti-icing (AI) and proactive snow and ice control technology. When the Lead States program was sunset, the responsibilities for developing and implementing the computer-based training program was handed off to the AASHTO Snow and Ice Cooperative Program (SICOP). The Aurora Consortium, an RWIS research consortium, had training as one of its top program priorities. The Aurora Consortium and AASHTO SICOP agreed to partner in the development of a national AI/RWIS training program with Aurora taking the lead in developing the scope of work and obtaining a contractor to build the computer-based training program. SICOP agreed to raising the necessary funding and coordinating the project.

A request was made to all state departments of transportation (DOTs), APWA, and NACE to make nominations for a team of experts in anti-icing and snow and ice control operations and instructors familiar with teaching maintenance field personnel. A technical working group (TWG) was organized from those nominations to develop the content of the training program and guide the contractor in building the training program.

The contract for the project was signed in March 2001. By September 2001, 800 pages of storyboards had been drafted for TWG review. By spring 2002, the contractor had draft copies of the first lessons on CD-ROM ready for TWG review. The state DOTs received CD-ROMs of the first three lessons in September 2002 and were asked to make recommendations for customizations to tailor the training to their individual state needs.

COURSE CONTENT

The course consists of seven lessons containing a total of 38 units. The content outline is listed below:

Lesson I: Introduction to Anti-icing and Winter Maintenance
- Unit 1: The New World of Anti-icing
- Unit 2: Benefits of Anti-icing
- Unit 3: Anti-icing in a Nutshell
- Unit 4: Units of Measure

Lesson II: Winter Road Maintenance Management
- Unit 1: Components of a Successful Anti-icing Program
- Unit 2: Preparing for the Winter Season
- Unit 3: Level of Service
- Unit 4: Data Collection and Record-keeping
- Unit 5: Anti-icing Communications and Legal Matters

Lesson III: Winter Roadway Hazards and Principles of Overcoming Them
- Unit 1: Water and its Winter States
- Unit 2: Road Surface Heat
- Unit 3: Condensation and Dew Point Temperatures
- Unit 4: Pavement Temperature—It's the Key!
- Unit 5: Snow, Ice and the Roadway
- Unit 6: Snow/Ice Bonds and Freezing-Point Depressants
- Unit 7: Dilution of Solution
- Unit 8: Chemical Concentrations and Application Rates
- Unit 9: Friction

Lesson IV: Weather Basics
- Unit 1: Weather and Winter Road Maintenance
- Unit 2: Air, Atmosphere, Heat and Humidity
- Unit 3: Weather Systems
- Unit 4: Regional Weather Influences
- Unit 5: Precipitation Hazards
- Unit 6: Non-Precipitation Hazards

Lesson V: Weather and Roadway Monitoring for Anti-icing Decisions

Unit 1: Radar
Unit 2: Weather Observation and Data Gathering
Unit 3: An Introduction to Road Weather Information Systems
Unit 4: The Importance of VAMS
Unit 5: Eight Critical Questions
Unit 6: Combining Anti-icing and the Traditional Approach

Lesson VI: Computer Access to Road Weather Information
Unit 1: An Introduction to the RWIS Screens
Unit 2: Navigating Through the System
Unit 3: Other Online Resources

Lesson VII: Anti-icing Practice in Winter Maintenance Operations
Unit 1: Preparing for the Season
Unit 2: Equipment Types, Preparation and Maintenance
Unit 3: Material Preparation and Storage
Unit 4: Chemical Application Rates
Unit 5: End-of-Season Tasks

COURSE DOCUMENTATION

- *AI/RWIS CBT Setup Guide* is a manual describing how to set up the CBT on your PC. The guide is written for the information technology staff.
- *AI/RWIS CBT User Guide* is the primary reference manual for the CBT. This manual is meant for the CBT users. The User Guide explains in detail how to use the software and provides a detailed description of each of the CBT's features and functions.
- *Training Manager Guide* is a guide for training managers. It details the Training Manager Tool.
- *Course Editor Guide* details the use of the Course Editor Tool. The Course Editor is designed for training managers.
- *Implementation Guide* is written particularly for training managers. It explains how to roll out the CBT and how to best monitor student performance both with the CBT and on the job.

USING THE CBT

The CBT structure and flow is explained below:

- A splash screen appears each time the CBT is launched. It is a composite of small images reflecting training program content. As the images appear, music plays in the background. The splash screen requires about 15 seconds to build. If the student desires to bypass this screen, pressing the space bar or enter key will advance to the log-in screen.
- A log-in screen must be completed each time so student progress can be recorded. Log-in requires first name, last name, password and job title. Thereafter the Microsoft Agent "Jake", an online assistant, will address the student by their first name. Jake is an animated conversational personality that walks the student through the tutorial (discussed below) and provides assistance when the student needs help. In addition to the role of a guide, Jake will appear on occasion to drive home a point or sometimes to just entertain.
- A welcome video will present a brief video introduction to the course. The welcome video will play the first time the student uses the CBT.

- A tutorial will familiarize the student to the features and functions of the CBT. The full tutorial requires 31 minutes. The student can go through the entire tutorial or select tutorial topics. When the student logs back into the program for a subsequent session, they can revisit the entire tutorial, select topics or skip the entire tutorial.
- A Road Map appears once the student exits the tutorial. The Road Map illustrates the student's progress and directs them to units within each lesson. Each road sign on the screen represents a lesson in the course. Lessons must be completed in order. Completed lesson signs will be checked off as soon as the student works through all of the lesson content and earns a passing score on the Post-assessment quiz and scenario.
- Lesson Introduction—each lesson begins with a video introduction to the content in that lesson. The main topics discussed in the forthcoming lesson are displayed on the screen as a real person host mentions them.
- A pre-assessment quiz is administered after the Lesson Introduction. The purpose of the Pre-assessment is to evaluate what the student knows before going through the lesson so it can be compared to what they know after going through the lesson. The quiz contains questions in a variety of common formats (multiple-choice, true/false, and fill-in-the-blank). On the last question of the Pre-assessment a "Check My Score" button will appear. Clicking on that button will display a score panel with student results.
- The Lesson Content in each lesson is organized into units. Each unit is broken down into screens. Lessons contain anywhere from three to nine units. Each unit has as few as five, or up to 40-50, screens. The lesson content is presented using multimedia elements, including the following:
  - text
  - bullets (key points)
  - photographs
  - illustrations
  - charts, graphs or tables
  - screen element highlighting
  - narration
  - animation
  - digital video
  - sound effects
  - mouse and/or keyboard-controlled interactive exercises and simulations
  - review questions

  Interactive exercises will "engage" the student and topic being discussed. Review questions will be presented about every 5 to 10 screens. These are designed to check the students understanding of the topic being discussed on the past few screens. Review questions are presented in a variety of formats, such as multiple choice, fill-in-the-blank, true/false, or drag-and-drop. Feedback will be provided so the student can see how they did and if they missed a question, what the correct answer is.
- The knowledge base is a warehouse of information related to AI/RWIS. The student should think of it as an online encyclopedia. Material in the knowledge base is arranged by tab groups discussion topics by subject or area or in an alphabetical index. In addition to text, knowledge base discussions may include photographs, diagrams, tables, web site links, digital videos, etc. Some discussions include links to other discussions. These are identified as blue underlined text. The student can click on these "hot terms" to jump to those discussions in the knowledge base.
- A glossary contains a list of AI/RWIS terms and their definitions.
- The post-assessment quiz is to evaluate what the student knows after going through the lesson. On the last question of the post-assessment a "Check My Score" button will appear. Clicking that button will bring results of the post-assessment quiz and pre-assessment scores so the student can

compare what they now know after going through the lesson compared with what they knew beforehand.

- Scenario—while the post-assessment quiz evaluated the student's knowledge of AI/RWIS facts, the scenario evaluates their understanding of the lesson content by asking them to put the knowledge they have gained into practice. It is well known that working with theories is one thing; working within the constraints of the real world can be quite different. The scenario room gives the student hands-on practice in a simulated winter maintenance facility so that they can develop and refine their winter maintenance decision-making skills. The scenario room is set up to look like a field maintenance garage office. It provides the student with the tools most maintenance facilities have in some form or other to learn of an impending winter weather event. They should be able to research the particular nature of the event and make operational decisions based on that research. Everything the student does in the Scenario Room is tracked and evaluated. The student is encouraged to strive to use all of the pertinent tools available, yet do not waste time clicking on objects that will not aid for the particular event. Detailed feedback will be provided once the student has made an operational decision. If the student does not pick the optimal solution to the problem, they will learn what the optimal solution is. The results of their decision will be compared with the results of the optimal solution. This way the student will learn the consequences of making a less-than-optimal operational decision. The feedback will also list each step taken, the order they took each step, and the time needed to complete the step. There are two scenario modes: Practice and Evaluation. Practice mode lets the student work through the scenario without being graded. A student can take up to three practice scenarios before tackling the Evaluation, or graded scenario.
- EPSS Mode—The AI/RWIS CBT continues to be a valuable tool even after the student completes the course. When the student finishes the CBT, a new feature is activated. This feature is known as the Electronic Performance Support System (EPSS) or EPSS Mode. The student can know access this feature through the Road Map icon on the Road Map screen. The EPSS Mode screen is divided into two main panels. The panel on the left includes a scrolling alphabetical list of discussion topics in the CBT. The student locates the topic they wish to review, highlight the topic by clicking on it and then clicks on the "Go to Selected Topic" button to jump to the first screen of that discussion. Above the alphabetical list of topics there is a search field. Rather than scroll through the extensive list, the student can type the first few characters of the topic of interest and the list will automatically scroll to the first topic matching the characters the student typed in. On the right hand side of the screen, topics are organized into a content tree. If the student needs help, click the Help button. Jake will appear and provide the assistance needed.

END PRODUCT

Two versions of the CBT program, one generic and the other customized, will be completed by the time this paper is presented. APWA and NACE selected the generic version while nearly all the states desired the customized version specifically tailored to the methods and chemicals used in their snow and ice control operations.

Feedback from the State DOT maintenance personnel and trainers who are preparing their customization needs is the product exceeded their expectations. The CBT was easily installed on their computers and will fit well into their training program. The CBT will work well in either the group or individual training mode.

A metric version of the CBT is being prepared for use in the Canadian provinces.

Analysis of the Environmental Justice Compliance of the Chicago Transit Authority (CTA)

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ABSTRACT

Environmental Justice (EJ) derives its principles and foundation from Title VI of the Civil Rights Act of 1964. The Executive Order 12898, issued in 1994 mandated all agencies receiving federal funds to be environmental justice compliant. According to this Order, Environmental Justice compliance ensures that the minorities, ethnic population groups as well as the low-income population in a region are not adversely affected due to the implementation of programs funded with federal dollars.

This paper presents a framework to evaluate the EJ-compliance of the Chicago Transit Authority (CTA), specifically focusing on the CTA's capital program. The CTA has divided its service area into six zones in order to facilitate allocation of capital funds.

It is the objective of this paper to analyze the CTA service area with the help of the decennial census. The analysis will identify the neighborhoods that have a significant percent of the target minority or low-income population, referred to as "EJ neighborhoods". The "EJ neighborhoods" will be compared versus the "non-EJ neighborhoods" to test the equitable allocation of funds.

The criteria for identifying the EJ neighborhoods depend upon the definition of the target population and while there is considerable variation amongst researchers about these definitions, this paper will acknowledge these differences and will make use of the most commonly accepted definitions in identifying these target population groups. The results from this research will help the CTA in ensuring that the needs of the minorities and the low-income are acknowledged and accounted for in the planning process.

INTRODUCTION

The Chicago Transit Authority (CTA) is the transit provider for the City of Chicago and operates both train and bus service in a mature travel market. In 2001, the 134 bus routes provided 303.1 million rides while the rail network provided 151.7 million rides. Ridership has increased for the CTA in each of the last five years as overall service has been improved and expanded. (*1*) The CTA is constantly refurbishing and renovating its capital assets and there is a need to ensure that the allocation of monies is done equitably and in compliance with the environmental justice mandate of the federal government.

The objective of this research is to develop a framework to evaluate the environmental justice compliance of the CTA. The stated objective entails a series of intermediate steps that need to be completed before one can draw conclusions about the compliance issue. One of the very first steps in this process is to perform a demographic analysis of the CTA service area and compare it to the capital improvement program distribution of the CTA.

This paper traces the history of the legislation of environmental justice and includes a review of environmental justice case studies based on a literature review. The paper sets the stage for the environmental justice analysis with a comprehensive demographic analysis based on data from the 2000 decennial census. The conclusions drawn from the analysis point to future work that needs to be addressed. The following sections present a background of the study area and the environmental justice movement.

Background (2)

The City of Chicago is home to a diverse population with a significant percentage of minority, ethnic, and low-income populations. The minority population in the six-county region that includes Chicago continues to increase at an unprecedented rate. Population in the minority categories (everything except non-Hispanic white only) now represents about 43% of the region's total population compared to less than 35% in 1990. Northeastern Illinois's Hispanic population has grown substantially, from 836,905 in 1990 to more than 1,405,116 in 2000. This represents a 68% increase over the ten years.

While the actual number of people in poverty increased for the region, the poverty rate for the region decreased from 11.3% to 10.6% in the 1990s. However, the City of Chicago realized a decrease of approximately 35,000 residents living in poverty despite a total population growth of 112,290 people during the decade of the 1990s. Suburban Cook County and the five other counties in the region saw an increase of nearly 68,000 residents living in poverty.

Despite increases in ridership for the CTA over the last five years, the percentage of work trips by public transportation declined from 15% to 13% during the 1990s. This can be attributed to the large increase of population and employment in the suburban counties. Public transportation remains a vital part of the transportation network in the region, especially in the densely populated areas of Chicago. It is therefore imperative for the transit providers of the region to ensure that the service caters to the needy and is equitable.

ENVIRONMENTAL JUSTICE

Equity in transportation has been a major concern of transportation planners and lawmakers dating back to the civil rights movement. Environmental justice has progressively gained momentum in the last decade and along with it have attempts to ensure that no target population is disproportionately affected by projects involving federal dollars.

The Federal Transit Administration has issued three principles of environmental justice to guide transit agencies in their compliance efforts:

1. Ensure that new investments and changes in transit facilities, services, maintenance, and vehicle replacement deliver equitable levels of service and benefits to minority and low-income populations

2. Avoid, minimize, or mitigate disproportionately high and adverse effects on minority and low-income populations

3. Enhance public involvement activities to identify and address the needs of minority and low-income populations in making transportation decisions.

Legislative History

Title VI of the 1964 Civil Rights Act required all federal agencies receiving federal financial assistance for a particular program or activity to ensure that no person is excluded from participation in, denied benefit of, or subjected to discrimination on the basis of race, color, national origin, age, sex, disability, or religion. The Civil Rights Restoration Act of 1987 broadened the coverage of Title VI to include all programs and activities of federal-aid recipients, sub-recipients, and contractors whether those programs were federally funded or not. Today, Title VI still provides the legal backing for the majority environmental justice complaints.

President Bill Clinton issued Executive Order 12898: Federal Actions to Address Environmental Justice in Minority Populations and Low-Income Populations. This order mandated that every federal agency administer and implement its programs, policies, and activities in a way that will identify and avoid disproportionately high and adverse effects on minority and low income populations. In addition, the order states that each federal agency should use their powers in the greatest extent allowed by law to achieve this goal.

These legislative issues and documents of guidance establish the basis for environmental justice policies. They clearly emphasize the level of importance that the federal government holds to environmental justice issues.

Importance of Compliance

Despite the increased attention on the subject of environmental justice, politicians, planners, and researchers have yet to establish a common methodology for analyzing an agency's compliance. In fact, numerous agencies have taken drastically different approaches in ensuring compliance. The importance of constructing a methodology that will properly assess the unique aspects of an agency is crucial. Projects and policies that agencies implement today are more highly scrutinized than ever before. Agencies that ignore compliance issues are likely to be served with lawsuits. This was the case in the 1990s for the Los Angeles Metropolitan Transportation Authority (MTA). The MTA neglected poor and minority citizens in their funding allocation and as a result spent millions of dollars in lengthy court battles, which they ultimately lost. Their decision not to comply with legislation will have ramifications for the MTA and Los Angeles County for many years to come. (*4*) Compliance should not only be looked at as a requirement, but also as a means for achieving a better understanding of the public that it serves. Understanding the populations in the service area will lead to more knowledgeable decisions in the future. For this reason, developing an effective methodology for measuring compliance can potentially give agencies a useful tool that could aid in the decisions to distribute funds and prioritize projects in future years.

OVERVIEW OF THE METHODOLOGY

This section reviews issues related to the boundaries of the CTA service areas, the selection of the geographical unit of analysis, and the selection of areas that are to be considered environmental justice neighborhoods.

Identification of the Study/Service Area

For the purposes of this project, the CTA divided its service area into six geographic zones (Loop, North, Northwest, South, Southwest, and West). The CTA chose to use broad descriptions of the zonal boundaries (e.g. "service limits to the west") and as a result the demographic figures compiled are approximations. However, we are confident that the approximations are representative of the zone's true demographic make-up. The inclusion or exclusion of certain populations in this analysis due to broad zonal boundaries is a concern and will be addressed in future months.

The demographic analysis for this research is conducted at two levels: (1) at the zonal level, and (2) at the census tract level. Therefore, our analysis includes all census tracts that have their geographic center located within the six zones. This method has given us realistic approximations of the demographic and economic conditions in each service area. However, these numbers are only approximations and future work on defining the appropriate boundaries will continue in the second half of this study based on discussions with staff at the CTA.

The data sources for this research are the decennial census and the CTA. The release of the Census 2000 data makes this analysis current and timely. Table 1 depicts the zonal populations as well as the number of census tracts within each zone. It should be noted that while Black and White are racial categories according to the census, Hispanic is not and is classified as an ethnic category. To qualify as low-income one's income must be within 150% of the poverty threshold. The selection of the criteria is discussed in the subsequent section.

TABLE 1. Target Population Demographics

Zones	Number of Census Tracts	Zonal Population	Black Population	Hispanic Population	White Population	Low-Income Population
Loop	6	16,244	3,213	936	10,613	2,204
North	123	479,587	68,970	49,632	338,099	95,039
N-W	274	1,198,257	50,946	357,619	817,015	227,275
South	266	744,923	568,184	64,556	88,756	276,154
S-W	160	689,267	142,902	190,988	417,475	141,644
West	268	942,571	329,562	291,458	408,932	261,637

Table 1 clearly shows the population differences between zones. It also reveals that there appears to be high concentrations of target populations living in the same zones (e.g. Blacks in the south zone)

Environmental Justice Neighborhood Identification

Various researchers have debated the process of identifying an environmental justice (EJ) neighborhood without consensus on the most appropriate method. In this section, various issues pertaining to the development of a compliance methodology are discussed.

Determining the unit of analysis

Agencies must address two issues when determining the specifics of their study area. First, agencies must consider the desired size of the unit of analysis. Size could refer to geographic measures as well as population measures. Secondly, agencies must ensure that data are available at the level of aggregation (disaggregation) of the geographic unit desired. Compromises in selecting a unit of analysis will often be made on account of data availability.

Another tradeoff that must be considered is with regards to the accuracy of the data and the ease of comparison between units of analysis. Using census blocks as a study area provides great accuracy and detail, but may be too overbearing in terms of identifying and comparing millions of blocks, as may be the case in a large metropolitan study area. On the other hand, dividing a large study area into a few zones provides for easy and quick comparisons between zones, however the accuracy and precision of the data will be compromised due to errors in aggregation. Past environmental justice studies have revealed no consistent selection for the level of aggregation to use. However, numerous studies have used census tracts.

Census tracts are apportioned based on population. The population of a tract typically falls between 1,000 and 8,000 residents. This level of disaggregation provides for acceptable accuracy when working with a large study area, such as a metropolitan area. Data accuracy can be improved and errors minimized by using smaller geographical units such as traffic analysis zones or quarter sections. However, the availability of data at the desired level of aggregation is always a cause for concern and hence the biggest reason why planners rely on larger zones for analysis.

Research has revealed problems with using large zones and furthermore recommends using census blocks or block groups. Discrepancies within a zone might cause inaccurate results to be inferred or stated. In a study conducted by David Forkenbrock and Lisa Schweitzer at the University of Iowa Public Policy Center, this issue was discussed in detail. (*3*) The stated goal in the paper was to make the unit of analysis "small enough to be relatively homogeneous in terms of population characteristics." However, often times obtaining a homogeneous population requires the use of census blocks, which does not have the same information available to the public for privacy reasons (e.g. income).

Some agencies such as the Southern California Association of Governments (SCAG) have used a different approach for analysis – one based on dividing the study area by demographic distribution rather than spatial units. In their 2001 Regional Transportation Plan SCAG divided the study area into five zones to determine the impacts that the plan would have on low-income individuals. (5) While this approach satisfied the agency's needs, the transferability and universality of this approach, as well as the accuracy of the analysis, remain inconclusive.

Demographic/Economic criteria for EJ neighborhood identification

Defining an EJ neighborhood also allows for broad interpretation. While there is no prescribed standard for this procedure, the criteria should have some degree of backing from past research and/or government legislation and policies. Multiple criteria could possibly be analyzed in an environmental justice study, including, but not limited to, race, ethnicity, income-level, disability, and age. For the purposes of this paper, criteria were selected on the basis of minority status, ethnic background, and income levels.

Race and Ethnicity: Race (Black) and ethnicity (Hispanic) are two criteria that are always analyzed in environmental justice studies. There are typically two ways a unit of analysis can qualify as an EJ neighborhood on the basis of race and/or ethnicity. A common way to decide this is through the use of a reference area. For example, if an environmental justice study were being conducted for a city, then the percentage of Blacks and Hispanics, as a total of the entire city population would be calculated. Consequently, any unit of analysis within the city that meets or exceeds this threshold would qualify as an EJ-neighborhood. For example, if a city's population included 15% Hispanic people, then any unit of analysis with a Hispanic population of 15% or higher would be considered an EJ-neighborhood. The reference area technique is becoming increasingly popular with municipalities and metropolitan planning organizations across the nation.

The other method that could be used to determine an EJ neighborhood on the basis of minority or ethnicity is by setting an arbitrary threshold. For example, the Agricultural Advisory Board in the Environmental Protection Agency defines a minority community as a census tract that has a minority group that accounts for greater than 30% of the total population in that census tract. This method is not as common, partly because the percentage chosen is subject to criticism as it typically has little backing by past studies or federal actions. The use of a reference area seems to be more logical, and furthermore it allows for methodologies to evolve smoothly over time and adapt to specific local trends.

Income: The last criteria that must be decided concerns low-income populations. This decision appears to be much more complex then the decision on racial and ethnic populations. In the past, studies have used the reference area technique. In order to use this technique, the percentage of people living at or below the poverty rate (or some other income measure) is calculated for the reference area and any unit of analysis that meets or exceeds the threshold is considered an EJ-neighborhood. The reference area technique is sometimes conducted through use of the median household income. The threshold would be determined by the region's median household income. If a unit of analysis has a median household income of a certain percent of the threshold, then it would be considered an EJ neighborhood. The percent used typically ranges from below 30% up to 80% of the area median household income. The Department of Housing and Urban Development has set four percentages to define lower income, low income, very low income, and extremely low income. In similar fashion, 150% of the poverty line is accepted as a low-income threshold in the poverty/environmental justice arena.

Another technique would be to use a set poverty percentage as the threshold. Numerous poverty research studies have used 20% of poverty as a threshold.

Other techniques such as the one used in the SCAG study where they used quintiles to determine the impacts on low-income citizens are accepted. They used the quintiles to study the impacts that the regional plan had on the poorest fifth of their population, second poorest fifth, etc.

As evidenced in this section, defining an EJ neighborhood on the basis of income is very complex. Numerous variations of the reference area technique could be used, which keeps the local perspective in the project. However, federal guidelines such as a 20% threshold could also be used with little or no adversity. This could be enticing for agencies as they are attempting to prove compliance with federal policies.

In addition to the above discussed methodology concerns, agencies should consider whether the use of varying degrees of EJ neighborhoods is appropriate. For example, should a census tract with a high level of poverty be analyzed in the same manner as a census tract with a high degree of minorities, low-income individuals, and people with disabilities?

The selection of the criteria to be used is very critical to establishing a good methodology. The way that an agency chooses to display the data is critical to what the results of the study will actually reveal.

COMPARISION OF EJ NEIGHBORHOODS AND NON-EJ NEIGHBORHOODS

The importance of adopting an acceptable definition of an EJ neighborhood was underscored in the previous section. Next, we look at the definitions used for this study.

EJ Neighborhood Definitions

The reference area methodology is used in this research for identifying geographic units of analysis (i.e. census tracts) that qualify as environmental justice neighborhoods. The reference area for this analysis is Cook County. Cook County contains both the CTA statutory service boundaries and the City of Chicago. The EJ neighborhood criteria are obtained by computing the appropriate demographic and economic information for Cook County and using that information as the threshold for the individual census tracts. The thresholds for each of the target populations are listed below.

- <u>Black</u> **–** According to the 2000 Census, 26% of all residents of Cook County are Black. All census tracts with a Black population greater than 26% qualify as EJ neighborhoods.

- <u>Hispanic</u> **-** According to the 2000 Census, 20% of all residents of Cook County are Hispanic. All census tracts with a Hispanic population greater than 20% qualify as EJ neighborhoods.

- <u>Low-income</u> - All households that are within 150% of the poverty threshold are considered low-income. Using the reference area technique, this population is 21.5% of the total population in Cook County. Accordingly, census tracts which have more than 21.5% of their population within the low-income threshold, qualify as EJ neighborhoods.

A breakdown of EJ census tracts and non-EJ census tracts by zone can be seen below in Figure 1.

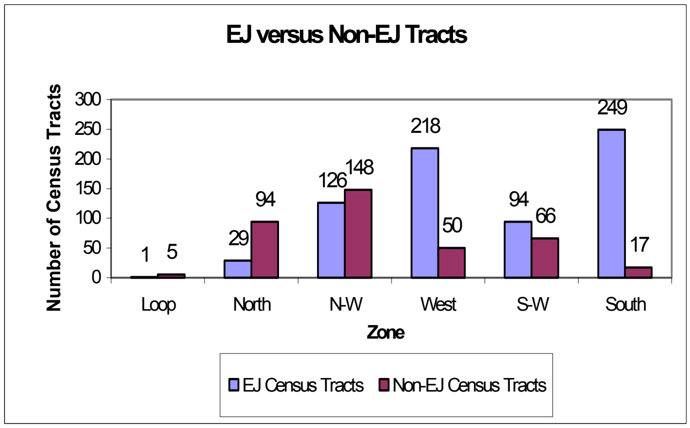

FIGURE 1. EJ and Non-EJ Tracts by Zone

Comparison of EJ Neighborhoods and Non-EJ Neighborhoods

The objective of this comparison is to provide a detailed look at selected demographic differences between the respective populations of people living inside census tracts that are considered EJ neighborhoods and those that are living in census tracts designated non-EJ neighborhoods within each of the six CTA service zones and the service area as a whole.

A census tract may qualify as an EJ census tract on one, two, or all three of the criteria (Black, Hispanic, Low-Income). Consequently an EJ census tract may not contain a comparatively high concentration of one or two the designation criteria. For example, a census tract may have a high concentration of low-income individuals and no Black or Hispanic individuals. This census tract would still be considered an EJ census tract. Comparisons of populations in this report were done between the populations of those living in EJ census tracts (regardless of the criteria of which it qualified) and those populations living in non-EJ census tracts.

We are looking for high concentrations of Blacks, Hispanics, and/or low-income residents and not at individuals themselves. Consequently, nearly all non-EJ census tracts are going to have some individuals who are Black, Hispanic, and/or low-income. However, these demographic populations are not concentrated to the extent that it exceeds the percentage of concentration in our reference area, Cook County. While considering people on an individual basis would be useful, it is unrealistic for a holistic analysis of this magnitude.

A final issue to consider when looking at this data is that one individual can be Black, Hispanic, and low-income. These populations do not have clearly defined borders rather they have overlapping boundaries. Therefore, as more data becomes available through secondary sources we will hopefully be able to systematically reduce or eliminate problems of double counting.

Keeping these crucial underlying concepts in mind, we believe that we have a very useful and realistic snapshot of the populations in EJ census tracts and non-EJ census tracts. Through the use of Census 2000 and geographic information systems we were able to accurately capture the demographics of individuals in EJ census tracts and non-EJ census tracts within each of the six CTA service zones. Comparing these populations within each of the six service zones and the service area as a whole has exposed some interesting statistics.

Results of the Comparison

The study area consists of 1,107 census tracts according to the 2000 decennial census. (Table 1) These tracts qualify as EJ tracts under either the race, ethnic, or low-income categorization. The most striking detail about the distribution is that the study area has more census tracts under the EJ classification (762) as opposed to the non-EJ classification (345), or 69% compared to 31%. An overwhelming majority of these EJ tracts are concentrated in the West and South zones (43% of all census tracts) with the Northwest zone's EJ tracts accounting for another 11% of the total census tracts.

In addition to a strikingly high number of EJ tracts, the data revealed extremely high concentrations of the target populations within the EJ neighborhoods. As previously stated, EJ tracts only have to exceed the Cook County threshold in one or more of the target populations. This means that there is no distinction between EJ tracts that meet the threshold and those that exceed the threshold by two or three times. However, the concentrations of the target populations were evident after running comparisons between EJ neighborhoods and non-EJ neighborhoods. The following statistics clearly show the high concentrations of the target populations in EJ neighborhoods.

- System-wide, 86% of the low-income population is found to live in the EJ tracts, with only 14% living in the non-EJ tracts.
- System-wide, 96% of the Black population is found to live in the EJ tracts, with only 4% living in the non-EJ tracts.
- System-wide, 87% of the Hispanic population is found to live in the EJ tracts, with only 13% living in the non-EJ tracts.
- System-wide, only 11% of the White population is found to live in the EJ tracts, with 89% living in the non-EJ tracts.

Looking at the zones with the greatest population of the target populations the data revealed even higher amounts of concentration.

- The South zone has the largest Black population in the in the study area (586,184) and the EJ tracts in this zone account for nearly all (99.9%) of the Black population in the zone.
- The Northwest zone has the largest Hispanic population in the study area (357,619) and the EJ tracts in this zone account for 83% of the Hispanic population in the zone.
- The South zone has the largest low-income population in the study area (276,154) and EJ tracts in this zone account for slightly over 99% of the low-income population in the zone.

Of the 762 EJ census tracts in the study area, 32 qualify as EJ tracts on all three criteria (i.e. they meet the threshold for race, ethnicity, and low-income). These are critical neighborhoods that need the utmost attention. These 32 census tracts are distributed amongst the six zones with the North, and Northwest having two and four tracts each respectively. The West zone includes nine tracts and the South zone includes ten tracts.

Table 2 depicts the work trips made in the study area by public transportation. This table reveals some interesting details.

TABLE 2. Work Trips by Public Transportation

	Tracts	Low-income population	Total Work Trips	Total Work Trips by Public Transportation	Percent Total by Public Transportation
Loop	6	2,204	10,675	2,724	25.5%
North	123	95,039	283,071	105,045	37.1%
N-W	274	227,275	550,855	92,430	16.8%
South	266	276,154	251,597	69,834	27.8%
S-W	160	141,644	278,256	37,222	13.4%
West	268	261,637	360,322	56,754	15.8%

Source: Census 2000.

The North zone has the highest public transportation trips in the region, both in volume (105,045) and as a percent of the zonal work trips (37%). The Northwest zone produces the largest number of work trips in the region, around 550,000, but only 17% (92,430) of these trips are made by public transportation.

Table 2 shows that each zone is dependent on public transportation, but to varying degrees. Failing to invest equitably in any zone would undoubtedly negatively affect thousands of residents. The zones with the highest low-income population (South, North-West, and West) theoretically need special attention, as access to jobs is probably a large barrier for residents.

CONCLUSIONS

The literature on environmental justice is not expansive and is evolving with time. In the absence of a consensus on proper methodologies or what constitutes an EJ neighborhood, this research has presented a methodology to identify potential EJ neighborhoods for the CTA service area.

The demographic analysis summarized in this paper is only the first step in developing a methodology of compliance for the CTA. However, the sensitivity of data to the chosen methodology (i.e. selection of the units of analysis and criteria for EJ neighborhoods) is a major concern in any study that must be addressed upfront. If inappropriate methods/ criteria are chosen results could become increasingly unrealistic and misleading.

From the demographic analysis we realized that a high percentage of target populations seem to be heavily concentrated amongst themselves. As the minority population is projected to continue on an upward trend it can be expected that the number of EJ neighborhoods will continue to rise.

It is crucial that the CTA becomes familiar with the demographic and economic profiles of its customers. Understanding the communities and how they are evolving will allow for more informed and efficient decisions. Understanding the demographic and economic profiles is only the first step in this research. A fair amount of work lies ahead in order to develop a completed methodology and analyze the compliance of the CTA.

FUTURE WORK

The research team will next have to tie the capital investments to the demographic information to analyze if there are inequities in the process. The team will make use of ridership information from the CTA as well as identify/develop other performance measures for the transit system to perform an equity analysis.

There is also a need to develop accessibility measures (number of transit users able to access jobs) and mobility measures (number of jobs accessible within certain time) based on data from the census as well as other transportation models developed in-house at the Urban Transportation Center. Finally, in order to gain an increased insight on the state of the various EJ neighborhoods, more economic and social data will be compiled. The increased data will reveal particular issues that plague many of the distressed neighborhoods. Understanding these issues could help the CTA in their project prioritization process.

The geographic realignment of the EJ tracts, based on a comparison of the 1990 census data with the 2000 census data, will also shed light on the trend in the target population. These tasks along with the development of a framework for future environmental justice analyses will equip the CTA with a tool to ensure equitable service investments in the Chicago area.

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Information is Our Seed Corn: An Overview of the Midwest Transportation Knowledge Network

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ABSTRACT

Professional engineers, planners, and administrators face incredible challenges in trying to meet the country's current and future need for safe and reliable transportation.

Fortunately research and innovation are hallmarks of transportation in the United State, thanks to the leadership of the Transportation Research Board, the U.S. Department of Transportation, universities, and state departments of transportation. Through their efforts, a continuous stream of time-and-money-saving technologies is flowing to transportation practitioners.

However, managing innovation and new information is a challenge in itself. How does the transportation community connect users to the new knowledge?

The mandate of the National Transportation Library is to increase access to and dissemination of information needed by federal, state, and local transportation decision-makers. A cost-effective approach to meeting this mandate is to help link the existing transportation libraries into a coordinated network that shares resources, services, collections, and expertise.

Toward this end, the Midwest Transportation Knowledge Network (MTKN) was formed. The MTKN is made up of cooperating libraries within AASHTO Region III that specialize in transportation-related information. The mission of the MTKN is to increase collaboration among the region's transportation libraries and information centers so managers, engineers and planners are better able to find and apply the most recent, credible, validated technical information to their current projects.

Goals of the MTKN include improving access to transportation information for every professional in the Midwest; educating transportation leaders on the importance of libraries to their missions; improving transportation systems nationwide by integrating libraries into the research process; and helping to preserve research funding.

Key words: knowledge network—information dissemination—transportation libraries

Note: Preparation of this paper was still in progress at the time of publication; final results will be presented at the symposium.

Using an Artificial Neural Network to Predict Parameters for Frost Deposition on Iowa Bridgeways

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ABSTRACT

Forecasting frost formation on bridge-ways in Iowa is an important yet difficult problem. Frost forms when water vapor in the air sublimates onto a surface (which occurs when the dew point temperature of the air is greater than the surface temperature), and the surface temperature is below freezing. Only small amounts of moisture are needed to cover surfaces with frost and create hazardous travel conditions.

Recently, a frost model was devised by Knollhoff et al. (2001) to predict frost deposition based on moisture flux principles. The inputs required by the frost model include the following: (1) air temperature, (2) dew-point temperature, (3) wind speed, and (4) surface temperature.

An artificial neural network predicts these four inputs at 20-minute intervals for a 24-hour period. The output from the neural network models can then be used as input into the frost deposition model to predict frost formation on bridgeways in Iowa. The proper development of an artificial neural network requires the dataset to be subdivided into at least a training set and a validation set. A test set can also be used to test the model(s) even further.

Key words: artificial neural network—bridgeways—frost deposition

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INTRODUCTION

Forecasting frost formation on bridge-ways in Iowa is an important yet difficult problem. Frost forms when water vapor in the air sublimates onto a surface (which occurs when the dew point temperature of the air is greater than the surface temperature), and the surface temperature is below freezing. Only small amounts of moisture are needed to cover surfaces with frost and create hazardous travel conditions.

Background

There are two primary ways in which frost forms. The first is due to radiational cooling. As turbulent mixing decreases as the sun sets, the atmosphere near the surface begins to cool, eventually becoming stratified, as a nocturnal inversion forms (*1*). Within these inversion layers, the air near ground level is cooled by the surface and becomes cooler than the air elevated above the surface around 15–20 meters (*1*). As the surface and ambient air cools, the relative humidity increases in the air if the amount of moisture in the air remains constant. A moisture flux is then directed towards the surface when the dew-point temperature of the ambient air near the surface is higher than the surface temperature. If the surface temperature is below freezing, then frost will form due to deposition as opposed to dew from condensation. When dew forms and thereafter the temperature falls below freezing, the dew will freeze and potentially impact travel. These conditions are not accounted for in this study because they are different from the conditions of frost formation.

The second type of front formation has been referred to as the advection method and it often times occurs near active fronts/boundaries. If cold air cools the surface below freezing and a moist air mass moves into the area, moisture will begin to be deposited on surfaces which have temperatures below freezing. This type of event is not as common as frost that occurs because of radiational cooling, but it still occurs a few times each season.

Several different frost predictive systems have been developed in the past, many of which relate to linear regression techniques. In this study, we will use a non-linear regression approach to predicting frost through the use of an artificial neural network. The artificial neural network uses a non-linear modeling technique based on the hyperbolic tangent function, which allows for both positive and negative values as input. Artificial neural networks have been successfully used in the past to predict various aspects of meteorology including hail size (*2*), thunderstorms (*3*), tornadoes (*4*) and quantitative precipitation forecasts (*5, 6*) to name a few.

Recently, a frost model was devised by Knollhoff et al. (*7*) to predict frost deposition based on moisture flux principles. The inputs required by the frost model include the following:

- air temperature
- dew-point temperature
- wind speed
- surface temperature

Artificial Neural Networks

The artificial neural network predicts these four inputs at 20-minute intervals for a 24-hour period. The output from the neural network models can then be used as input into the frost deposition model to predict frost formation on bridge-ways in Iowa. The proper development of an artificial neural network requires the dataset to be subdivided into at least a training set and a validation set. A test set can also be used to test the model(s) even further.

METHODOLOGY

Since each model is predicting a different variable at a different time step, each model needed to be created independently (*8*). The models were set up in such a way that parameters are predicted at 20-minute intervals throughout the day. This means that each parameter is predicted using 72 different models predicting at different time steps. Since four parameters are predicted, a total of 288 models are used over the 24-hour period. The models were developed with application at the Iowa Department of Transportation (Iowa DOT) in mind, so that predictions are made after 1800 UTC observations are collected for the model run. Because the emphasis is on frost prediction, which generally occurs during the late night or early morning hours, the first forecast time for the suite of artificial neural network models is at 0000 UTC the following day (6 PM the same evening local standard time). The input data into the ANN includes the 1200 UTC nested grid model (NGM) model output statistics (MOS) (*9*) output paired with road weather information system (RWIS) observations from the cold seasons of 1995- 1998. For non-numerical input, categorical variables were used since all input into the ANN must be numerical. A cold season for purposes of this study is defined as October 1 through April 30. This time period was chosen because frost observations made by Iowa DOT personnel were readily available for these months at four stations across the state of Iowa. The four stations used in the development of the model were Waterloo (ALO), Des Moines southwest (DSM), Mason City (MCW), and Spencer. The model, once developed, was then later tested on data for Ames over the cold seasons of 2001-2002 and 2002- 2003.

Four different types of models were created to predict the input variables required by Knollhoff et al.'s model. Due to limitations on the number of variables that can be input into the neural network prior to ranking the variables, only one observed parameter other than the parameter itself was included as input. The two different types of RWIS observations used for the various models are shown in Table 1.

TABLE 1. RWIS Observations Used for Various Models

Forecasted Parameters	Observed Parameters (RWIS) Used as Input
Air temperature	Air temperature, dew point temperature
Dew point temperature	Dew point temperature, air temperature
Surface temperature	Surface temperature, air temperature
Wind speed	Wind speed, wind direction

Numerous models were created by varying the training time, architecture and number of inputs. It was noted that as more variables were added as input to the model, the performance eventually leveled off and often decreased. As can be seen from Figure 1, model performance increased very little with the addition of more than 8–10 input variables. To keep the complexity and the number of free parameters as low as possible, the number of inputs into each model was restricted to no more than 10. However, various architectures and training times were tried in various models with fewer inputs than this.

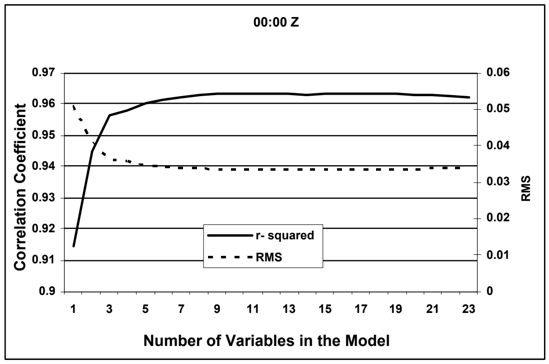

FIGURE 1. Model Performance and Input Variables

In total, 181 initial inputs were fed into the ANN for each model: 111 RWIS inputs, 65 1200 UTC NGM MOS inputs, and four unordered variables to test if the location of the RWIS sites was important to the model. Unordered variables had to be used to test if the location was important because the locations of the sites used are not related. From the 181 inputs, a general regression neural network was then used to rank the variables based on their relation to the output. Various time intervals were tried for training the model, and it was determined that a training time of 2 minutes gave the network ample time to learn the characteristics of the data. It was also found that using around eight ranked variables from the initial 181 as input into the ANN model itself produced the best results in terms of performance.

In total, there were anywhere from 1200–1500 cases included in the training and validation sets. The validation set comprised the last 15 percent of the original data set. This allowed for around 200 cases in each validation set, which adequately allowed us to test the performance of the models created.

RESULTS

Figures 2–5 show the results of a 6-hour forecast for each of the forecast variables. The first 85 percent of the data was used for training the model, and the last 15 percent was used as the validation set. The residual (error) is also plotted in yellow.

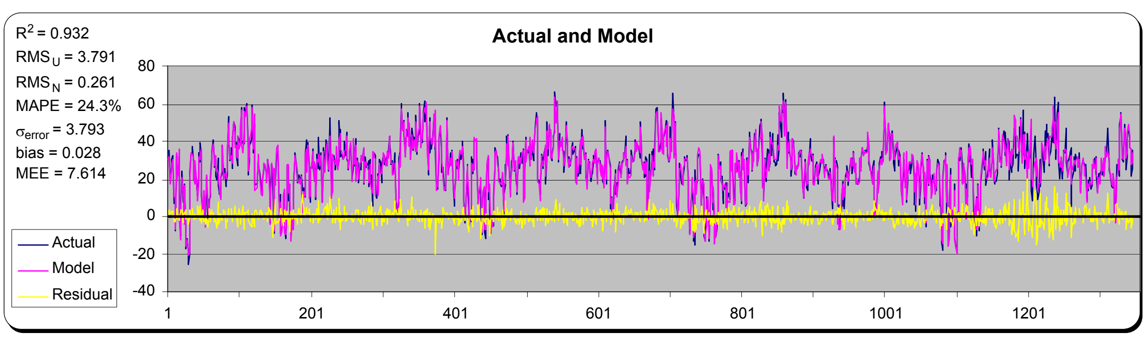

FIGURE 2. Dew Point

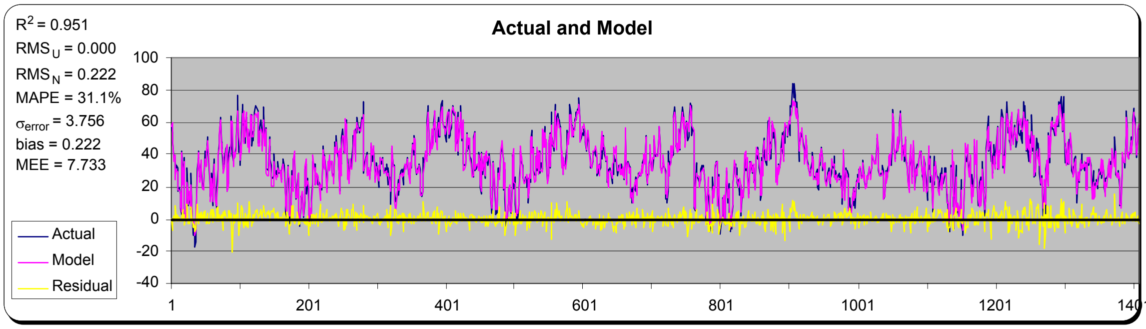

FIGURE 3. Air Temperature

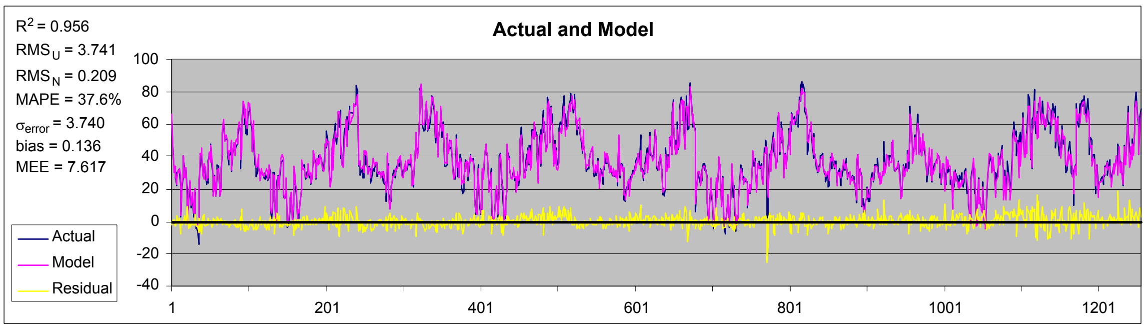

FIGURE 4. Surface Temperature

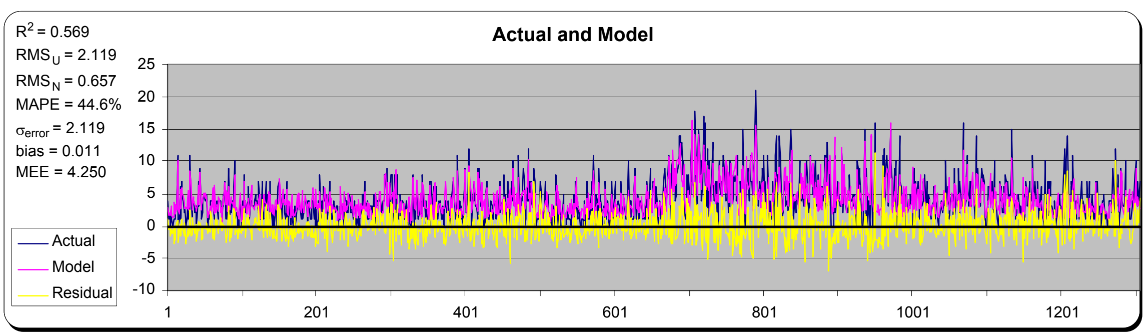

FIGURE 5. Wind Speed

Over all, the predictions correlate well with the RWIS observations, and generally perform better than the NGM-MOS output alone (for the similar forecasted variables).

Regarding a comparison of the predicted values with the MOS output alone, temperature errors using the artificial neural network were around 2 degrees F with MOS errors slightly higher, approaching three degrees. The greatest differences between MOS and neural network output were noted in the prediction of road surface temperature (because road surface temperature is not a direct MOS output, the forecasted air temperature was used as a proxy for road surface temperature). Errors with MOS again averaged close to three degrees F over the nighttime hours, but were much worse during the daytime when solar heating occurred, and road temperatures usually became much higher than 2m air temperatures. Errors with the artificial neural network were smaller, averaging just over 2 degrees. Similarly, the neural network error for the prediction of dew-point temperature overnight was close to one degree F, while it was over two degrees for MOS. Finally, the error in the prediction of wind speed was around 1–2 knots using the artificial neural network, while the errors associated with MOS averaged close to three knots across the entire time period.

CONCLUSIONS

Although not discussed in this present paper, it should be noted that although the nonlinear models usually performed best, for some parameters at some times, the linear models were better. These along with many other results will be presented at the conference.

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Spatial Data Integration for Low-Income Worker Accessibility Assessment: A Case Study of the Chicago Metropolitan Area

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ABSTRACT

The transformation of numerous and often disparate data sources into knowledge that supports critical decisions in a timely manner is essential in today's fast-growing market and digital government. Data integration is being increasingly recognized in the transportation sector as a valuable asset-forming activity aiding decision-making.

This paper will describe a spatial data integration application for a specific policy area: low-income worker accessibility in urban areas. This policy context is that entry-level job growth, which is appropriate for the skill level of welfare clients and low-income individuals, is mostly occurring far away from inner city neighborhoods or isolated pockets in the suburbs, where bulk of low-income and economically deprived families live. Many job-rich areas do not have affordable housing, including affordable rental housing, or there are implicit barriers to fair housing. Transit connections between job rich areas and where the bulk of the low-income and welfare recipients live (inner city neighborhood and isolated pockets in the suburbs) are limited and in some cases, non-existent. Vehicle ownership rates among such individuals are also very low. Most individuals in this category also do not have driver's licenses. Many entry-level jobs start at off-peak times. Job locations that are accessible during normal business hours are inaccessible during off-peak hours, since most transit services served peak hour transit markets. There are three strategies of overcoming the spatial mismatch between jobs and home locations: (1) bring jobs closer to low-income neighborhoods (2) bring affordable housing solutions near employment generating areas and (3) provide transportation connections between jobs and housing. For effective and holistic solutions to the low-income worker accessibility solution, a database integrating information supporting all three strategies above is fundamental, in order to identify sub-markets that are likely to benefit most from a particular strategy.

To facilitate the planning and development of effective cross-sectoral strategies, a Geographic Information System that ties together data from the transportation, affordable housing and economic development sectors is currently under development. This paper will describe the architecture of this GIS and the data components of the system. A survey of urban geospatial data managers in relevant sectors in the Chicago area will drive the data requirements of the integrated system. Further, data quality issues of the integrated geospatial data will be described, focusing on coverage/scope, consistency, completeness, accuracy, timeliness and accessibility. Since this GIS is an integration of existing geospatial data, the scope of conflation tools and the impact of spatial aggregations in the disparate input databases will be documented. Since data specialists from the different sectors "speak different languages", the development of effective metadata is

also crucial. The paper will conclude with examples of the value-added information emerging from the integration process.

Key words: —data integration—data quality—GIS—low-income accessibility—transportation planning

INTRODUCTION

In recent years, the development of planning and evaluation tools for accessibility to economic opportunities by low-income has received increased attention. Entry-level job growth, which are appropriate for the skill level of welfare clients and low-income individuals, are mostly occurring far away from inner city neighborhoods or isolated pockets in the suburbs, where bulk of low-income and economically deprived families live. Many job-rich areas do not have affordable housing, including affordable rental housing, or there are implicit barriers to fair housing. Transit connections between job rich areas and where the bulk of the low-income and welfare recipients live (inner city neighborhood and isolated pockets in the suburbs) are limited and in some cases, non-existent. Vehicle ownership rates among such individuals are also very low. Most individuals in this category also do not own drivers licenses. Since many entry-level jobs which is the appropriate job pool for the educational and skill levels of individuals transitioning to work start at off-peak times, job locations that were accessible during normal business hours were inaccessible during off-peak hours, since most transit services served peak hour transit markets.

As transportation planners are increasingly called on to address such social considerations beyond historical mobility-based considerations in transportation planning, methods and tools that allow the study of these issues and which focus on accessibility concerns are needed (*1*). Motivated by these considerations, this paper describes the development process of one aspect of a Spatial Decision Support System (SDSS) that is currently undergoing development in the University of Illinois at Chicago (*2*). This SDSS is intended to help planners develop projects and policy makers take decisions about the Labor Market and Transportation (LMT) problem. The LMT problem is defined as "*the synergistic strategies in the transportation, housing and economic development sectors such that spatial access to jobs is facilitated for low-income individuals*". The high-level architecture of the LMT-SDSS is described in a design document (*2*). The LMT-SDSS has two major components: a geospatial data component, and a decision-support component using the Analytical Hierarchy Process (AHP) (3). In this paper, we restrict our attention to the GIS component.

This paper describes the system development approach, the goals and objectives of the system and the criteria used a system design principles. First, the system described here is being developed by integrating the perspectives of key LMT stakeholders in the Chicago region from the transportation, housing, economic development and human services sectors with prior, ongoing work in the transportation sector on the same issues. Stakeholders provided their perspective on technical and institutional barriers to cross-sectoral data integration as well as their "requirements" on data content and media/format for information dissemination. The process of developing the GIS component of the LMT-SDSS, along with problem area objectives and Measures of Effectiveness (MOE's) that serve as system design principles are described in Section 2. This section has several different subsections describing information content, system functions, system design. And second, it summarizes the process of developing the GIS portion of the LMT-SDSS is described, given the considerations described in the paper. Future work is described in Section 4.

GIS DEVELOPMENT PROCESS

To extend the learning that took place from the welfare to work and the EJ process to systematically include the needs and perspectives of the non-transportation stakeholders in the LMT problem area, we conducted a survey of urban geospatial data managers and policy makers from all four sectors (transportation, housing, economic development and human services). These combined perspectives drove the information base for the LMT-SDSS. The survey was administered in a face-to-face interview setting. The survey included multi-choice, Likert-type scales as well as open-ended scenario based questions on three major issues: data/information content needed for LMT problem solutions, data dissemination methods/media and finally, perceived technical and institutional barriers. The methods used for data reduction included Soft Systems Methodologies (*4*) as well as an expert-opinion rating approach that is presented here.

Goals, Objectives and Measures of Effectiveness

The high-level goal that the GIS component addresses is to enhance information availability for the identification and implementation of LMT projects by closing the gap between existing, piece-meal information availability and desired integrated state of information availability. Information availability is enhanced when data from the different sectors are gathered together in a repository and warehoused in one place, available in one platform and "cross-linked" in some sense, information processing is seamless for data managers from all sectors and information dissemination tools and media support desirable knowledge-seeking.

This high-level goal was transcribed into two more targeted objectives, from which we developed Measures of Effectiveness (MOEs) which would ultimately drive the GIS design. The specific objectives that translate this goal into operational benchmarks guiding system development are:

1. Increase information and LMTP decision support;

2. Enhance communication and LMTP partnership development potential.

These objectives can be seen in Figure 1. The horizontal axis is the Information and Decision Support dimension and the vertical axis, the Communication and LMTP partnership dimension. The ideal system would enhance information availability such that the vertex (1,1) is reached. The MOE's that would maximize both dimensions are also shown in the figure.

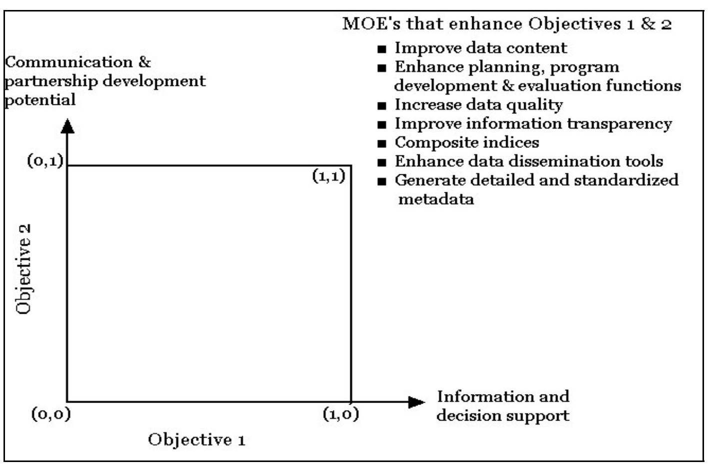

FIGURE 1. GIS Development Principles - Objectives and Measures of Effectiveness

Objective 1: Information and decision support refers to the ability of the GIS to provide reliable, credible and "good-quality" data to raise awareness of the problem, identify potential sub-populations and geographical areas in need of LMT solutions and develop potential courses of action that may benefit target populations and areas. To evaluate if the GIS increases this objective, it becomes necessary to examine the following MOE's:

- MOE1: Content/type of information that the system supports including core (inherited) data and composite (derived) data or indices

- MOE 2: Functional capability of the system (what the data are able to do) and

- MOE 3: Data quality.

Objective 2: Communication and partnership development potential of data, on the other hand, refers to the potential of the information to reach out to potential collaborative partners for developing and implementing LMT projects. As indicated in Section 1, development of partnerships is critical for the implementation of LMT projects and the problem awareness creation aspect of Objective 1 should be augmented by the strength of evidence of collaborative benefits. To meet this objective, it is useful to consider the following MOE's:

- MOE 1: Transparency, intuitive and ease of understanding of data.

- MOE 2: Cross-linkage capabilities and composite indices (it will be seen later in the section that this MOE overlaps with MOE 1 of Objective 1).

- MOE 3: Attractive and easy to use information dissemination tools.

- MOE 4: Detailed and standardized metadata.

Data Content

There are many ways in which a typology of data in the GIS may be developed. For example, a typology may be developed simply on the basis of the sector: transportation data, housing data and so on. While this is an important perspective on the data, a more meaningful typology for the purposes of development are whether the data are core data or derived data. The contents of the GIS data warehouse as desired by the respondents include both core data and composite derived data. Table 1 shows the core (inherited) data that was desired whereas Table 2 shows the composite (derived) data that respondents would find useful. As described in Section 1, core data acquisition is a matter of importing existing data in the four sectors that are deemed relevant to the problem at hand and as such is a matter of data acquisition issues such as identifying the right contact person, signing (if necessary) data sharing agreements, checking data for consistency, completeness and other data quality indicators that will be identified in Section 4. On the other hand, composite data are those indices or estimates that require not only the core (raw) data but also a model of the underlying situation.

It quickly emerged from the survey that in all sectors, a measure of "spatial accessibility" (to jobs, social services and other destinations that allow the fulfillment of necessary activities) is highly desirable. It is important to note here what is actually a derived or composite measure in one sector may be perceived as raw data by another sector. A case in point, again, are accessibility measures. The transportation and regional science literature has seen accessibility measures going through an evolution from Hagerstrom's time-space accessibility concept, the Hansen-type measure to the spatial interaction based measures that can be viewed as a "weighted index of travel cost and competition for the destination activity". We found that when respondents unfamiliar to the transportation literature talked about accessibility, they usually conceptualized it as spatial proximity or in terms of travel times alone.

The survey results indicated that lack of transparency of composite or derived data in one sector by stakeholders in other sectors pervasive, but also that within the same sector, misconceptions regarding these differences persist across organizational levels (data manager versus administrative/executive staff). From the small survey that we did, it is not possible to generalize whether cross-sectoral differences in lack of transparency regarding data are greater or less than cross-organizational differences. However, we believe that the creation of necessary metadata and qualitative description of data would have high educational value.

TABLE 1. Core Data Needs Identified by Survey Respondents from Different Sectors

	Information Expectations		
	Transportation	**Housing**	**Economic Development**
Transportation	Origin and destinations of trips	Location of affordable housing, food shelters, emergency housing shelters	Location of one-stop centers
	Location of transportation services in relation to daycare, hospitals, jobs, one stop centers, schools and residences	Housing quality, availability, rents, costs and other housing characteristics	
	Transit operations	Crime rates, location of schools and other neighborhood quality data	Location of entry-level jobs
Housing	Data on transportation opportunities; Data on the proximity of transportation to housing locations	Location of affordable housing and household sociodemographicinformation	Employment statistics
			Section 8 voucher recipients' location.
Economic Development	Location of transit hub	Location of housing, affordable housing and housing characteristics	Employment and poverty statistics; Location of low wage jobs and industries.

The above table serves as a first step toward addressing the metadata needs and should be interpreted thusly. The three sectors of transportation, housing, and economic development are portrayed in the three rows, while across the table, in the columns are the data pertaining to each of these three sectors. Thus, reading from left to right in the first row of the table, we have the responses from survey participants affiliated to the transportation sector. The first column indicates the transportation data that they are used to working with and the remaining two columns portray the data needs from the housing and economic development sectors that transportation planners would like to know more about and possess in order to move toward a more holistic decision-making process. Since the survey respondents were a mix of people with varied background and training, it was difficult to elicit any further specificity about the data than that reflected in the table.

As a result of these issues, we used open-ended discussions and prior knowledge to separate out information desired in the GIS into core data (primary and secondary data sources that which we can simply inherit from one or more sectors) and composite (derived) data (that which we need to model according to core data, some modeling rules and assumptions).

TABLE 2. Composite Data Needs Identified by Survey Respondents from Different Sectors

Transportation	Housing	Economic Development
Accessibility of system to daycare, hospitals, jobs, one stop centers, schools and residences.	Location of affordable housing at the trip origin and transportation hubs.	Clusters of unemployment
Index to measure user perceptions on the reliability of the transportation system		
Index on customer satisfaction.		
Index of transportation barriers of low-income population.	Accessibility of affordable housing locations to jobs, employment and social services.	Jobs at trip destination
Measure of transportation as an index of opportunities.		
Accessibility of transportation to affordable housing locations		
Transportation connections and costs between origin and destination	Socio-demographic information on income, race, population of households.	Information of low-income population at an aggregate level or index.
Transportation system characteristics, operations and performance data	Neighborhood quality indices	Job growth rate indices
Socio - economic/demographic information on, traveler information	Gentrification rate indices	

Metadata and Data Quality

As indicated earlier, the LMT-SDSS GIS inherits core raw data from different sectors, to the extent possible and derives composite indices on the basis of the core data. Hence, it is a large-scale data integration exercise. Nevertheless, the data quality assessment aspect is as important here as in the case of other data warehousing exercises. Since we are dealing with three totally different groups of stakeholders with wide differences in their view of data, approaches to dealing with data, data use and data histories, the creation of metadata is crucial. While metadata is often tersely described as “data about data”, in the current context, metadata consists of structured information that describes, explains, locates or otherwise makes it possible to retrieve, use or manage an information resource. The LMT-SDSS metadata describes the whole dataset: the data, its attributes and most importantly, its quality; they help to organize data collections, manage data resources and provide documentation for users.

Metadata does not include actual data sets and is instead a documentation of the content and quality of the data set it intends to describe. The Federal Geographic Data Committee (FGDC) created in 1990 has set the Content Standard for Digital Geospatial Metada (CSDGM). The FGDC has also been associated with the International Organization for Standardization Technical Committee 211 to develop a metadata standard.

In order to create the metadata for a data set, one has to understand the data set clearly and also decide on the metadata standard to adopt. The USGS has outlined a four-step process in the creation of metadata (http://geology.usgs. Gov/tools/metadata). The four steps are (1) to assemble information about the data set, (2) to create a digital metadata file, (3) decide on the syntax of the metadata file, and (4) to ensure the accuracy of the metadata by checking the subject and content. The subject and content of the data sets can be kept under control by ensuring high standards of data quality. This exercise is a difficult enough task when dealing with just one data set or data sets from one sector. In the case of a cross-sectoral approach as adopted in this research, it becomes complex and this section focuses the attention on the difficulties embedded in working with diverse data sources and data sets.

The linking of separate databases gives rise to its own types of data quality issues (*2*). A key factor in the level of success of the data integration process with relational databases is the quality of the key/identification variables in the separate, input files. With spatial data, where disparate data are linked together on the basis of one or more spatial attributes, the level of "cleanliness" with the spatial identification data is critical.

TABLE 3. Data Quality Dimensions and Criteria for the LMT-SDSS GIS.

Dimensions	**Data Quality Criteria**	**Description**
Contextual	Coverage/scope	Extent to which GIS repository covers sectors and attributes pertinent to LMT project development and collaboration development analysis
	Completeness	Degree to which key identifier variables have values
	Consistency	Degree to which separate, input databases from different sectors are spatially and temporally consistent.
	New Value Created	Extent to which linked repository adds new information for LMT-SDSS objectives over and above that possible from analysis of separate, input databases
	Timeliness	Degree to which specified data values are up to date and reflects the temporal updating cycle of input datasets
Intrinsic	Accuracy	Extent to which key data elements are error-free and measures the intrinsic properties of attributes
Representational	Documentation & metadata	Degree to which print and online documentation of input databases are available in a timely, accurate and readable format for free or at nominal cost.
Accessibility	Ease of use and transparency	Degree to which LMT-SDSS information uses input databases that are accessible easily, without written contracts, data sharing agreements and special permission and also the extent to which users can use information from the system without a great deal of prior knowledge and in a format/media that can be used easily using standard equipment and software

We identify four core dimensions of data quality consisting of eight measurable criteria (Data Quality Criteria) that should be satisfied by the GIS. These are shown in the left-most column of

Table 3. The second column of the table shows the DQC's that operationalizes the core dimensions.

While researchers have accepted no single definition of data quality, there is agreement that data accuracy, currency, and completeness are important areas of concern. Based on prior work that one of the authors of this paper had conducted with data linkage and data quality in the transportation safety sector, we describe here a framework for data quality assessment for the LMT-SDSS GIS.

In order to integrate the data from Table 1, the following steps are suggested: 1. Identify data sources with the relevant data. 2. Decide on a spatial aggregation unit, for instance, census tracts or quarter sections. 3. If datasets are GIS coverages, make sure the projection is compatible with other datasets. 4. If datasets are in any text format such us excel or dbase, decide on what software should be use to clean and join datasets, our suggestion is to leave the datasets independent and join the tables. 5. Create the necessary composite indices according to the planning needs, for example accessibility and/or mobility measures. These same steps will be followed in the implementation of the case study.

It is clear from the discussion on data quality that building the metadata and ensuring data quality are tied together in more ways than one and are intrinsic to the successful development of a prototype. The prototype design envisioned for this research involves the use of the ArcGIS suite of software.

The system design for the prototype is as follows: It is embedded on Linux, and Oracle platform.

The data are imported into Oracle and the metadata are extracted with the help of ArcGIS ArcCataglog. These metadata documents are stored in ArcSDE and ArcIMS hosts the data application. In other words, ArcIMS enables the dynamic querying capabilities to the end-user at a remote location (ESRI, 2002).

SUMMARY OF SYSTEM DESIGN PROCESS

The data architecture of this system is unique; nevertheless it may be implemented using different combinations of software and hardware. It is necessary to incorporate (1) a database management tool, (2) a geographic information software, (3) an interface for the former two, and (4) a dissemination processor. The system designed by the Urban Transportation Center is the result of the human and technical resources available. Figure 2 shows the data sources and steps necessary for the design of the system. Similarly, it shows the two possible outputs of the system: (1) a GIS tool for displaying results and (2) an instrument to perform scenario analysis to tackle problem situations and develop policy guidelines.

For the implementation of the proposed system, a data warehouse is being deployed in a Linux Intel server (Linux 9.3), in Oracle 9i Database Release (9.2.0.1.0). The system has two data dissemination aspects: off-line/static and online/dynamic. The purpose of this dissemination falls into three different levels of end-uses: (1) long-term strategic planning decisions, (2) short-term policy analyses, and (3) project coordination purposes.

The off-line/static dissemination pertains to the derived data developed in-house at the Urban Transportation Center (UTC) and will be in the form of graphs, static maps, and tables. Data from the Consumer Expenditure Survey (CES), the Current Population Survey (CPS), and the

Decennial Census are included as part of this dissemination effort. For example, the Current Population Survey releases information about various aspects such as the labor force, industry, work times, etc. at different times of the year. The raw data gives information, say, about the work start times of a sample of the population. This data can be enriched by developing a model to predict job start times, if the information about an individual's area of work (industry), age, education level, gender, etc. are known. This information in turn can help transportation/transit planners to plan for the future based on the travel patterns of people in a region.

The on-line/dynamic data pertain to the data from the three sectors and will be mounted on the ARC-GIS tools for dissemination. For example, data from the census can be queried on-line to extract a wealth of information pertaining to a range of transportation related issues. This can also be combined with data from a region's transit provider/network of roads to enrich this information.

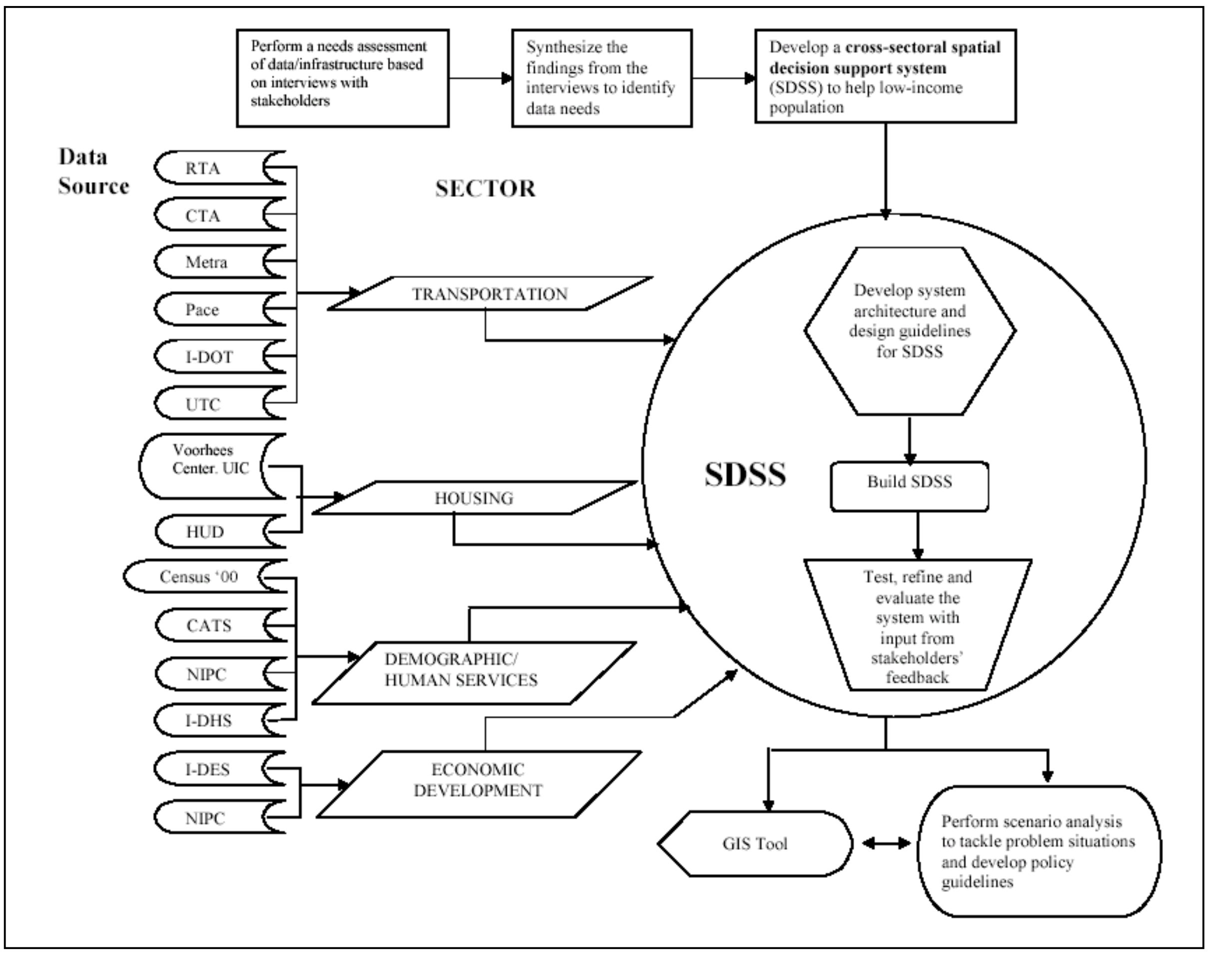

FIGURE 2. System Design Process

CONCLUSIONS AND RECOMMENDATIONS

It is a titanic mission to assemble all the data from the various sectors, therefore this data architecture will be built by the incremental inclusion of more variables, indices and models that will lead to the creation of different applications and eventually to the creation of Participatory-GIS. As a subset of the proposed SDSS, general public will be able to identify information such as housing, daycare, job locations, transportation network and transit. Data will be made available at any feasible level of resolution that the researcher might seek. The difficulty typically arises in either collecting the data, or in integrating different data sets together to enrich them. The barriers to data collection are addressed next followed by issues associated with data integration.

Organizational barriers: (1) Data analysts and data managers are almost possessive about the derived data that they develop and are more often than not reluctant in sharing their efforts with people outside their own department/agency. (2) At a higher level is the nature of the data, i.e., whether it affects individuals directly or reveals the identity of the individuals that lead organizations to guard them vehemently.

Geographical barriers: (1) Data are typically collected at various levels of resolution depending upon the objective of the research that they are being collected for. Thus, when data are shared outside of their own research environment, the question of aggregation/disaggregation becomes a big issue that presents as a barrier to integration of data.

Temporal barriers: Similar to the geographical barriers are the temporal barriers, which can be both microscopic and macroscopic in nature. They can be pertaining to the time of day/month that the data were collected or there can be issues about integrating data from different time periods. A typical example can be comparing the decennial census data from the year 2000 with the Chicago Transit Authority's (CTA) ridership information from, say, 1995.

Technological barriers: While this can be overcome with the right training and investment, organizations should ensure that it is not overlooked. It is very essential to ensure that the appropriate training methods are made available, and monies are set-aside in order for individuals to cross the technological divide.

The organizational, geographic, temporal, and technological barriers are significant, as are the issues associated with data integration. The most important aspect of data integration in such efforts, as discussed in this paper, is to ensure that the quality of data are not compromised to satisfy short term goals/needs of decision-makers. It is in this respect that developers of such systems should understand the marriage between data quality and metadata and how they feed off of each other.

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Adaptability of AASHTO Provisional Standards for Condition Surveys for Roughness and Faulting in Kansas

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ABSTRACT

The Kansas Department of Transportation (KDOT) currently uses a comprehensive, network-level pavement management system known as Network Optimization System (NOS). Annual condition surveys for roughness and faulting generate important inputs for NOS. Recently, AASHTO has published provisional standards for condition surveys in order to harmonize data collection efforts among the states. To study the effects of these provisional standards on KDOT NOS, profile data was collected on about 346 km (215 miles) of Kansas highways following these standards. The comparison data came from KDOT's annual condition survey using KDOT standards. The roughness values, in terms of International Roughness Index, IRI, were computed and aggregated for 20 test sections and the faulting values were computed and compared for four test sections. Various statistical analyses compared the results from the algorithms following KDOT NOS and the AASHTO provisional standards. The roughness measurements and subsequent analysis using AASHTO provisional standard PP-37-00 and current KDOT methodology tend to produce statistically similar results. This may indicate this standard (PP 37-00) can be adopted for NOS without any major changes in current practice. However, significant differences were found in calculated fault values computed from the two methods even after some modification to PP 38-00 following current practices in Kansas.

Key words: condition surveys—faulting—pavement management system—roughness

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INTRODUCTION

The Kansas Department of Transportation (KDOT) uses a comprehensive, successful pavement management system (PMS). The network level PMS of KDOT is popularly known as the Network Optimization System (NOS). In support of NOS, annual condition surveys are conducted based on the methodologies proposed by Woodward Clyde consultants (now URS Corp.) and subsequently, refined by the KDOT Pavement Management Section. Current annual condition surveys include roughness, rutting, fatigue cracking, transverse cracking and block cracking for flexible and composite pavements, and roughness, faulting and joint distresses for rigid pavements. Different severity levels and extent are measured in the survey. While the roughness, rutting and faulting data are collected using automated methods, cracking and joint distress surveys are done manually. These survey results constitute basic inputs into the NOS system. The performance prediction methodology in the NOS system is based on the Markov process. The technique uses transition matrices to predict future condition based on current condition for multi-year programming. Thus, the surveys are essential to define the current condition *(1)*. Similarly historic data is used to generate and update the transition matrices, so historic data consistency is also essential.

PROBLEM STATEMENT

FHWA published protocols developed in 1996 by Texas Research and Development Foundation (TRDF) for condition survey data collection. These protocols were developed primarily for the Highway Performance Monitoring System (HPMS) reporting with the eventual goal of using it in Pavement Management Systems (PMS). The objective was to harmonize condition data collection among the states. The American Association of State Highway Transportation Officials (AASHTO) has modified and adopted these protocols as Provisional Standards. The AASHTO provisional standard for quantifying roughness is PP 37-00 and PP 39 -00 is for faulting (*2*). Each of these standards differs some from the KDOT method. For instance, these standards ask for the metric (0.1km) aggregation for data collection and analysis.

As mentioned earlier, in support of NOS, KDOT conducts annual condition surveys, and the results of these surveys constitute basic inputs into the NOS systems. Preliminary investigations of these standards on KDOT practices presume the impact to be severe. However, no data had been collected using these standards from which the impacts can be evaluated. KDOT reports roughness to FHWA in terms of International Roughness Index (IRI) from actual profile measurements, which is required in AASHTO PP-37. But, KDOT uses a 0.062-km (0.1-mile) aggregation for data collection, processing and analysis. KDOT’s automated fault detection algorithms are based on first identifying potential fault locations and then quantifying the extent of the fault. AASHTO PP39-00 contains a similar approach to the extent determination, but has no means of identifying a potential fault location.

ROUGHNESS

Road roughness is an important attribute in evaluating pavement condition because of its effects on ride quality and vehicle operating costs. In its broadest sense, road roughness has been defined as “the deviations of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamics loads, and drainage” (*3*). The current practice is to measure roughness in terms of the longitudinal profile of the road surface since it is known to cause vertical acceleration of vehicles, and in turn, user discomfort.

Recent surveys by the National Partnership for Highway Quality (NPHQ) and the Federal Highway Administration (FHWA) have shown that the primary concern for the motoring public is road condition, or pavement smoothness. Currently, most state highway agencies (SHAs), as well as many county and municipal agencies measure pavement smoothness on pavement rehabilitation and construction projects. Because some types of roughness are damped by vehicle dynamics and others are amplified, a statistic to quantify roughness transferred to vehicle occupants, International Roughness Index (IRI), was developed. The statistic allows road profile data to be compared as road users experience it. Unfortunately, questions remain about the effects of pavement surface temperature, texture, moisture, signal processing (noise and filter impacts) and other extraneous variables on both profile and IRI.

FAULTING

Faulting is the difference of elevation across joint or crack (*4*). Faulting is considered an important distress of jointed plain concrete pavements (JPCP) because it affects ride quality. If significant joint faulting occurs, there will be a major impact on the life-cycle costs of the pavement in terms of rehabilitation and vehicle operating costs. Faulting is caused in part by a buildup of loose materials under the trailing slab near joint or crack as well as the depression of the leading slab. Lack of load transfer contributes greatly to faulting (*4*).

TEST TRACK SELECTION

In order to assess the effects of the AASHTO provisional standards on the profile data analysis, data was collected on a test track located in northeast Kansas. The track consists of 20 sections of asphalt, portland cement concrete, and composite pavements with an approximate total length of 346 km (215 miles) as shown in Table 1. The track was selected to be representative of all bituminous and composite pavements mileage in Kansas in terms of both total mileage and pavement condition. All test sections, except one, are on two-lane roads with varying shoulder width.

TABLE 1. Test Sections Selected for Study

Section	Route	County	Begin County MP	End County MP	Type
1	177	31	13.413	0.013	BIT
2	56	56	0	14.069	BIT
3	56	70	0	12.936	BIT
4	56	70	13.447	32.971	C0MP
5	24	44	6.516	0.154	C0MP
6	24	89	17.739	0	BIT
7	16	43	0	14.825	BIT
8	4	44	19.465	10.317	BIT
9	192	52	0	8.51	BIT
11	73	52	17.171	8.815	PCC
12	73	7	9.461	20.906	BIT
13	36	66	23.987	9.732	BIT
14	36	58	25.145	9.207	BIT
15	15	101	18.91	12.094	COMP
16	9	101	26.247	14.253	BIT
17	24	81	0	2.478	BIT
18	82	81	11.859	7.924	BIT
19	70	31	7.124	11.282	PCC
20	18	31	15.341	18.044	PCC
21	18	81	0.234	2.349	PCC

PROFILE DATA COLLECTION

Pavement profile data consists of elevation measurements at discrete intervals along a pavement surface. The data collection program on the road profilers store its data in the binary format to save both space and time while creating the files in the vehicle. While these binary files are efficient, they cannot be reviewed or edited with standard text editors. A software program, such as the ICC report program, is necessary to process the data into reports and to convert the data to text files that may then be read and processed by standard software programs. RP090L is one such report program used to report data from these road profilers. The profile data for this project was analyzed with RP090L v 3.34 for computing pavement roughness statistic (IRI) and faulting. The inputs to this software are event files, speed files, and profile files.

Profile data on the test track was collected on both wheel paths by KDOT using an International Cybernetics Corporation (ICC) South Dakota-type profiler. Data collection was done at highway speeds (usually 50 mph or 80 km/h). The sensors measure the vertical distance from the vehicle body to the pavement surface, and the profiler is equipped with accelerometers at each of the wheel path sensors to compensate for the vertical motion of the vehicle body. The KDOT ICC profiler is equipped with three Selcom 220 laser sensors. The outer two sensors are spaced at about 1.67 m (65.8 in.) apart. The third sensor is located in the middle. KDOT profiler aggregates profile elevation at every 75 mm (3 in.).

Longitudinal profile data was also collected by CGH Pavement Engineering, Inc. of Mechanicsburg, Penn. using an ICC profiler. That data was collected and reported at 145 mm (approx. 6.07 in.) intervals.

ROUGHNESS MEASUREMENT

The pavement engineer cannot readily use raw profile data. The profile, by itself, does not represent how rough a road is. It must be processed in some manner to produce a meaningful representation of the pavement roughness. A number of summary statistics are available to represent road roughness using road profile data. International Roughness Index (IRI) is most commonly used by many agencies since IRI is required by FHWA for HPMS reporting. The IRI was developed mathematically to represent the reaction of a single tire on a vehicle suspension (quarter-car) to the roughness of the pavement surface, traveling at 80 km/h (50 mph). IRI is expressed in mm/km (inches/mile).

The RP090L software uses the following steps to produce IRI from collected data (*5*):

1. Combine height sensor and accelerometer data together for each wheel path.
2. Filter data and produce profile points for each wheel path.
3. Put profile into a quarter car simulation at 80 kmph (50 mph) to create a vehicle response set of data points.
4. Put response points into a formula that calculates the IRI using sum of rectified slopes.
5. Produce an IRI report broken up in intervals, sections, or the combination of the two.

The software gives the IRI values for both left and right wheel paths. Average IRI value was used in this analysis.

AASHTO Procedure

AASHTO provisional standard for roughness, PP 37-00 specifies 0.1 km (0.0625 mile) aggregation steps in IRI computation. The standard also calls for a quality assurance plan encompassing certification and training, equipment calibration, verification sections, and data quality checks.

KDOT Procedure

The current KDOT procedure is to get IRI values using 0.16 km (0.1 mile) aggregation steps. KDOT uses the RP090L software, supplied by ICC, to compute the IRI values.

FAULTING MEASUREMENT

KDOT Procedure

The fault values are calculated from the profile data using an algorithm developed by KDOT internally. In this process, anytime the absolute relative elevation difference between two points at 150 mm (6 in) intervals for the right sensor from the output of RP090L exceeds 2.3 mm (0.09 inch), then the relative elevation difference (fault) values are algebraically summed until either three consecutive fault values are less than 2.3 mm (0.09 in) or 0.9 m (3 ft) has been traversed. The calculated fault value would be the algebraic sum of the points divided by two. Once a fault has been detected, the next fault must be located at least 3.05 m (10 ft) away. A 0.16-km (0.1-mile) aggregation was used for the data analysis.

AASHTO Procedure

The AASHTO protocol for faulting PP 39-00 specifies a 0.1-km aggregation. For automated surveys, relative elevation measurements must be taken at points 75 to 225 mm (3 to 8 in.) from the joint or crack and separated by 300 mm (11.8 inch) as shown in Figure 1. The faulting value should be recorded only when it exceeds 5 mm (0.2 in). It also states that care must be taken not to measure spalling and report it as faulting. However, the proposed standard does not specify any particular way to do that.

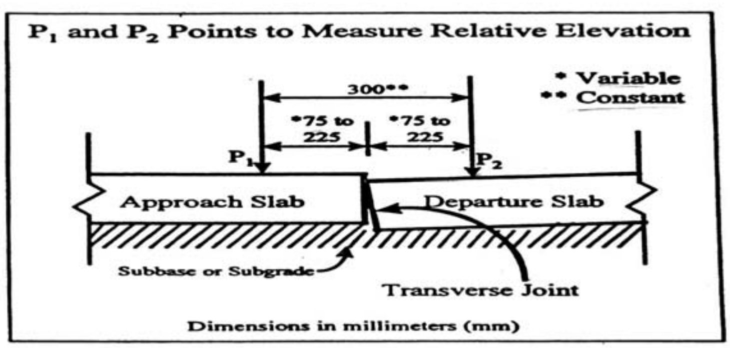

FIGURE 1. Faulting Measurements According to AASHTO PP 39-00 (after *2*)

DATA AGGREGATION AND ANALYSIS

Roughness

KDOT Methodology

The aggregation interval used was 0.1 mile. Data on less than 0.16–km (0.1-mile) segments was omitted from the analysis, following the practice of truncating data for NOS input. Roughness is calculated as the average of the two IRI values in left wheel and the right wheel path (historically KDOT has used the right wheel path only following HPMS guidelines). Then the average roughness for every 1.6 km (one-mile) segment was computed as a length-weighted average.

AASHTO Methodology

The aggregation interval used was 0.1km. Data on less than 0.1–km (0.0625-mile) segments was omitted from the analysis, following the practice of truncating data for NOS input. Roughness is calculated as the average of the two IRI values in the left and right wheel paths. Then the average roughness for every 1.6 km (one-mile) segment was computed as a length-weighted average.

Comparison of Roughness Data

Statistical analysis techniques were used to compare the roughness values from both algorithms. The roughness values were initially tested for normality and equality of variances using the Shapiro-Wilk and Levene's tests at 10% level of significance (*6*). If the data were normal with equal variances then the equality of means was tested using the Analysis of Variance (ANOVA) F-test at 95 % confidence interval (5% level of significance). If the normality and variance tests failed, Kruskar-Wallis nonparametric test was done (*6*). If there were only a few data points then the Welch t-test was performed to test for the equality of means at 95% confidence interval (level of significance = 5%) (*12*). All statistical analyses were performed using the MINITAB software (*7*). The results are shown in Table 2. The roughness values from both methodologies were statistically similar for all sections.

TABLE 2. Comparison of Roughness Measurements (m/km) from KDOT and AASHTO Methodologies

Sec	Route	Begin MP	End MP	Type	KDOT Roughness	AASHTO Roughness	p-Value	Similar
1	177	13.413	0.013	BIT	0.848	0.851	0.887	Yes
2	56	0.029	14.069	BIT	1.014	1.058	0.646	Yes
3	56	0.061	12.936	BIT	1.552	1.548	0.96	Yes
4	56	12.936	32.971	BIT	1.716	1.712	0.932	Yes
5	24	6.516	0.154	C0MP	1.500	1.510	0.95	Yes
6	24	17.739	0	C0MP	1.069	1.091	0.68	Yes
7	16	0.032	14.825	BIT	0.953	0.948	0.834	Yes
8	4	19.465	10.317	BIT	1.075	1.074	0.991	Yes
9	192	0.092	8.51	BIT	1.059	1.295	0.064	Yes
11	73	17.171	8.815	PC	1.8	1.8	0.878	Yes
12	73	9.461	20.906	BIT	1.071	1.062	0.603	Yes
13	36	23.987	9.732	BIT	0.882	0.890	0.963	Yes
14	36	25.145	9.207	C0MP	0.942	0.942	0.992	Yes
15	15	18.91	12.094	BIT	1.238	1.246	0.89	Yes
16	9	26.247	14.253	BIT	1.153	1.150	0.963	Yes
17	24	0	2.478	BIT	1.529	1.506	0.951	Yes
18	82	11.859	7.924	BIT	1.036	1.043	0.913	Yes
19	70	7.124	11.282	PC	1.5	1.4	0.352	Yes
20	18	15.341	18.044	PC	2.0	2.0	0.818	Yes
21	18	0.234	2.349	PC	2.289	2.594	0.65	Yes

Comparison of Roughness Data from the CGH ICC Profiler

Similar statistical approach was followed to analyze roughness data from the CGH ICC profiler according to the KDOT and AASHTO methodologies discussed earlier. The results from the statistical analysis show that the roughness values from both methodologies were statistically similar for all but two sections (Sections 1 and 14).

Faulting

KDOT Methodology

The aggregation used was for the analysis was 0.16 km (0.1 mile). The fault detection and calculation methodologies described earlier were followed. The absolute fault values were used for the analysis.

AASHTO Methodology

The aggregation used was 0.1 km (0.0625 mile) and the KDOT profiler data was used for analysis. As per PP39-00, the relative elevation difference (fault) measurements must be taken at 75 to 223 mm (3 to 8.8 in) from the crack or joint and separated by 1300 mm (approximately 12 inch). Since this is difficult to quantify from the output of RP090L, when the fault value from the right sensor exceeded 2.3 mm (0.09 in), only then the fault values were computed. This was done to be consistent with the KDOT method. The fault value was taken as the sum of that value and the consecutive identifiable fault values after that, and then divided by two. The fault value was reported only if it exceeded 5 mm (0.2 inch) per PP 39-00 requirements. A minimum distance of 3.05 mm (10 ft) was maintained between two successive fault values. Although this was not specified in the AASHTO PP 39-00, it also was done in this study to be consistent with the KDOT methodology and to eliminate the possibility of reporting spalling as faulting.

Revised Method (a)

In this method, the aggregation interval used was 0.1 km (0.0625 mile) and all fault values obtained from the AASHTO methodology were used for the analysis (unlike PP 39-00 where the faulting values greater than 0.2 inch must be used for analysis).

Revised Method (b)

In this method, 0.1 km (0.0625 mile) aggregation interval was used and the KDOT methodology was followed in the analysis.

Statistical analysis, for comparing the results from two methodologies, was done in the same way as roughness results. The results for the PCC sections of the test track (Sections 11, 19, 20 and 21) are summarized below:

Section 11:

- The fault values obtained from the KDOT and AASHTO methods were significantly different for all one-mile segments.
- The fault values obtained from the KDOT and AASHTO (revised methods (a) & (b)) methods were statistically similar for all one-mile segments.

Section 19:

- The fault values obtained from the KDOT and AASHTO methods were significantly different for all one-mile segments.

- The fault values obtained from the KDOT and AASHTO (revised methods (a) & (b)) were statistically similar for all one-mile segments except one for the revised method (a).

S*ection 20:*

- The fault values obtained from the KDOT and AASHTO methods were significantly different for all one-mile segments.
- The fault values obtained from the KDOT and AASHTO (revised methods (a) & (b)) were statistically similar for all one-mile segment.

CONCLUSIONS

The following conclusions can be drawn from this study:

- The KDOT and the AASHTO algorithms compared well for the roughness statistics determination. All test sections had statistically similar IRI values. On these sections, the effects of 0.16 km (0.1 mile) and 0.1km data aggregation are insignificant.
- The KDOT and the AASHTO algorithms for faulting produce significantly different fault values.
- The KDOT and the AASHTO (revised method (a) & (b) following somewhat KDOT algorithm for fault detection and threshold fault values) algorithms had statistically similar fault values.

RECOMMENDATIONS

The following recommendations can be made from the above study:

- The AASHTO provisional standard PP 37-00 is compatible with the current KDOT practices and can be implemented easily.

- Although modified AASHTO method PP 39-00 produced fault values similar to the KDOT NOS algorithm, further research is needed to investigate ways to recognize the joint or crack in the concrete pavement as specified in the standard. The standard also does not specify ways to recognize faulting at the beginning. More study is needed in this area.

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Development and Maintenance of the Electronic Reference Library

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ABSTRACT

This paper presents a case study of the Electronic Reference Library (ERL) that is produced biannually by the Iowa Department of Transportation. This project began with the goal of producing a CD that contains all of the information necessary for designers, inspectors, and contractors working on transportation projects. This paper details the progression of the ERL from concept to its current production status. Other issues discussed are the benefits of electronic documents, a development strategy based on lessons learned, and the challenge of maintaining a large, cross-referenced, electronic library.

Key words: electronic documents—specifications

INTRODUCTION

Successful construction projects depend on good communication, and for construction projects, drawings and specifications are critical for communicating the design intent. At the Iowa Department of Transportation (Iowa DOT), these documents include the Standard Specifications for Highway and Bridge Construction that contain non-project specific information, Construction Manual for field administration and inspection, Instructional Memoranda (IMs), Standard Road Plans, and the Standard Bridge Plans. These documents are updated twice a year through the General Supplemental Specifications for Highway and Bridge Construction, the Supplemental Specifications, and through updated and additional IMs. The Electronic Reference Library (1) began as a method to take these and other documents and make construction specifications better.

One of the characteristics of the ERL is that it has a well defined target user group consisting of engineers, inspectors and contractors. Understanding users' needs and establishing design requirements for the ERL was the first step in this project. Two focus group meetings and a number of interviews with potential users were conducted from August 1998 to January 1999 (2, 3). The results of these focus groups are listed in Tables 1 and 2. Table 1 lists the desired contents for the ERL, and Table 2 lists the desired attributes of the ERL.

DEVELOPMENT PLAN

The following documents were selected because they are frequently updated and commonly used for construction projects. These documents include:

- Standard specifications
- General Supplemental Specifications
- Materials Instructional Memoranda (IMs)
- Standard Road Plans
- Standard Bridge Plans
- Road Design Aids Manual
- Bridge Design Manual
- Construction Manual

Table 3 lists desired ERL contents that are not included in the current version of the ERL. These suggestions should be considered for future versions.

TABLE 1. Desired Contents of the ERL

Standard Specifications	General Supplemental Specifications
Design Manuals	Road Standards
Materials Instruction Memoranda	Construction Manual
Telephone books	Letting dates
Iowa DOT programs	Trade association manuals
Job site posting requirements and posters	Davis Bacon wage rates
List of who is under contract for which jobs	County IM's
Bid item list	Average unit prices
CFR sections that apply	Iowa Code section that apply
Forms (fill out and send electronically)	Standard proposal notes
Urban Standard Specifications for Public Improvements	ASTM, AASHTO, MUTCD manuals – subject to copyright restrictions
DBE information as of issuance date	Equipments rental rates -- subject to copyright restrictions
Links to internet sites	

TABLE 2. Desired Attributes of the ERL

User-friendly	Good search engine
Easy to read	Printable
Hyperlinks between documents	Able to open multiple windows
Able to fill in and send form	Able to ensure authenticity
Will retrieve correct documents when letting date is input	Provide technical support
Electronic documents similar to paper documents	Make connections between ERL and SiteManager®

Three types of navigation structures were considered, hierarchical, non-linear, and mixed (4, 5, 6). In a strict Hierarchical text, nodes are linked to form a strict hierarchy, in which a node can only access those nodes directly above and below it. In non-linear text, nodes are connected to form a complex network based on a large number of referential links. Mixed text has a basic hierarchical structure with a number of referential links that allow users to jump across the branches of the hierarchy.

The mixed text approach is considered more suitable because of the organization and cross-references in the documents and is actually applied in the ERL.

TABLE 3. Suggested Content for Future Versions of the ERL

Telephone books: Iowa DOT Local jurisdiction officials Contractors Consultants	Links to other internet sites (users could link to the site directly to the CD if on-line) Requirements for job site posters and electronic version of posters that could be printed by user
Davis Bacon wage rates	Computer programs developed by Iowa DOT for use by contractors, consultants, and Iowa DOT employees
Historical units prices	Bid letting dates
Design aids published by trade associations	Iowa DOT web site
Standard proposal notes	Applicable CFR and Iowa Code sections

Hypertext links can be classified as two basic types: navigational and associative links (7). Navigational links connect main content with other sub-content and function as path-finding tools. They serve as backbone of a user interface. Associative links offer parenthetical material, footnotes, digressions, or parallel themes that the author believes will enrich the main content (8, 9).

To avoid or minimize disorientation of the users and other adverse side-effects of using hyperlinks, two approaches of making associative links are discussed:

- Open the referred content in a new browser window
- Use frames

Table 4 presents the status of use of electronic documents in other states. The table shows the states that provide electronic versions of their specifications. The WWW column marks the states that provide the documents through the web, the CD column marks the states that provide the documents on CD, File Format lists the electronic format that is used, Search Engine is marked if they provide the ability to do keyword searching, Internal Hyperlinks is marked if the document contains hyperlinks within the document (i.e. links within the Standard Specification), and External Hyperlinks is marked if links are provided between documents (i.e. links from the Standard Specifications to the Construction Manual).

TABLE 4. Comparison of ERL with other DOTs

State	WWW	CD	File Format	Search Engine	Internal Hyperlinks	External Hyperlinks
AL	☑	☐	PDF	☐	☐	☐
AK	☐	☐		☐	☐	☐
AZ	☑	☐	PDF	☐	☐	☐
AR	☑	☐	PDF	☐	☐	☐
CA	☑	☐	PDF	☐	☐	☐
CO	☑	☐	PDF	☐	☐	☐
CT	☑	☐	PDF	☐	☐	☐
DE	☑	☐	HTML	☑	☑	☐
FL	☑	☐	PDF	☐	☐	☐
GA	☐	☐		☐	☐	☐
HI	☑	☐	PDF	☐	☐	☐
ID	☑	☑	PDF	☑	☑	☑
IL	☑	☑	PDF	☑	☑	☐
IN	☑	☑	PDF	☐	☐	☐
IA	☑	☑	HTML	☑	☑	☑
KS	☐	☐		☐	☐	☐
KY	☑	☐	PDF	☐	☐	☐
LA	☑	☐	PDF	☐	☐	☐
ME	☐	☐		☐	☐	☐
MD	☑	☐	PDF	☐	☐	☐
MA	☑	☐	PDF	☐	☐	☐
MI	☑	☐	PDF	☐	☐	☐
MN	☑	☐	PDF	☐	☐	☐
MS	☑	☐	PDF	☐	☐	☐
MO	☐	☐		☐	☐	☐
MT	☑	☐	PDF	☐	☐	☐
NE	☑	☐	PDF	☐	☐	☐
NV	☑	☐	PDF	☐	☐	☐
NH	☐	☐		☐	☐	☐
NJ	☑	☐	PDF	☐	☐	☐
NM	☐	☐		☐	☐	☐
NY	☑	☐	PDF	☐	☐	☐
NC	☑	☐	PDF	☐	☐	☐
ND	☑	☐	PDF	☐	☐	☐
OH	☐	☐		☐	☐	☐
OK	☐	☐		☐	☐	☐
OR	☑	☐	PDF	☐	☐	☐
PA	☑	☑	PDF	☑	☐	☐
RI	☑	☐	HTML	☑	☑	☐

State	WWW	CD	File Format	Search Engine	Internal Hyperlinks	External Hyperlinks
SC	☑	☐	PDF	☐	☐	☐
SD	☑	☐	PDF	☐	☐	☐
TN	☑	☐	PDF	☐	☐	☐
TX	☑	☐	PDF	☐	☐	☐
UT	☑	☐	PDF	☐	☐	☐
VT	☐	☐		☐	☐	☐
VA	☐	☐		☐	☐	☐
WA	☑	☑	PDF	☐	☐	☐
WV	☑	☐	PDF	☐	☑	☐
WI	☐	☐		☐	☐	☐
WY	☐	☐		☐	☐	☐

IMPLEMENTATION PLAN

Implementation of the ERL was divided into four stages: alpha version, beta version, pilot version, and the first mass published version. A hypertext application development process was adopted as a guideline for implementation of the ERL. Major tasks at this stage included:

1. Selection of document format. Two file formats are used in the ERL: Hypertext Markup Language (HTML) or Portable Document Format (PDF). Documents are converted into HTML files whenever possible. If the documents have complex forms, then they are converted to PDF format, which saves time and reduces translation errors.

2. Preparation of electronic document files. The contract documents were compiled in Microsoft Word, WordPerfect, and Bentley Microstation. Converting these documents to HTML or PDF format and inserting hyperlinks constituted a significant portion of the development work.

3. Implementation of the desired attributes. When designing the user interface for the ERL, the research team worked to achieve usability, consistency, portability, and flexibility.

4. Management of the development team. The project team includes the developers of the original documents plus the ERL team. All teams are working to produce updated specifications twice a year. Task assignment, specialization, communication, and coordination issues were critical to the performance of the project.

Using the information gained from focus meetings, the design team developed an alpha version of the ERL. For the alpha test, the ERL did not need to implement all desired features. The team decided that ii would be better to quickly issue an early copy to test the design concept. Ten copies of the alpha version were distributed to experienced computer users who also were familiar with the hard copy of the specifications. This version yielded the following suggestions:

- Navigation must be quick and efficient

- A good search engine is mandatory

Experienced specification users memorize common specification section numbers, because knowing these numbers provides for quick navigation though the printed specification book. Much as a person reading the local newspaper knows were the weather forecast is printed, and experienced specification users know where to find concrete mix designs without checking the table of contents. The design team responded to this issue by creating a feature called quickjump. Quickjump allows the user to enter a specification number at any time, and the ERL responds by quickly displaying that section. If the section number is not known, then the user can navigate through the ERL using the table of contents, or can use the search function.

One overwhelming comment from all surveys is that a good search engine is critical to the success of the ERL. If the user does not know the location of the desired material in the specification book, then it is often quicker to search for the desired terms than it is to locate the material using the table of contents. The search function also helps when the user remembers some keywords from a section they wish to locate, but does not remember the specification number.

MAINTENANCE

Update and maintenance of ERL is a continuous effort. Every six months, a new revision of ERL, which contains new documents or changes to existing construction documents, will be released by Iowa DOT. Constrained by the limited resources, the process of updating and maintaining the ERL is always a challenge to the project team. After carefully reviewing the ERL development and maintenance process and available technologies, the following improvements were proposed and implemented (10). These included:

- Automating the document conversion process;
- Documenting the ERL update and maintenance process;
- Understanding users' changing needs.

Automating the HTML File Preparation and Modification Process

HTML File Development Process. For a normal document like the Construction Manual or Standard Specifications, a person requires about 10 - 15 working days to finish the task of inserting hyperlinks for all cross references in the documents. If the updater is involved in other works, the job could take more than a month to finish. Also, because reading page after page of specifications for weeks at a time can be tedious work, conversion errors are inevitable.

Typical errors found in the past include:

- Unlinked cross references;
- Broken links;
- Links refer to a wrong document.

Thus, another one or two weeks is required to fix errors in these files.

Fortunately, the documents used for this project are methodically arranged, and cross references can be easily recognized by a computer. Two convenient scripting utilities were identified and introduced into ERL development and maintenance for their capability of manipulating contents of text and binary files using common scripting languages (11, 12). Due to the large size of construction documents and the time required to manually classify these documents, there is an interest in automatically processing these documents (13).

Documenting the Development and Maintenance Procedure

One of the issues that come along with the long time span of the ERL life cycle is capturing the knowledge of the developers and writing a procedure manual. Although most skills needed for ERL maintenance and update are not very complicated, they are not typical skills for those unfamiliar with website development. Each time a new member joins the team, it takes about half a year for these new members to master the skills and get acquainted with the update procedure due to the large size of the specifications, and they are very likely to make mistakes on their first update task. To reduce the time of learning and cost of errors, an instruction manual was compiled to assist successive updaters.

Before the instruction manual was compiled, the only documentation that recorded ERL development and maintenance procedures were the theses written by earlier graduate students. While these theses can provide helpful information for later project members, they cannot take the place of a good instruction manual. Unlike a thesis, which normally focuses on the particular tasks of a graduate student, an instruction manual should document all procedures involved in the development and maintenance process completely and in sufficient detail to allow new employees to learn quickly. Moreover, an instruction manual focuses on how a certain task is accomplished without regard to why this solution is selected.

Once a solution is written in an instruction manual, it becomes the standard solution to be followed by later maintainers. Thus, the solution must be carefully tested and must be proven effective and efficient before being included in the ERL update process.

The instruction manual records the best practices of ERL development and maintenance available to the compilers. As time goes by, new tools and methods may be found by successive updaters. Thus revision to the instruction manual will be an ongoing effort as long as new versions of ERL are to be published.

CONCLUSIONS

The ERL project at Iowa DOT and Iowa State University aims to develop and maintain a hypertext electronic publication that contains Iowa DOT standard contract documents. Currently, a new ERL is released every six months, and comments have been good. Key factors responsible for the success of the ERL project include:

- Conducting focus group sessions
- Developing the rapid prototype
- Minimizing the ongoing task of maintenance
- Development of a good procedure manual

While the ERL brings convenience and efficiency to the target users, implementation and maintenance of the ERL had been a challenging experience for the project members. Based on reviews and the experience from implementing and maintaining the ERL, a few tools and methods were created to address the various challenges faced by the ERL project team including automatic hyperlink creation for cross references and improved PDF file link verification.

Preparation (formatting, adding bookmarks and hyperlinks) of HTML files and checking errors in the PDF files are two of the most time consuming works in the ERL maintenance. To improve the work efficiency, scripts are used to automate the preparation and error checking. Documentation of the working procedures was used in the ERL project to share the knowledge. The methods proposed in this report have been used in the ERL project. These methods have been found to significantly help in improving work efficiency and quality.

The ERL project will continue for the foreseeable future. Because of the changes in users' needs, available tools, etc, searching for improvements in the ERL and its development methods will be a continuous work. Possibilities for future improvements include:

- Continue the effort of including more documents and links in the ERL to meet users' needs.

- Collect feedbacks from users.

- Search for the necessity and possibility of using new forms of storage and distribution method.

- Develop the ERL for handheld devices.

- Incorporate video and audio files to illustrate testing methods, and other concepts in the ERL.

ACKNOWLEGEMENTS

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An Investigation of Object-Oriented Specifications for Iowa DOT and Urban Standards

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ABSTRACT

Currently, individuals including designers, contractors, and owners learn about the project requirements by studying a combination of paper and electronic copies of the construction documents including the drawings, specifications (standard and supplemental), road standards, design criteria, contracts, addenda, and change orders. This can be a tedious process since one needs to go back and forth between the various documents (paper or electronic) to obtain the complete picture. There are also special provisions as well as standard specifications referenced in the contract documents that need to be understood. As transportation projects become more complex to design and build with fewer resources available, it is important to take advantage of appropriate innovative technologies that make it easier for designers and construction personnel to grasp the project requirements in a quick and efficient manner. This could ultimately reduce the chance of error, improve quality, decrease rework, and shorten the project duration. The use of object-oriented computer aided design (OO-CAD) to graphically portray information is one such technology. OO-CAD allows users to point and click on portions of an object-oriented drawing that are then linked to relevant databases of information (e.g., specifications, procurement status, and shop drawings). This paper presents the research method and primary results of an investigation of the use of OO-CAD specifically for providing more direct specification linkages to standard details used for designing and constructing projects. The study involves integrating both the Iowa Department of Transportation and urban specifications into one graphical database using OO-CAD for use by designers and construction personnel involved with any type of transportation project in Iowa. This would centralize the specification process, thus, saving on resources and providing a visual format for accessing specifications. OO-CAD would also make it easier for designers and contractors to more readily understand project requirements.

Key words: animation—drawing—object-oriented computer-aided design—specifications—three-dimensional image

INTRODUCTION

Today, members of the project team including designers, contractors, and owners do not have enough time to fully understand the project details and are in need of a tool to help them become more efficient. A graphical navigation approach can solve this problem. Currently, individuals learn about the project requirements by studying a combination of paper and electronic copies of the construction documents including the drawings, specifications (standard and supplemental), road standards, design criteria, contracts, addenda, and change orders. This can be a tedious process since one needs to go back and forth between the various documents (paper or electronic) to obtain the complete picture. There are also special provisions as well as standard specifications referenced in the contract documents that need to be understood. As transportation projects become more complex to design and build with fewer resources available, it is important to take advantage of appropriate innovative technologies making it easier for designers and construction personnel to grasp the project requirements in a quick and efficient manner. This could ultimately reduce the chance of error, improve quality, decrease rework, and shorten the project duration. The use of object-oriented computer-aided design (OO-CAD) to graphically portray information is one such technology. OO-CAD allows users to point and click on portions of an object-oriented drawing that are then linked to relevant databases of information (e.g., specifications, procurement status, and shop drawings). This study investigates the development of an OO-CAD specification system for providing more direct specification linkages to standard details used for designing and constructing projects.

LITERATURE SEARCH

OO-CAD systems have long been touted as the way forward for more intelligent CAD systems. In an object oriented design, the object contains all of the data necessary to fully describe that object. The idea behind this concept is that the design involves several objects that have information associated with them. When the user clicks on an object, information pertaining to that object appears. This information can be fixed or dynamic in a sense that the information about that object changes with time. Only a few OO-CAD systems have been previously developed, and most of these are on high-end UNIX machines that are not affordable by the majority of practitioners in the construction industry. This appears to be changing with the introduction of OO components in many of the popular systems (e.g., AutoCAD with objects defined in C++) *(1)*. Major CAD packages such as AutoCAD and MicroStation are presently working towards greater interoperability in the Windows environment by supporting such emerging tools as aecXML (Architect/Engineer/Contractor eXtensible Markup Language) *(2)*. Other object oriented animation programs such as Flash and Shockwave are also available.

Two efforts are currently underway to make it easier to find required specifications and provide unification of the many different city and county specifications that are used in Iowa. In order to make it easier for design and field personnel to locate Iowa DOT specifications *(3)*, an Electronic Reference Library (ERL) *(4)* has been created. This is a hyperlinked electronic version of the standard specifications, supplemental specifications, Material Internal Memoranda, Standard Road Plans, and Construction Manual that has been copied to a CD ROM format for statewide distribution (thousands of copies have already been distributed). State agencies, local agencies, and construction contractors using the Iowa DOT specifications can quickly and efficiently identify relevant specifications using the key word search capability. New CDs are released on a six-month basis reflecting updates to the specifications. It is important for projects under design to have the most current information. Thus far, users (i.e., designers, contractors, inspectors, field engineers, suppliers, FHWA, counties, cities, and other state DOTs) have found electronic specifications to be useful for quickly locating information.

Continued attention has been made to improve design and construction coordination between project team members. Currently, there are two concerns facing the transportation design and construction organization. The first one is resource availability, which is becoming more constrained due to budgetary reductions. This is particularly true in the area of maintaining specifications and design standards where updates are disseminated every six months at the Iowa DOT. Local specifications are not typically updated as frequently. With the move toward an integrated specification approach between the state, cities, and counties there is an opportunity to make more efficient use of resources in this area. The other is documentation readability. Individuals learn about the project requirements by studying a combination of paper and electronic copies of the construction documents. These documents include, for example, the drawings, specifications (standard and supplemental), road standards, contracts, addenda, and change orders. This can be a tedious process since one needs to go back and forth between the various documents (paper or electronic) to obtain the complete picture. Also, it can be quite time consuming for new designers to learn design standards and specifications. Therefore, the need exists to use information technology to develop a new specification system that can help designers and contractors and improve the project performance.

STUDY OBJECTIVES

The purpose of this study is to develop a prototype model of object-oriented specifications. This model will be used to test the feasibility of the concept above and assess its impact on the design and construction of transportation projects. Both Iowa DOT and urban specifications *(5)* are included in an object-oriented format to demonstrate the concept. The end product will be a graphical or visual front-end system for the ERL. Full-scale development and maintenance issues will also be addressed as part of this project.

STUDY METHODOLOGY

The methodology involved includes several steps:

1. Develop an understanding of the user requirements through a series of meetings and face-to-face interviews with designers and field personnel.
2. Coordinate selection of formatting issues for plans and specifications (CAD, paper based, HTML, PDF, etc.) with Iowa DOT and SUDAS (Statewide Urban Design and Specifications).
3. Research various OO-CAD software packages and make a selection.
4. Develop a prototype model in object-oriented format and link it to certain provisions of appropriate specifications.
5. Obtain user feedback and modify the model.
6. Conduct a study of the operational feasibility of this concept. The operational feasibility would investigate how such an approach would affect the Iowa DOT’s standard operating procedures.

INVESTIGATION AND PRIMARY RESULTS

At this point in time, the study has not been completed yet. However, much progress has been made and the investigation shows good indications of usefulness of such a system. The following discussion presents a few key perspectives of the investigation.

Prototype

Several types of images have been used to illustrate the concept and one prototype has also been developed during the investigation. Figure 1 shows how this concept works. This figure depicts a typical cross-section of a roadway. The user is able to point and click on any aspect of the design; a popup menu appears that directs the user to the database containing the relevant specification information for that portion of the design. In Iowa different specifications are used depending on the project type and location. Iowa DOT specifications are used on state, federal, and county highway projects. Urban specifications are used primarily on city projects. These specifications might be different depending on the location in Iowa. For example, Loess soil conditions in western Iowa might require different design requirements than in other parts of Iowa.

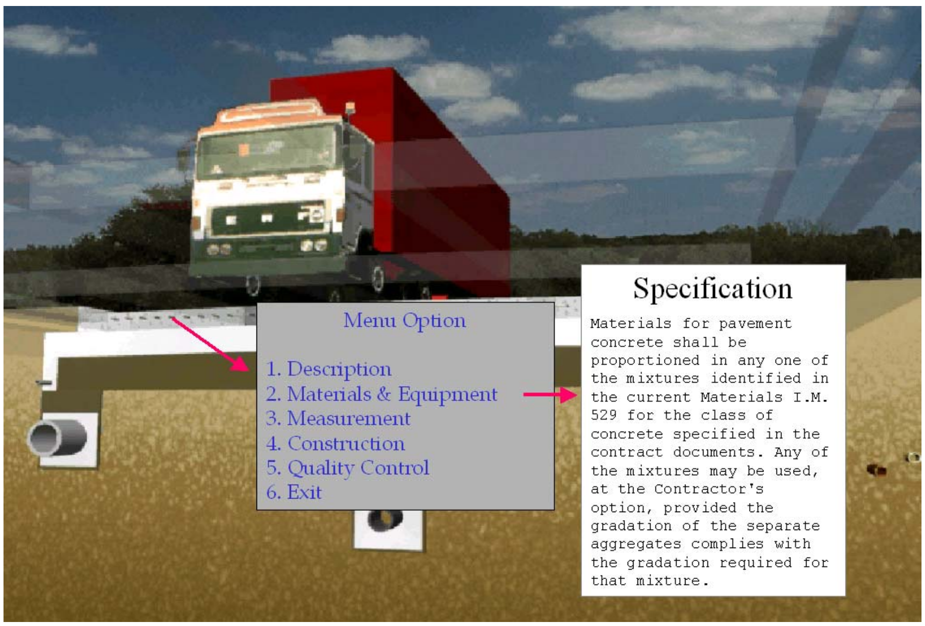

FIGURE 1. Object-oriented Specification Illustration

Figure 2 illustrates the major features of the final product. The system should be constructed by building a repository of high quality, reusable components of specifications, which project engineers can combine in various ways to produce new reusable components at higher and higher levels of abstraction. Therefore, a set of reusable components of specifications are constructed, each of which is built from a few fundamental sub-elements. Specification objects in transportation systems - pavements, base and sub-bases, curbs, traffic pole - store non-graphical data in a logical structure together with the standard graphics that, in three dimensions, are in object-oriented CAD formatted files and are carefully structured and managed. The system accommodates multiple specifications (Iowa DOT and Urban) in one system. The common and different components should be clearly indicated and the relationships should be well established. The system uses an interactive image (middle of figure 2) instead of a movie or picture. The non-graphical data is easy cut and pasted.

FIGURE 2. Object-oriented System Features

Figure 3 shows an example of a system element–traffic signal poles. The signal pole was constructed by components in three dimensions using Macromedia Director. The image can be rotated, panned, and zoomed. Each component is associated with certain specification information, which is stored in a central database. Users can easily locate information and project requirements from the graphics as shown in the figure.

Mapping

To manage the specification data and the graphics more meaningfully, a mapping technology has been employed in the study. Figure 4 illustrates a mapping flowchart of a traffic signal pole. Mapping means that all the information related to a major component in the specification is pulled out and reorganized into different layers and matched to the graphics. In the figure, the traffic signal poles are divided into traffic signal mast arm poles (TSMAP), pedestals, and signs. The TSMAP, for example, is further split into standards and processes, materials, strength, welding, testing and appurtenances. The details of the specification associated with each of these items will be linked and stored in an external base. The mapping structure functions as a bridge between the graphics and the database.

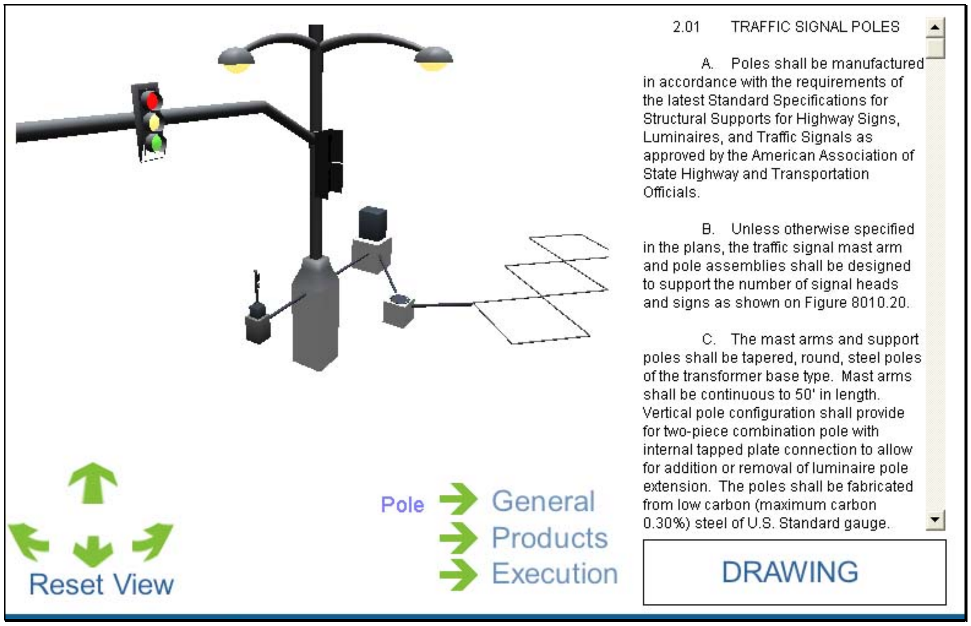

FIGURE 3. Object-Oriented Traffic Signal Poles

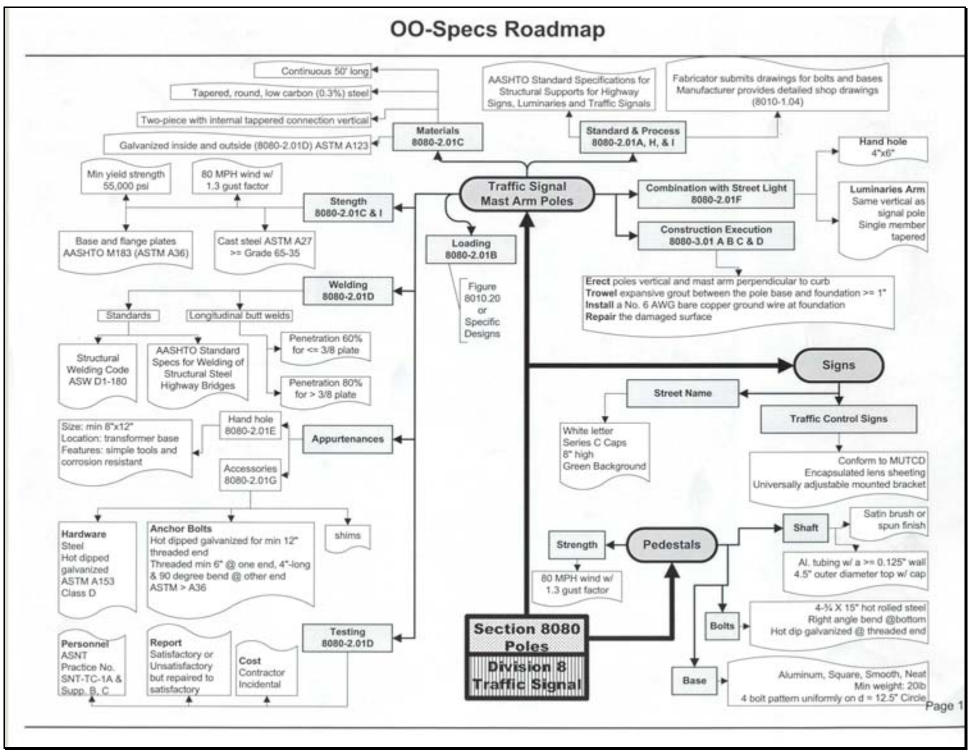

FIGURE 4. SUDAS Specification "Roadmap" (Signal Poles)

User Requirements

The concept and prototype have been presented to the specification committee (designer, contractor, contract engineers, etc.) to gather some end-user requirements and additional meetings will occur later. The primary end-user requirements through this channel are implemented to the prototype model and summarized as follows.

- Design a standard picture/scene to allow the user to get specification information by clicking on the various objects.
- Consider the issues related to updating standards or specifications.
- Use standard features instead of unique features of project.
- The graphics must cover all pertinent specifications.
- The system should facilitate quick decision-making and help new engineers learn.
- The system should be useful to anybody in the civil and construction fields.
- Integrate the various codes and plans.
- Provide a search function for end users.

POTENTIAL BENEFITS

Some feedback has been obtained from designers, contractors, and other engineers. Several benefits are anticipated by developing and applying such an object-oriented specification system.

1. It will be easier for designers, field personnel, contractors, suppliers, and manufacturers to find the specifications relevant for a specific portion of the design. This should improve the efficiency of preparing the design documents and interpreting them in the field.
2. This graphical approach will make it easier for new designers to learn about standards and specifications more quickly.
3. This approach will make it easier to combine both the Iowa DOT and urban specifications as is currently being considered by the SUDAS committee. The specification updating function might ultimately become more centralized, thus freeing up resources at the city and county level to help maintain the system.
4. Additionally, it will help Iowa transportation agencies in maintaining a cutting-edge presence in information technologies since this may be a new paradigm in which projects will be constructed in the future.

However, further testing should be done before drawing the final conclusions.

CONCLUSIONS

This study involves integrating both the Iowa DOT and Urban specifications into one graphical database using OO-CAD for use by designers and construction personnel involved with any type of transportation project in Iowa. This would centralize the specification process, thus, saving on resources and provide a visual format for accessing the specifications. It would also make it easier for designers and contractors to more readily understand project requirements. However, because the study is still ongoing, the formal conclusion should be drawn after the study is completed.

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Investigation into Pavement Curing Materials, Application Techniques, and Assessment Methods

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ABSTRACT

Different types of curing compounds were applied to lab and field concrete at different times after casting. Two application methods, single and double layer applications, were employed. Concrete properties, such as electrical conductivity, moisture content, sorptivity, degree of hydration, strength, and permeability were evaluated, and the results were employed for assessment of the curing effectiveness. The investigation indicated that properties of the concrete in the near-surface-area of a pavement without any curing compound were considerably different from those of the internal concrete. Proper curing can significantly increase the moisture content and reduce the sorptivity of the concrete in the near-surface-area, thus providing the pavement with uniform concrete properties throughout depth. For given concrete materials and mix proportion, water retention ability of a curing material can be estimated by monitoring conductivity of the surface concrete. Sorptivity test for lab specimens appeared a good assessment method for curing effectiveness, and the measurement captured the subtle changes caused by different curing materials and application techniques. These research results established a baseline for rational assessment of curing compound effectiveness in both lab and field concrete.

Key words: curing compound—electrical conductivity—hydration—pavement—sorptivity

INTRODUCTION

A variety of curing methods and materials are available for concrete pavements, including water spray or fog, wet burlap sheets, plastic sheets, insulating blankets, and liquid-membrane-forming compounds. Burlap or insulating blankets are considered ideal for retaining heat and moisture, but their application is labor intensive and time consuming. In contrast, liquid membrane-forming curing compounds can provide a similar insulation and be applied much more easily. Control of heat and moisture loss by application of a curing compound, especially in hot or cold weather conditions, has aided contractors in enhancing concrete quality, permitting early open of pavements to traffic and extending the available construction season. Curing compounds are economical, easy to apply, and maintenance free (*1*).

Concrete practice has indicated that the performances of curing compounds are closely related to the characteristics of the curing materials, application methods (single- or double-layer spray), and application time (*2*). However, limited research has been done to investigate the effectiveness of different curing compounds and their application techniques. No reliable standard testing method is available to evaluate the effectiveness of curing, especially of the field concrete curing.

RESEARCH OBJECTIVES AND SCOPE

The present investigation was conducted to explore testing methods for measuring the effectiveness in the lab and field as well as to evaluate effectiveness of curing compounds and application techniques for concrete pavements.

The investigation contained two parts: lab study and field application. Three curing compounds with different efficiency indexes were selected and applied to lab and field concrete at different times after casting. Two application methods, single and double layer applications, were employed. Electrical conductivity, moisture content, sorptivity, degree of hydration, strength, maturity, and permeability tests were performed to evaluate the curing effectiveness and concrete properties in the lab and field studies. Some of the results are presented in the following sections. More detailed results can be obtained in separated documents (*3*,*4*).

LAB EXPERIMENT

Curing Materials

The curing compounds used are listed in Table 1, where C95.9 and C89.0 are water-based compounds, while C98.1 is a resin-based curing compound. The Efficiency Indexes of the curing compounds were tested according to the test method: Iowa 901-D (*5*).

TABLE 1. Typical Properties of Selected Curing Compounds

Compound	Color	ASTM Specification	Efficiency Index	Solids Content (%)
C98.1	White	Type 2 Class B	98.1	43.5
C95.9	White	Type 2 Class A	95.9	29.2
C89.0	White	Type 2 Class A	89.0	17.1

Curing Methods

Three reference curing conditions (without curing compound) were selected to simulate the worst, the best, and hot weather curing conditions of field concrete, and they are: air curing (Reference 1: T=77 ^{o}F, and R.H.= 38%), wet curing (Reference 2: covered with burlap and stored in a fog room, T=73 ^{o}F, and R.H.≥ 95%), and oven curing (Reference 3: T=100 ^{o}F at daytime and 77 ^{o}F at night time, and R.H.= 35%), respectively. Three cases were also designed for specimens with different applications of the curing compounds, and they are as follows:

Case 1 (Air Curing + Single-Layer Curing Compound, or Air-SL): In this case, a selected curing compound was sprayed on specimens at 15, 30, and 60 minutes after casting. The specimens were cured in the air (T=77 ^{o}F, and R.H. = 38%) before and after application of curing compound until testing.

Case 2 (Oven Curing + Single-Layer Curing Compound, or Oven-SL): In this case, a selected curing compound was sprayed on specimens at 15, 30, and 60 minutes after casting. The specimens were cured in the oven (as described in Reference 3) before and after application of curing compound until testing.

Case 3 (Oven Curing + Double-Layer Curing Compound, or Oven-DL): In this case, two layers of a curing compound were applied onto the surface of the specimens. The first layer was sprayed at 15, 30, and 60 minutes after casting. The second layer of the same curing compound was spayed 5 minutes after the first application of the curing compound. Samples were cured in the oven (as described as Reference 3 curing condition) before and after application of curing compound until testing.

Specimens

Two types of mortar specimens were prepared: one was 2"x2"x4" prisms and the other was 10" (length) x 5" (width) x 4" (depth) slabs. The mortar was made of Type I cement and river sand with fineness modulus of 2.94. The sand to cement ratio was 2.75, and w/c was 0.42.
The prism specimens were used for moisture content and sorptivity tests. The slab specimens were used for temperature monitoring, conductivity tests, and three 2"-cores were drilled from the slabs for compression tests. Small cement paste slabs, 10" (length) x 5" (width) x 4"(depth), were prepared for testing degree of cement hydration tests. The paste has the same cement as that used for mortar with a w/c of 0.28. At the 15, 30, and 60 minutes after casting of the specimens, selected curing compounds were uniformly sprayed on the surface of the specimens until the prescribed rate (3.3 m^2/liter) was reached.

Test Methods

Moisture Content: Moisture content was measured from a set of three mortar prisms. In the tests, a mortar prism was fractured along designed notches that divided the prism into three equal pieces: top, middle, and bottom. Each piece was weighed initially (W_i) and then put in an oven at a 105^{o}C for 48 hours to obtain a constant weight (W). The moisture content (MC) is given by MC = $[(W_i - W)/W]*100$.

Electrical Conductivity: After a mortar slab was cast, a pair of copper plates, 0.75" (height) x2" (width) x 0.125" (thickness) and 3" apart, were vertically inserted into the sample, approximately dividing the sample equally in length. Solomat MPM 2000 conductivity meter was used to measure the resistivity between the two copper plates.

Sorptivity: Sorptivity of mortar prisms was measured at 3 days. The prisms were cut evenly into three pieces. The lateral surfaces of each piece were sealed with five-minute epoxy, and the top surface was covered with plastic. After weighed, the bottom surface of the samples was immersed into tap water, with a maximum immersed depth of 3 mm. The samples were then weighed again after 5, 10, 20, 30, 60 minutes, and 6 hours of immersion. The test was finished when the slope of mass gain per unit area versus square root of testing time was constant. This constant slope is the sorptivity coefficient.

***Degree of Hydration**:* A 2” diameter core was taken from a paste sample. Three 0.5” thick pieces were cut from the top, middle, and bottom of the core. Each piece was crushed and sieved with a #16 sieve. The samples were heated at 105°C for 18 hours and weighed (W_{105}). They were then heated at 1000°C for another hour and weighed again (W_{1000}). The degree of cement hydration (α) is expressed as: $\alpha = [(W_{105} - W_{1000}) / (0.24 * W_{1000})] * 100\%$. (*6*)

Test Results

Moisture Content

Curing compound is considered as materials that can form a coating on the surface of newly placed concrete and prevent a quick loss of moisture in the concrete. Table 2 presents one-day moisture content of mortar specimens tested.

TABLE 2. One-day Moisture Content of Mortar Specimens (%)

	(a) Reference Specimens			(b) Case 2: Oven-SL, Measured at Top Layer			
Layer	**Oven Curing**	**Air Curing**	**Wet Curing**	**Spray Time**	**C89.0**	**C95.9**	**C98.1**
Top	4.77	6.26	8.31	15 min	7.35	7.35	7.26
Middle	5.73	7.04	8.31	30 min	7.20	7.29	7.24
Bottom	5.74	6.88	8.00	60 min	6.96	6.82	6.92

As shown in Table 2(a), for reference specimens (without any curing compound), the top-layer, the one-day moisture content of the mortar slab was 4.77 percent for the oven-cured specimen, 6.26 percent for the air-cured specimen, and 8.31 percent for wet-cured specimen. At hot weather (oven curing) condition, the difference in moisture contents between top- and middle-layer of the mortar (5.73 percent) was approximate 17 percent. At mild weather (air curing) condition, the difference in moisture contents between top- and middle-layer of the mortar reduced to 11 percent. Wet curing provided the mortar with uniform moisture content through depth. The slightly low moisture content in the bottom-layer mortar was found in all specimens, and it might be attributed by aggregate settlement and bleeding water rise in the mortar.

Table 2(b) indicates that for specimens sprayed with a single-layer curing compound at a hot weather condition (oven curing), regardless the type of a curing compound, the one-day moisture content in the top-layer of the mortar slabs was approximate 7.3 percent, compared with 4.77 percent in the reference slab without curing compound. With curing compound, the top-layer moisture content in the slabs was very close to the middle layer moisture content of the slab although it was still lower than the moisture content of the wet cured specimens. As the spraying time of a curing compound was delayed, the moisture content of the top-layer mortar slightly reduced.

Electrical Conductivity

As cement hydration progresses and free water is lost, the numbers and/or mobility of ions in the concrete pore solutions are changed. This in turn causes a change in the electrical conductivity of the concrete (*7*). For given materials and mix design, the conductivity of concrete depends primarily on the cement hydration process and moisture content in the concrete, which is related to the curing conditions.

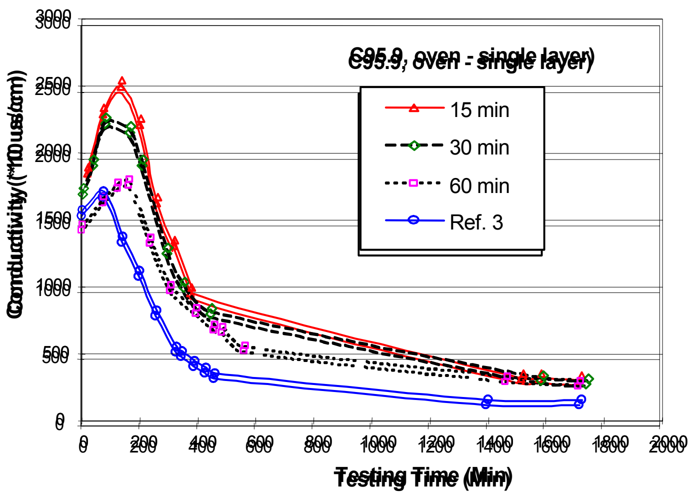


FIGURE 1. Conductivity of Mortar Specimens

Figure 1 illustrates conductivity of mortar specimens with single layer curing compound of C95.9 sprayed at 15, 30, 60 minutes after casting. As shown in the figure, specimen Reference 3, oven cured without curing compound, had the lowest conductivity values, possibly due to fast water loss. When a curing compound was applied, conductivity of the mortar increased with early spray time. It was also observed that wet-cured specimen had the highest conductivity value among all specimens. Specimens with curing compounds all fell within the boundaries formed by Reference 2 (wet curing) and Reference 3 (oven curing). The conductivity of samples sprayed with high efficiency-index curing compounds tended close to that of wet curing specimen (*3*).

Statistic analysis of the test data has demonstrated a good relationship between conductive and moisture content:

$$\text{Conductivity} = 1/[A-B \log (MC)] \tag{1}$$

Where A and B are constants related to concrete materials and mix design, which are 18.65 and 7.68 respectively based on the present research, and MC is moisture content of the specimens tested. The reliability coefficient of the equation, R^2, is 0.79, indicating a reasonably reliable relationship. According to this relationship, the electrical conductivity measure can be used to estimate moisture retaining ability of field concrete cured with different materials and application methods.

Sorptivity

Concrete sorptivity is closely related to its pore structure, which in turn influence concrete durability. Absorption test is considered as one of the most reliable methods for assessing curing effects of concrete (*8*). Poor curing of the concrete may result in high sorptivity, and this effect is more pronounced in the near-surface layer (1–2 inches from the surface) of concrete (*9*).

Table 3 demonstrates sorptivity values of mortar specimens at different depths, with different curing compound materials, and different application methods. As observed in Table 3(a) and (b), sorptivity of the near surface layer (top layer) of the specimens is generally higher than that of the middle and bottom layer of the specimens regardless whether or not a curing compound is applied. This may be also attributed to the aggregate settlement. In freshly cast concrete, aggregate settles due to its gravity and bleeding water raises, thus increasing porosity of top-layer concrete. As a result, sorptivity of the top-layer concrete is higher than that of bottom-layer concrete.

TABLE 3. Three-Day Sorptivity of Mortar Specimens ($g/m^2/min^{1/2}$)

(a) Reference Specimens				(b) Effect of Curing Materials: Case 2 (oven-SL)			
Layer	Oven Curing	Air Curing	Wet Curing	Layer	C89.0	C95.9	C98.1
Top	4.23	3.64	0.41	Top	0.63	0.44	0.42
Middle	0.98	1.94	0.36	Middle	0.38	0.3	0.33
Bottom	0.15	1.06	0.17	Bottom	0.11	0.13	0.13
(c) Effect of Application Methods (top layer)				**(d) Effect of Spray Time (case 2, top layer)**			
Case	C89.0	C95.9	C98.1	Spray time (after casting)	C89.0	C95.9	C98.1
Case 1(room-SL)	0.76	0.7	0.6	15 min	0.63	0.44	0.42
Case 2 (oven-SL)	0.63	0.44	0.42	30 min	0.52	0.45	0.46
Case 3 (oven-DL)	0.5	0.34		60 min	0.59	0.56	0.49

At hot weather (oven curing) condition, the sorptivity of the near surface layer mortar without a curing compound is the highest (4.23 $g/m^2/min^{1/2}$) among all specimens tested; while the sorptivity of the specimens with curing compounds are comparable with that of specimen cured at wet condition (0.41 $g/m^2/min^{1/2}$). For the specimens at hot weather (oven curing) condition without curing compound, the difference in the sorptivity values between the top-layer and the middle/bottom-layer mortar is large (over 3 times); while for the specimens with curing compound, the difference is much small (1.3-2 times). These indicate that use of curing compounds is a very effective way to improve concrete microstructure and reduce concrete porosity or permeability. Table 3(b) also suggests that the higher efficient-index curing compound generally provides its mortar specimens with lower sorptivity. Table 3(c) illustrates that sorptivity value reduces approximate 25 percent when double-layer curing compound is applied and compared with single-layer curing compound application. High temperature (oven) curing provides the mortar specimens with lower sorptivity, when compared with room temperature (air) curing. As shown in Table 3(d), sorptivity measurement can also well reveal effect of application time of curing compound on mortar microstructure. At the mild weather condition (air curing), curing compound spraying at 30 minutes after casting is the optimal compared with spraying at 15 or 60 minutes. At hot weather (oven) condition, the research implies that spraying curing compound at 15 minutes after casting is the optimal (*3*). The optimal time for spraying a curing compound is right after disappearing bleeding water but before drying of the concrete. Sorptivity measurement appears a rational test method for evaluating concrete curing effectiveness among other tests employed in this research.

Degree of Hydration

Table 4 demonstrates degree of cement hydration of paste specimens with depth. Typically, the top layer of a slab specimen (with or without curing compound) has the lowest and the middle layer has the highest degree of hydration. Application of curing compound generally increases degree of cement hydration, especially in the top layer of the specimens. The specimens at hot weather condition (oven) and sprayed with C89.0, C95.9, and C98.1 had degree of hydration of 46.7 percent, 52.3 percent, and 51.6 percent, respectively, compared with 43.6 percent of the oven specimen without curing compound. Application time and double layer spray seemed not having clear effects on degree of hydration of the specimens.

TABLE 4. Three-Day Degree of Hydration of Paste Specimens (%)

(a) Reference Specimens				(b) Case 2: Oven-SL, Spray Time: 15 min.			
Layer	Oven Curing	Air Curing	Wet Curing	Layer	C89.0	C95.9	C98.1
Top	43.62	41.00	47.11	Top	46.72	52.48	51.64
Middle	55.17	47.87	51.68	Middle	55.35	57.48	54.96
Bottom	55.45	46.63	48.47	Bottom	52.05	54.33	51.7

A statistical analysis of all test data was performed to investigate whether there was any relationship among the parameters measured. The analysis indicates that the sorptivity is closely related to MC and degree of hydration (α), and the relationship is as follows:

$$\text{Sorptivity} = 29.11 - 3.56(MC) - 0.53\alpha + 0.07(MC) * \alpha \quad (2)$$

The reliability coefficient of Equation (2) is $R^2 = 0.79$, or $|R| = 0.89$. According to this equation, sorptivity of concrete may be estimated from simple tests of cement hydration and moisture content, which is in turn related to conductivity of concrete.

FIELD INVESTIGATION

Description of Test Sections

The field tests were applied on highway US 65 in Polk County. The materials and proportion of the pavement concrete are as follows: limestone : limestone chips : sand : Type I/II cement : Class C fly ash : water = 3:1:(2.43):1:(0.18):(0.47). The test sections included the following:

- 600 linear feet of curing compound C98.1 single application
- 600 linear feet of curing compound C95.5 single application
- 600 linear feet of curing compound C95.5 double application
- 600 linear feet of curing compound C89.0 double application
- 600 linear feet of wet curing
- 20 linear feet with no curing materials applied

All sections were placed in the same day. For analysis purposes the 600-foot sections were subdivided into three 200-foot subsections. Each subsection had a test station; and thus a total of 16 test stations were established for this investigation. Each test station was in the middle length of a subsection and two feet from the edge of the pavement slab.

Data Collection Methods

The data collected included the concrete properties (such as temperature, moisture content, conductivity, and permeability) and the environmental conditions (such as air temperature, wind speed, relative humidity, and cloud condition). The measurements were taken every two hours from the morning to the night (about 8:00 a.m. to 9:00 p.m.), and the gathering of the measurements lasted seven days.

A thermocouple was attached to a wood dowel to monitor the temperature of concrete at the top (1 inch below top surface), mid-depth, and bottom (1 inch above the base) of a pavement slab. A pair of copper plates, 1” (height) x 4” (width) x 1/8” (thickness), were vertically inserted to the top and mid-depth of the slab to measure the concrete conductivity (4). Three 2" and 4" diameter cores were taken from each subsection for sorptivity and rapid chloride permeability tests. Since sorptivity test data from the field specimens did not show a linear relation of absorbed water with testing time, sorptivity test results are not presented in the paper. The unexpected test data may be caused by non-uniform moisture distribution in the samples.

Field Test Results

Maturity of Field Concrete

Maturity of concrete was calculated from the concrete temperature monitored. Figure 2 illustrates the differences in maturity of pavements cured with different curing compounds and application methods. Each data point represents the average value of three stations cured with the same method.

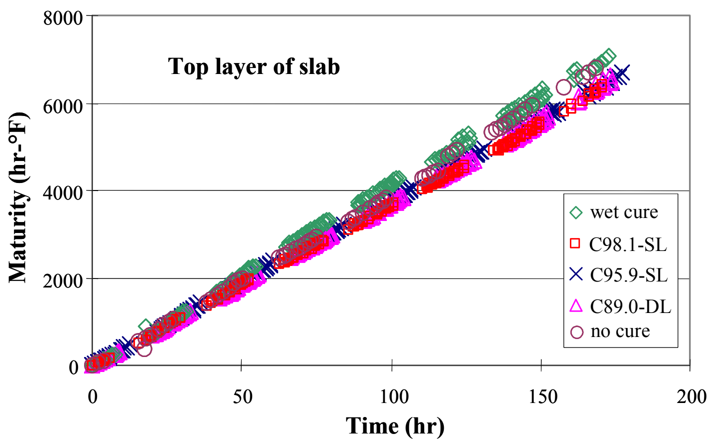
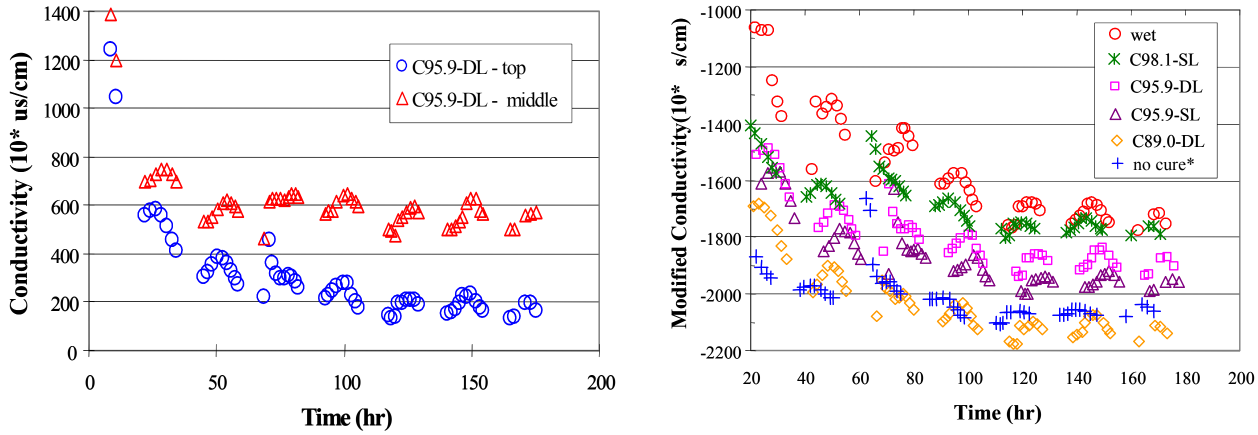


FIGURE 2. Effect of Curing on Maturity of Concrete

The figure indicates that pavements sprayed with different curing compounds had little differences in their maturity values. The top-layer concrete of the pavement with wet curing or without curing had slightly higher maturity values than the pavements with the curing compounds. The high humidity might contribute to the higher maturity of the wet-cured pavement, while the high heat absorbed by the concrete from Sunlight might contribute to the higher maturity of the pavement with no cure. It was also observed that had less difference existed in the middle- or bottom-layer concrete cured with different methods.

Conductivity of Field Concrete

As mentioned before, for a given material, the electrical conductivity depends primarily on the cement hydration process and the moisture content inside concrete. The lab study on mortar specimens curing at different conditions has shown a good statistical relationship between the electrical conductivity and moisture content. Figure 3 demonstrates some conductivity test results of the field concrete.



(a) Effect of Depth

(b) Effects of Curing Methods (Top)

FIGURE 3. Conductivity of Field Concrete

As observed in Figure 3(a), conductivity of the top-layer concrete sprayed with curing compound C95.9 and double application is much lower than that of the middle-layer concrete. This indicates low moisture content in this top-layer concrete due to its surface moisture loss, and the results are consistent with those of moisture contest tests performed at lab. Other conductivity measurement for concrete cured with different curing methods had the same trend. Figure 3(b) demonstrates effects of curing materials and application methods on concrete conductivity. The researchers found that the initial conductivity of field concrete was very sensitive to the weather condition under which paving was performed. To overcome this influence, modified conductivity, the difference between measured conductivity and the initial conductivity of the concrete, was employed. According to Figure 3(b), the wet curing is the best curing method for water retention in concrete, followed by the C98.1-single layer, C95.9-double layer, C95.9-single layer, and C89.0-double layer applications. This trend is consistent with the lab test results.

Permeability of Field Concrete

Permeability of concrete is often considered as the most important factor that influences concrete durability because it controls the entry rate of the moisture flow that may contain aggressive chemicals. In this investigation, the cores were drilled from three selected stations at the seventh day after paving. Rapid chloride permeability tests of the field samples were performed according to AASHTO T277, and the results (average of three tested samples) are presented in Table 5.

TABLE 5. Effects of Curing Methods on Concrete Permeability

Specimen Location	Rapid Chloride Permeability (Coulombs)		
	Wet Curing	**C95.9-Double Layer**	**No Curing**
Top	3084	3084	3289
Middle	2448	2214	2288
Bottom	1479	1488	1500

Table 5 indicates that regardless curing conditions, rapid chloride permeability of concrete decreases with depth. The difference in the chloride permeability values among layers might result from different curing condition and/or from the aggregate settlement in the concrete. Note that there is no significant difference in rapid chloride permeability values among the concretes cured with different methods. A better curing method (wet curing or with curing compound) may reduce but not eliminate the difference between top-layer and internal concrete.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations can be made based on the presented study:

- Regardless whether or not a curing compound is applied, properties of the near-surface-area concrete, such as moisture content, permeability, degree of hydration, and sorptivity, differ from those of the internal concrete. The differences may be caused by surface moisture loss and/or aggregate settlement.
- Application of a curing compound significantly increases moisture content and decreases sorptivity of the near-surface-area concrete, thus reducing the differences in concrete properties between the near-surface-area concrete and internal concrete.
- The non-destructive conductivity test method showed a close relation with moisture content of the concrete in lab study. For given concrete materials and mix proportion, water retention ability of a curing material may be estimated by monitoring conductivity of the near-surface-area concrete. Field conductivity test results showed a similar trend to the lab test results of concrete with different curing materials and application methods. This test method may further modified for future evaluation of curing effectiveness in both laboratory and field tests.
- Of all the test methods applied in lab, the sorptivity test is the most sensitive one to provide a good indication for the subtle changes in the properties related to microstructure of the near-surface-area concrete caused by different curing materials and application procedures. Sorptivity measurements of the near-surface-area concrete have demonstrated a close relation with moisture content and degree of hydration. However, when the method is used for testing field sample, special care of the moisture balance in the samples is needed.
- Maturity measurements in the near-surface-area concrete did not provide good indication for the subtle changes provided by different curing conditions.

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Accomplishments of an Innovative Statewide Van Lease and Purchase Program

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ABSTRACT

Tennessee Vans is an innovative statewide vehicle procurement service that provides vans for lease and/or purchase to commuter groups, employers, public agencies and private non-profit community organizations. In exchange for access to vehicles and affordable financing provided by Tennessee Vans, the program participants agree to provide safe and reliable transportation services to meet identified needs and to pay for the Tennessee Vans vehicles.

The cost recovery strategy for Tennessee Vans is a critical approach to maintaining the viability and longevity of the program. The initial seed grants are provided by local, state, and federal governments with the stipulation and expectation that Tennessee Vans will recover vehicle and administrative costs to the highest extent possible. Tennessee Vans strives to constrain administrative expenses, minimize financial loses, and maximize vehicle cost recovery. Revenues received from program participants are used to purchase replacement vehicles in the lease program and to procure additional vehicles for the purchase program. With no driver costs and other operating costs largely absorbed by the agencies, Tennessee Vans has a distinct financial advantage.

Tennessee Vans has experienced a steady rate of growth in funding and service development since its implementation in 1990. Financial resources provided during this time have enabled the program to place over 500 vehicles with 300 different organizations. Currently over two million annual trips are provided by organizations utilizing Tennessee Vans with 354 vans being in active service, 115 in the lease program and 239 in the purchase program.

Key words: public transit services—vehicle lease and purchase program

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SERVICE DELIVERY APPROACH

The service delivery approach used by Tennessee Vans has evolved over the past ten years in response to growing demands for transportation resources among diverse population groups throughout Tennessee. Van transportation services have been used as an energy conservation technique, a measure to help alleviate traffic congestion and air pollution, and an economic resource to assist persons with access to employment and job training opportunities. A variety of van transportation programs have been implemented, including employer sponsored programs, independent owner operators, private third party operations, and public agency programs. Tennessee Vans is an evolving service delivery model designed to meet the changing nature of mobility needs in Tennessee. *(1)(2)(3)*.

The Tennessee Vans program was initiated on February 1, 1990, as a continuation of the Tennessee Department of Transportation's (TDOT) supportive role in the development of van transportation services in Tennessee. The Tennessee Vans program is operated by the University of Tennessee Center for Transportation Research and provides vehicles for lease and/or purchase by commuter groups, employers, private agencies, and public and non-profit community organizations. Tennessee Vans is a provider of vehicle resources to program participants, who are the primary mobility service designers. The program participants design the travel routes, operating schedules, and financial options for those they serve. The only requirement from Tennessee Vans is that the program participant meets the basic requirements for vehicle repayment and travel safety. Tennessee Vans uses grant funds provided by federal, state, and local sources to purchase vehicles for use by program participants. The vehicle costs and associated administrative expenses for Tennessee Vans is recovered from program participants through fees charged for the lease or purchase of vehicles. These generated funds are in turn used to purchase vehicles to replace older vehicles or add more vehicles to the fleet. During the past ten years, the Tennessee Vans fleet has grown to over 500 vehicles across the state (Figure 1). This paper presents a summary of the ten years of accomplishments of Tennessee Vans.

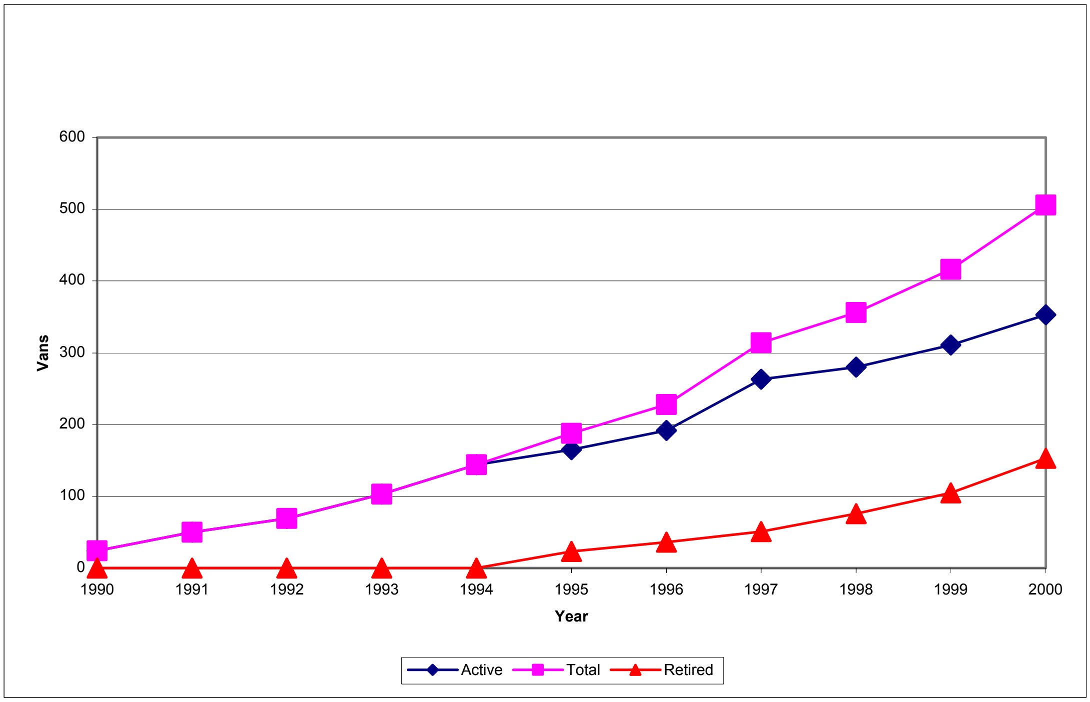

FIGURE 1. Tennessee Vans Vehicles Per Year

BASIC PROGRAM SERVICES

Qualified program participants can lease and purchase Tennessee Vans vehicles. Three basic service programs are available: the Employee Vanpool Lease Program, the Agency Vehicle Lease Program, and the Agency Vehicle Purchase Program.

The Employee Vanpool Lease Program provides vehicles, insurance, maintenance, and fleet management assistance to commuter groups who want to travel to and from work in a vanpool. Minivans and fifteen passenger vans are provided to groups of commuters who wish to ride together and share the monthly costs of operating the vanpool. The monthly fee covers the vehicle costs, maintenance, gasoline, insurance, and fleet management expenses. A member of the commuter group volunteers to drive the van, collect monthly rider fares, and keep the vehicle properly serviced. The typical vanpool monthly lease fee for a current model fifteen passenger van traveling seventy miles (70) round trip daily is $780.00. Each member of the group pays a portion of the monthly fee (e.g., $65.00 each for a group of twelve paying passengers).

The Agency Vehicle Lease Program provides the opportunity for public and private organizations to provide transportation through an affordable vehicle lease plan. Transportation services include transporting persons to and from work, job training sites, work-trip related events, and other activities which facilitate the mobility and meet the travel needs of persons served by the organization. Qualified agencies pay monthly vehicle lease fees on a fixed cost plus mileage basis. The agencies provide their own insurance at program specific coverage levels. The lease costs include the cost of the vehicle, vehicle maintenance, and fleet management expenses. A typical agency monthly lease fee for a current model fifteen passenger van is $450.00 fixed cost per month plus $.10 per mile.

The Agency Vehicle Purchase Program provides the opportunity for program participants to purchase vehicles for transportation purposes through an affordable financing plan. Participants include public and private non-profit organizations that currently provide or would like to provide transportation services. The transportation services provided by the agency are the same as for agencies who lease Tennessee Vans vehicles. Vans are assigned to participating organizations through simple purchase contracts. The participating organization agrees to pay monthly fees until the vehicle contract is paid in full. Upon payment of the vehicle contract cost, the vehicle title is fully transferred to the participating organization. Under the vehicle purchase program, the program participant provides the vehicle insurance, maintenance services, and qualified drivers. The typical agency vehicle purchase contract cost for a current model fifteen passenger van is $25,000. The contract cost is amortized over 72 months with monthly payments of approximately $348.00.

PROGRAM FUNDING

Program funds to administer and operate Tennessee Vans vehicles have been provided by TDOT, local agencies (e.g., Metropolitan Planning Organizations), and program generated revenue. Tennessee Vans strives to be financially self-sufficient through its cost recovery program, reinvestment of program generated revenues, and continuous solicitation of capital resources to meet increasing service demand.

The funds used for capitalizing Tennessee Vans during its implementation have been provided by several sources. Approximately 15% of the funds have been provided by TDOT in the form of the initial seed grant and periodic supplemental grants. Local MPOs have contributed approximately 42% to support local van services as part of congestion mitigation, air quality and surface transportation programs. The remaining 43% has been received as program generated revenue in the form of service fees received from program participants. The operating cost for the Tennessee Vans program is approximately $250,000 annually.

CURRENT FLEET STATUS

Of the vans operating in Tennessee, 70 percent are included in the purchase program, 10 percent in the commuter vanpool program and 20 percent in the agency vehicle lease program. The lease program is of interest to public transit and work force development organizations that are looking to acquire vehicles to address short term mobility needs or respond to grant programs with uncertain time durations. Program participants come from an array of different organizations as noted in Table 1. The program categories and current participating numbers of organizations in Knoxville were as follows:

1. community economic development organizations – 56 (88 vans)

2. faith based organizations – 75 (68 vans)

3. public/private transit services – 7 (27 vans)

4. workforce development organizations – 14 (40 vans)

5. youth services organization – 49 (72 vans)

6. commuter vanpooling – 35 (37 vans)

TABLE 1. Tennessee Vans Program Participants by Category

Van Programs
1. Community and Economic Development Health Care Facilities Housing Authorities Environmental Groups Community Development City and County Agencies Residential Group Homes Drug Elimination Program
2. Faith Based Organizations
3. Public/Private Transit Providers
4. Work Force Development Educational Facilities Private Industry Councils Supported Employment Programs Job Training/Placement Services Employers Work Release Programs
5. Youth Based Day Care Centers Youth Service Programs
6. Commuter Vanpools

Some examples of agencies participating in the program include the following:

The Knoxville Community Development Corporation is a community economic development organization that uses a program vehicle to meet mobility needs of participants in its community micro-enterprise loan program. A minivan was purchased to transport program staff and participants to community based training events and activities that support the development of small businesses.

The Knox County Community Action Committee is an example of a public transit operation that uses the vans to transport clients. This organization is a demand responsive transit service that transports clients to jobs, employment training, medical appointments, and recreational activities. An example of a private transportation business that uses the vans is Kid Trans Enterprises. Kid Trans Enterprises is a small business formed as part of the Knoxville Transportation Business Development Program and leases several vehicles to transport children to and from daycare centers and after school care facilities, and also provides transportation for field trips.

The Sertoma Center is a work force development organization that uses several vans to transport clients from community homes to jobs and training facilities. They also transport clients from their central training facility to job interviews and other work related events and activities.

The Boys and Girls Clubs is an example of a youth service organization that uses vans to support its basic activities. They use the vehicles to transport their clients to and from activities at their central facilities, as well as to community based activities and for field trips.

The Eternal Life Harvest Center is an example of a faith-based organization that uses several vans to support its community based ministries and services. The vehicles are used primarily to meet the mobility needs of its youth ministry and after school service program. Bethel Baptist Church also uses a vehicle to transport clients to and from its day care center and other events in the community.

With an average placement of more than 50 vans per year, the interest in Tennessee Vans vehicles shifts with mobility needs. In the past five years there has been increased participation by community and economic development and youth service organizations.

Participation in the program categories also varies by community and reflects the mobility needs in these areas. For example, today the vanpool market remains strong in Nashville, a large metropolitan area with a strong downtown employment base. The conventional commuter vanpool market in Memphis and Knoxville has all but disappeared. Through the leadership of the Metropolitan Planning Organizations (MPO) in Memphis and Knoxville, Tennessee Vans has substantially broadened its role to meet a variety of mobility needs. In these communities there are 200 vehicles operated by 156 different organizations.

TENNESSEE VANS VEHICLE UTILIZATION

To understand more about the exact nature of the vans usage, a survey of participants was conducted. The definitions of van utilization were based upon a telephone interview of 45 percent of the organizations/agencies operating one or more Tennessee Vans vehicles across the State of Tennessee. These include 99 telephone surveys in total. Information was obtained about the trip structures served, number of riders, characteristics of riders and their access to alternative modes if a Tennessee Vans vehicle was not available. The survey data were supplemented with annual van odometer mileage obtained from the Tennessee Vans administrative office. Estimates were made from the surveys and trip logs of the additional vehicle-miles of automobile travel required to serve the existing mobility patterns of the Tennessee Vans. It was determined that an average Tennessee Van vehicle is operated 910 vehicle miles per month, but if these trips were to be served by a private vehicle, riding with family or friends or a staff vehicle, an additional 4600 vehicle-miles of travel would be required. This considers the availability of alternative transportation and vehicle occupancies based on the specific trip structures used by the agency. Thus, to maintain existing mobility levels, the Tennessee Vans vehicles are reducing automobile vehicle-miles of travel, improving air quality and providing fuel conservation benefits to the community. The benefits from Tennessee Vans include a reduction of air pollution by 44,453,000 grams/day for HC; 418,649,000 grams/day for CO; and 29,333,000 grams/day for NO_x, and a reduction in fuel consumption by 1.4 million gallons annually *(4)(5)*.

Summary statistics indicate that the Tennessee Vans vehicles provide about 2 million annual trips. Financially, the Tennessee Vans service is significant in contrast to the services provided by conventional fixed route, fixed schedule bus services. When considering expenditures, revenue, and cost per trip, Tennessee Vans generates a strong revenue stream. With no driver costs and other operating costs largely absorbed by the operating agencies, Tennessee Vans has a distinct financial advantage. Based on a current financial analysis, the public cost per trip for Tennessee Vans is approximately $0.17 per trip versus $1.97 for the small urban transit systems, $2.50 for the large city urban transit systems and $6.63 for the rural transportation operators in Tennessee

(5).

MOBILITY IMPLICATIONS OF TENNESSEE VANS

From the telephone survey, 29 percent of the organizations stated that in the short term they could not maintain their existing programs without access to a Tennessee Vans vehicle. Another 17 percent stated that some clients would be left without mobility and could not participate in the programs. Only 10 percent stated their clients could rely on public transit or walking. Another 44 percent stated that clients would have to rely on private vehicles, parents or carpooling. Tennessee Vans vehicles are operated by many organizations that provide essential mobility to their clients. Closure and curtailment of services would fall hardest on organizations providing community/economic development and youth services.

RELIANCE ON TENNESSEE VANS VEHICLES

While some organizations use Tennessee Vans vehicles to supplement their fleet purchased from other sources, many had attempted to purchase a vehicle from a private dealership with little success. In most cases, the private sector would not provide the financial flexibility or extend the credit required for the organization to secure a van. Also, about a fourth of the organizations attempted to acquire a vehicle as part of public capital grant programs, but without success. Almost half of the organizations reported that a Tennessee Vans vehicle replaced an older vehicle. These organizations benefited from acquiring a newer, safer and more reliable vehicle. The sources or revenue used to pay for the Tennessee Vans vehicles include:

Source of Funds:	Percent of Organizations Selecting
Fares	6.1%
Organizations/Program Revenue	19.5%
Daycare or Tuition Fees	9.8%
Social Service Grant & State Vouchers	23.2%
Donations	41.4%

It was stated that the Tennessee Vans vehicles were utilized because of the attractive payment plan, reasonable rates and no down payment. The organizations stated Tennessee Vans vehicles were selected for the following reasons:

Reason:	Percent of Organizations Selecting
No Down Payment Required	35%
Low Cost and Attractive Payment Schedule Plan	61%
No Interest Payment	10%
Simplicity with Maintenance, Insurance, Flexibility of Program and Help of Staff	19%
Obtain New Van in timely Manner	10%
Had No Other Options	1%

It is clear that these organizations value the simplicity and financial flexibility provided by Tennessee Vans. They are not in a good financial position to utilize conventional credit to lease or purchase a van. Yet the Tennessee Vans program has a default rate of less than 5 percent of vehicles placed in the purchase program.

CONCLUSIONS

Tennessee Vans has experienced a steady rate of growth in funding and service development since its implementation in 1990. Financial resources provided during this time have enabled the program to procure over 500 vehicles statewide and to place into service approximately fifty vehicles per year in the lease and purchase programs. These vans are helping to fill mobility gaps through the provision of affordable vehicles.

The underlying service delivery approach has the basic premise of allowing customers to design the mobility services that directly meets their needs. This user-based service design model differs substantially from the typical provider-based service design, wherein mobility services are solely designed and operated by a centralized service provider (e.g., fixed route, fixed schedule, public transit). In the provider-based model, the services are centralized, design decisions are provider-driven, and operations are provider-based. In the user-based model, the services are decentralized, design decisions are user-driven, and operations are user-based. The user-based service design model enables a high degree of user participation in transportation decision making, thus making the Tennessee Vans service essentially market driven with regard to its evolution of services. In essence, the service delivery approach adopted by Tennessee Vans is highly viable because it works intentionally to meet the mobility needs of its evolving market, as indicated by the results of the program participant surveys.

Tennessee Vans is an example of an effective model of forging public/private partnerships to meet mobility goals and objectives. Most of the program participants are private commuter groups, businesses, and private non-profit organizations. Very few participants are public groups and organizations, presumably because these groups have ready access to vehicle and financial resources through public grant programs. Each participant forms a partnership with Tennessee Vans to provide van transportation services to meet the mobility needs of persons served by the participating organization. In exchange for access to vehicles and affordable financing provided by Tennessee Vans, the program participant agrees to design and provide safe and reliable transportation services to meet identified needs and to pay for the Tennessee Vans vehicles. The public/private partnership is expanded through the participation of private companies that manufacture, maintain and insure the vehicles and by the private individuals and groups who pay for the vehicles and their use.

While the nature of the Tennessee Vans service varies from traditional fixed route and demand responsive services, Tennessee Vans is comparable to transit operations in terms of fleet size, annual trips served and annual vehicle miles. With regard to public costs, Tennessee Vans services provide a cost-effective approach with public subsidies that are much lower than subsidies for public transit operations.

The van transportation program provides an affordable option for program participants as they strive to overcome transportation problems that are barriers to achieving their organizational goals (e.g., employment, training, community service, etc.). The Tennessee Vans program provides essential services to meet the transportation needs of diverse travel markets, including employment, job training/education, health care, and human services. This program is an innovative, cost-effective, user-based service delivery model that helps to meet growing mobility demands in communities now and into the future.

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Transportation Asset Management Today: An Evaluation of an Emerging Virtual Community of Practice

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ABSTRACT

Asset management is defined by the Federal Highway Administration (FHWA) as "a systematic process of maintaining, upgrading, and operating physical assets cost effectively." As an emerging discipline, asset management means different things to different practitioners, and its practice continues to evolve as researchers develop guides, states implement specific components, and all units of government work to define what asset management means to them.

Recognizing that communication is a key element as different stakeholders tailor asset management to the needs of their organization, the American Association of State Highway and Transportation Officials (AASHTO) and the FHWA have developed a virtual community of practice (COP) via a Web site that facilitates communication among the different stakeholder groups.

The COP itself is a self-defined group of practitioners with an interest in transportation asset management. The Web site provides access to related links and a calendar of events, while individuals can register for topics that match their personal interests, view reference materials and post their thoughts on discussion boards. The site was launched in early 2002, but was not

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publicized until late 2002. As of May 2003, the community of practice includes over 250 registered individuals.

This paper evaluates the elements of transportation asset management that make the creation of an Internet COP an appropriate method for advancing its education and practice. In addition, this paper gives an overview of the history, structure and content of the Web site, and offers a preliminary evaluation based on site usage and other observations.

Key words: asset management—community of practice—internet infrastructure management

INTRODUCTION

Asset management is an emerging discipline among those who manage our country's substantial transportation system (*1,2*). This systematic approach to maintaining system infrastructure and monitoring its performance over time offers a new way of doing business for most transportation agencies, one that promises more efficiency and better accountability to the public.

The American Association of State Highway and Transportation Officials (AASHTO) has taken a leading role in promoting the practice of asset management, and a key part of their efforts is occurring via the Internet. With the support of the Federal Highway Administration (FHWA) and the cooperation of the Transportation Research Board (TRB), AASHTO has created *Transportation Asset Management Today*, a "community of practice" web site that allows anyone in the transportation industry to access asset management reference materials and interact with other professionals about its practice (*3*).

The first section of this paper will explain asset management from a transportation perspective, and provide a brief look at where its practice now stands. Next, the paper will describe AASHTO's Internet efforts for promoting asset management practice and education, reviewing the initiative to create a site, how the site has been structured, and some of the site's history up to this point.

ASSET MANAGEMENT IN TRANSPORTATION

As it applies to the transportation industry, asset management is more of an overarching strategy than a set group of well-understood tactics. While there is a general understanding of the goals and theory of transportation asset management, there is no absolute definition of what its practice entails. Put more clearly, asset management can mean different things for different people.

One useful "working definition" has been created by the FHWA. Asset management is "a systematic process of maintaining, upgrading and operating physical assets cost effectively. It combines engineering principles with sound business practices and economic theory, and it provides the tools necessary to facilitate a more organized, logical approach to decision-making." (*4*)

In reviewing this definition, two things become evident. First is the central idea that by logically and efficiently investing in transportation infrastructure such as roads, bridges, and tunnels, a public agency can more cost effectively operate its assets. Second, for asset management to be effective in practice, a multidisciplinary approach is required that integrates such disciplines as engineering, economics and business. This facet of asset management is an especially important factor for AASHTO's Internet efforts, as will be discussed later.

As an agency approaches the practice of asset management, these goals of cost-effectiveness, efficiency and public accountability will be at the forefront. But as noted above, a number of different strategies can be used to those ends. Some agency activities that would fall under the banner of asset management are listed below, and even within these strategies are a variety of different approaches:

- Creating and consistently updating an inventory of assets operated by the agency, including some measures of condition, use and performance.

- Changing methods of asset valuation to recognize user costs and benefits.

- Fully reporting agency inventory and expenditure decisions.

- Implementing maintenance and capital improvement activities based on evaluation of user costs and benefits, optimization of investments, and/or maintaining a certain level of overall system performance.

- Building an overarching system that allows the comparison of investments across different asset groups.

No matter which actions are undertaken, the reasons why transportation agencies are now approaching asset management are clear. Asset management's growth as a discipline can be seen as the outcome of a larger shift in the transportation field — from creating the transportation system to managing it. This is especially true in for highways, where the network has been built out at a great public cost, new construction is exceedingly rare, and many segments of the existing system need upgrading and repair. Additional reasons for the growth in importance of asset management include greater public demands for government accountability and the developing capabilities of computer systems (*5*).

PRACTICE

While its need has become apparent, the practice of asset management has lagged behind its theory. Truly, part of the reason for a lack of clarity about the definition of asset management is that its application and benefits are still primarily theoretical.

A number of institutions and agencies — such as the FHWA, AASHTO, TRB, and the American Public Works Association (APWA) — have produced written reference materials and research reports on transportation asset management (*4, 5, 6, 7, 8*). However, these are generally an introduction to its concepts rather than an instructions manual or step-by-step implementation plan. And while these federal and membership organizations have lead the way in asset management awareness, the ultimate implementation of asset management principles falls to the state and local governments that actually manage the transportation infrastructure.

In this regard, asset management is in its infancy. Leading states have explored and implemented asset management initiatives, most often beginning by creating better inventories and collecting data. But excepting a few cases these efforts are not formalized or mandated by legislation, and actual results are often not yet publicly available (*9*). This lack of formal, documented asset management practice can be seen as a barrier to the wider adoption of asset management principles. Not only do state and local agencies not have specific asset management blueprints to follow, even examples and case studies can be difficult to come by.

This is not the only obstacle to a wider practice of asset management. Because an effective implementation of asset management could require a major change in institutional philosophy, and the true benefits will be seen over the long term, the advancement of its practice could often depend on the long-term mindset of key political and transportation agency leaders. In the transportation field, where each state and local agency deals with a set of similar but geographically separate issues, drives their own policies and procedures, and do not necessarily seek advice from outside their region, it seems that asset management will advance incrementally, as agencies take time to discover its benefits in their own environments.

TRANSPORTATION ASSET MANAGEMENT TODAY

The most persistent advocates of transportation asset management in the U.S. have been federal agencies such as the FHWA and national membership organizations such as AASHTO. Through conferences, meetings and educational outreach programs, these organizations have attempted to answer many of the existing questions about asset management, and explain the benefits of applying its principles in state and local environments (*10*).

A major challenge in educational outreach is the lack of educational materials. Because asset management is an emerging field, much of the knowledge that does exist about its practice is still developing and has yet to be codified. When combined with the challenge inherent in the nature of asset management — that it is hardly a one-size-fits-all proposition and requires knowledge from a variety of disciplines — there is a particular need for innovative outreach approaches.

Recognizing these particular challenges, AASHTO — along with FHWA and TRB — created Transportation Asset Management Today, an Internet site that offers an electronic library of reference materials on asset management and provides a “virtual” space where interested transportation professionals can interact on a peer-to-peer level. This not only allows individuals to access the most recent publications, research and case studies, but to also take their own individual experiences — the tacit “knowledge” that will never be written in a book — and share them with people throughout the field of transportation. Together, the hope is that professionals in the transportation field can use the site to help advance the state of the practice and begin establishing the “common language” of transportation asset management.

SITE INITIATIVE

The initiative to create the site was that of AASHTO’s Task Force on Transportation Asset Management, a committee created in 1997 to guide the association’s efforts in asset management and create an official “asset management guide” for the association (*11*). Along with help from the FHWA and TRB’s Task Force on Asset Management, members of AASHTO’s Task Force built an initiative to provide a central Internet location for their outreach efforts, but in a way that was interactive and recognized the particular challenge of asset management.

Participation by the FHWA was key to this decision. This is because, for a number of years, the FHWA had been developing its “knowledge management” initiative, a group of Internet communities that provide a mechanism for communication and shared learning between its employees and the many other members of the transportation community. This effort has included internal, password-protected sites for FHWA employees as well as external “knowledge” sites built around topics such as highway safety and environmental requirements. These FHWA sites were built to tackle many of the same challenges faced by those hoping to provide education on asset management; namely, how to educate a dispersed population of transportation professionals on issues central to the FHWA’s mission, and how to take advantage of the tacit knowledge that already exists within this talented group of employees (*12*).

With the FHWA’s experience and technology in place, creating a site around the asset management education initiatives of the AASHTO Task Force was a matter of applying new principles to an existing technology. Working with AMS — the technology consulting firm that had created and managed the technology for FHWA's family of "knowledge management" sites — the initial structure of the site was created.

SITE STRUCTURE

The site's structure mirrors that of the other FHWA knowledge sites, and is built around different "topic areas" that all have the same structure and features. The site is completely open to the public, with all site features accessible for anyone to browse through.

Upon first visiting the site, the user is offered the choice of 19 different topic areas, and can navigate according to the facets of asset management that interest them most. The site is arranged into 19 different topic areas (15 that have existed from the beginning, and another four that have since been added):

- *Asset Management 101*: the basics of transportation asset management
- *AASHTO Guide for Asset Management*: updates and news about this recently published guide from AASHTO
- *Innovation and Success*: case studies and examples of asset management initiatives from around the country
- *Pavement Management Systems*: information on the latest technology and software for monitoring pavement investments
- *Bridge Management Systems*: a focus on the technology and methods available to professionals involved in bridge management
- *Tunnel Management Systems*: advice and current topics in maintaining transportation tunnel structures
- *Roadway Hardware Management Systems*: the latest information about tools and methods for managing other roadway assets (signals, signage, guardrails, etc.)
- *Maintenance Management Systems*: updates on procedures for documenting and coordinating their maintenance activities
- *Transportation Preservation*: initiatives for the preservation of existing transportation infrastructure
- *Integration of Data and Management Systems*: discussion and materials on what can be a major obstacle to effective asset management
- *Engineering Economic Analysis Tools*: updates and advice about the economic analysis of asset management initiatives
- *Research*: the latest studies and reports about the value and practice of asset management

- *GASB 34*: information about Governmental Accounting Standards Board Statement 34, which lays out a modified procedure by which government agencies can report their assets

- *AASHTO Asset Management Task Force*: information sharing for the task force themselves, and information about the group's meetings and goals

- *Asset Management at the Transportation Research Board*: similar information about asset management activities and conferences at TRB, including the TRB Task Force

- *Public Transportation (added 03/03)*: materials and discussion for public transportation agencies, which have a wide variety of assets to manage (vehicle fleets, guide ways)

- *Performance Measures (added 03/03)*: a topic considering the various methods of rating the performance of a piece of infrastructure

- *Local Government Perspectives (added 03/03)*: for local governments practitioners, who are responsible for managing much more than just transportation infrastructure

- *Education (added 03/03)*: a topic for the many individuals, both within the academic field and elsewhere, tasked with teaching the theory and practice of asset management

Within each of the different topic areas listed above are a set of features that have the same look and feel no matter which area one is in. Each of these different features can be accessed by clicking on one of a group of four tabs arranged across the top of the screen.

The discussion boards are the primary feature of the site, and are thus the first to display when a site visitors enters a topic area. Any site visitor can post new topics on the discussion boards or respond to previous posts, and users have the option to identify themselves, or to post anonymously.

The references area contains a categorized group of postings that provide access to and explain what materials there are that exist on asset management, including primers that have been written by the FHWA, research reports and presentations from recent conferences. These posts, with web links or attached electronic files, can only be made by site facilitators and administrators (explained below).

The works in progress portion of each topic area is a space where individuals can post reports, papers and other items that they have been working on, submitting it to the site in hopes of receiving feedback from others. Once a post has been created by a site administrator or facilitator, anyone can respond and give comments based on their experience.

Finally, each topic area contains a directory, which is a list of those individuals that have registered on the site and indicated interest in that particular topic. This listing includes names, professional affiliation and an email address. This directory is seen as important to helping establish a core group of individuals, and helps the users get a sense for the types of individuals (public sector, local, consultants, etc.) that make up the transportation asset management "community."

ADDITIONAL SITE FEATURES

In addition to the various topic areas listed above and the features within them, there are a two other key features that shape the user's experience. These are the site's event calendar and registration form.

The event calendar offers users access to information on asset management-related conventions, meetings, workshops and training. This information appears as a chronological listing of events, generally with a brief introduction to the event, where it is located, and where to go for additional information.

The site registration form, accessed off of the home page by clicking on a link labeled "My Interests," is one of the most valuable features of the site. This allows users to personalize their site experience by choosing the topic areas that most pertain to their particular interest. Registering on the site not only places their names in the site directory, it ensures that they are sent an email update every time that a new posting or reference has been put in an area of interest. This feature has driven a significant amount of traffic to the site, as users are able to click directly from the email through to the posting that they have an interest in.

SITE ADMINISTRATORS AND FACILITATORS

Another key component of the site's current success and future potential is a group of practicing transportation professionals that have been enlisted to guide the site's offerings and serve as a resource to site visitors: these are the site administrators and facilitators.

Site administrators are individuals that were tasked by the AASHTO Task Force to create and administer the site. These individuals have the ability to alter any aspect of the site, including deleting inappropriate or mistaken postings from site users. The site's administrators are Lou Adams, webmaster, from New York Department of Transportation; Sue McNeil, University of Illinois-Chicago; and Thomas Van, FHWA.

In addition to the administrators, who have overseen the implementation of the site as a whole, each topic area on the site is hosted by a facilitator—an individual who serves as an "expert" in their respective topic area and ensures that any postings to the site are answered in a timely fashion. In addition, site facilitators help to drive the content within the different topic areas, keeping an eye out for news updates and possible reference materials.

Because of the variety of topic areas on the site, there are a wide variety of facilitators participating, representing different backgrounds and mirroring the diversity of individuals working within transportation asset management. For example, site facilitators include officials from the FHWA; individuals representing many of the state departments of transportation; academics; and private consultants. In addition, these individuals have professional backgrounds in fields such as engineering, finance, economics, and public administration, helping site users to gain access to a variety of asset management perspectives.

SITE HISTORY AND USAGE

Although the structure of the site was finished near the end of 2001, it took a year before the site had been fully organized and promoted. In that time, facilitators for all of the areas were sought out, and each topic area was populated with what reference materials were in existence. Table 1 below offers a brief timeline of the site thus far.

TABLE 1. Site Timeline

Date	Activity
Mar 2001	AASHTO Task Force approves use of FHWA Community of Practice template for development of site
Dec 2001	Site created by AMS, site administrators chosen, first references posted
Aug 2002	University of Illinois-Chicago team begins efforts for promoting, managing site
Oct 2002	Existing reference materials organized, facilitators trained on site operations
Jan 2003	Promotional brochures released, site is demonstrated at TRB 2003
Mar 2003	New topic areas added
May 2003	Site traffic reaches nearly 2,000 visitors per day

Since December 2002, the site's promotion has driven traffic numbers up and helped introduce the site to new users. These efforts have included an informational brochure mailed to transportation officials around the country; a live demonstration at the AASHTO booth during the 2003 TRB Meeting, where individuals were able to see the site live and register; and articles in various transportation publications about the initiative (*13*).

Since the site's official launch in early 2003, site traffic and registration has continued to grow. Table 2 shows some of the vital statistics starting from December 2002 (FHWA, *unpublished data*), when word about the site began to spread, up until the present time. In addition to this documented site activity, as of May 2003 over 250 individuals have registered to receive the regular email updates, which have served to as a significant boost for traffic to the site.

TABLE 2. Site Traffic Trends

Month	Successfu l Hits	Page Views	Page Views per Day	Visitor Sessions	Avg. Sessions per Day	Unique Visitors
Dec 2002	31,931	10,373	334	1,709	55	879
Jan 2003	41,599	11,113	358	2,394	77	1,180
Feb 2003	44,852	14,259	509	2,557	91	1,232
Mar 2003	49,395	15,904	513	3,116	100	1,455
Apr 2003	46,333	13,247	441	2,996	99	1,454
May 2003	54,621	15,729	507	3,821	123	1,981

In addition to consistent growth in traffic, there continues to be consistent activity on the site's discussion boards. By May, one post asking about the definition of asset management had received 19 responses. There is certainly more room for growth in the future, as the goal of site administrators is to create a regular destination for visitors wanting to keep up with the latest trends in asset management from across the country and around the world.

CONCLUSION

After less than a half year, the Transportation Asset Management Today site has built a strong following, reaching an average of nearly 2,000 visitors a day. Many of these individuals have posted information to the site and the collective pool of information has consistently grown.

As the practice of asset management begins in many agencies — and begins showing results in others — there is an expectation that this trend will continue. It is certain that the site will remain a key portion of the efforts of AASHTO and FHWA to promote learning and information-sharing about asset management.

Finally, it will be valuable down the road to assess the state of asset management practice and the effect that this Internet initiative had in connecting a "community" of asset management practitioners. From this, it could be ensured that this innovative use of the Internet as a collaborative learning and outreach tool offers lessons for similar transportation-industry efforts in the future.

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Monitoring and Evaluation of a Plate Girder Bridge

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ABSTRACT

Many of Iowa's multiple steel girder bridges are experiencing fatigue cracking of the girder webs at diaphragm connections. The state, along with Iowa State University, has been investigating a retrofit method involving bolt loosening in the diaphragm/girder connection. Testing of this retrofit has included continuous monitoring of a demonstration bridge to assess the impact of the retrofit over an extended period of time.

A continuous monitoring system was designed and installed at a bridge site and connected to instrumentation to monitor the bridge's response to the retrofit. Electrical and telephone utilities were installed at the site to power the unit and to allow for remote control of the system with data transfer capabilities. The data collection software was fully programmable and could be adjusted and reprogrammed as needed. The data downloaded from the system were used to determine the effectiveness and stability of the retrofit over an extended period of time.

The continuous monitoring testing was a pilot project for health monitoring bridges in Iowa. The developed data acquisition system can be easily adapted to a variety of remote monitoring applications. The robustness of the system and its ability to be controlled remotely make it a useful tool for future remote monitoring activities.

Key words: continuous monitoring— fatigue—girder bridges—retrofit—web gap

INTRODUCTION

Bridge 4048.2S017, shown in Fig. 1, is a three span bridge crossing the Boone River on Highway 17 in central Iowa. As shown in Fig. 2, the end spans are 29.7 m and the interior span is 38.1 m. The girders are labeled as G1 to G5 and the diaphragms in each span are labeled as D0 to D5. The bridge has five main plate girders with an integral concrete deck as shown in Fig. 3. The 1.5 m deep girders are connected by X-type diaphragms every 6.1 m. A visual inspection revealed no fatigue cracking in the girder webs near the connections to the diaphragms. However, many multiple steel girder bridges with this diaphragm connection detail in Iowa have experienced cracking in that region.

FIGURE 1. Photographs of Bridge Deck and Profile

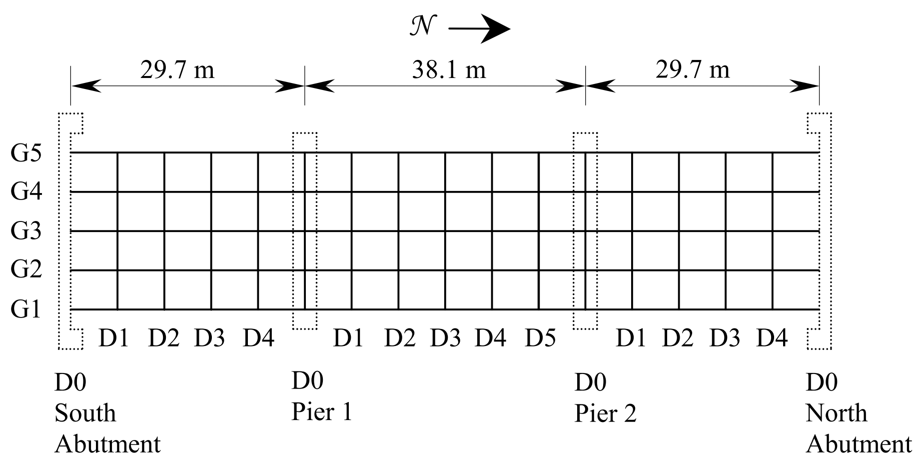

FIGURE 2. Plan View Illustration of Superstructure

The bridge was selected for study because visual inspections have not yet identified signs of fatigue cracking in the web. A bolt loosening retrofit designed to eliminate web cracking was installed on the bridge [1,2,3]. The retrofit consists of loosening bolts connecting the negative moment diaphragms to the main girders. Ambient truck loading behavior data (i.e. web strain, out-of-plane displacement, etc.) were collected over several months before and after installing the retrofit to study the effectiveness and stability of the retrofit.

The on-site remote monitoring system was constructed from off the shelf components to record data from the bridge. The stand alone Data Acquisition System (DAS) was used in conjunction with strain, displacement, and temperature sensors to monitor the bridge continuously. The system was controlled remotely and relayed data back to the laboratory for reduction and interpretation. Data were collected only when significant loading was present on the bridge to save storage space and to simplify analysis.

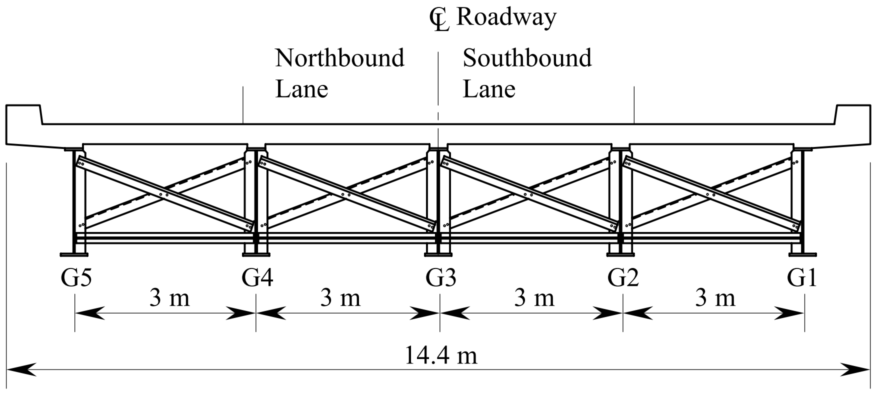

FIGURE 3. Illustration of Bridge Cross Section at Negative Moment Diaphragm

DESCRIPTION OF PROBLEM

Many existing steel girder bridges in Iowa had the same connection design between the diaphragms and webs. Fatigue cracking has occurred in the web gaps of this design, especially in the negative moment region, because the stiffener is not attached to the girder top flange which allows out-of-place distortion. The web gap is defined as the region of the web between the top flange fillet weld and the stiffener weld and is typically only an inch in length in the vertical direction. Typically the cracks found in this region are horizontal, parallel to the top flange, and extend up to a few inches away on either side of the web gap.

The Iowa Department of Transportation (DOT) has attempted to control the cracking in the web gaps by drilling holes at the crack tips to reduce the stress intensity factor [4,5]. The hole drilling technique has not always proved successful and cracking has continued past the holes in some cases. Cracking past the drilled holes can be caused by inaccurate placement of the hole with respect to the crack tip, or by stresses high enough to continue crack propagation.

Fatigue cracking in the web gap is a result of differential deflection of the adjacent girders due to asymmetric traffic loading on the bridge deck. As vehicles cross a bridge, the adjacent girders deflect differently at the same cross section and the diaphragms between the girders are distorted. This is especially true on bridges with skewed piers. Forces created in the diaphragms cause a rotation at the connection between the girder and the diaphragms. The force is transferred to the girder by the web stiffener, which causes the web gap to displace out-of-plane. The stiffener pulls at the web, but the top flange, being integral with the deck, resists movement and the result is out-of-plane displacement as illustrated in Fig. 4. Continued cycles of this out-of-plane displacement result in fatigue cracking in the web gap. Exterior girders are especially prone to fatigue cracking because, unlike interior girders, they do not have a diaphragm on both sides of the girder to oppose the out-of-plane forces.

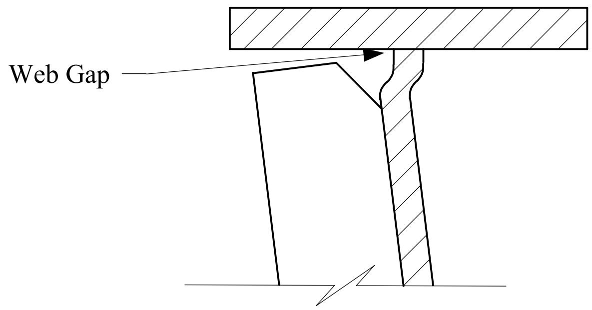

FIGURE 4. Out-of-plane Distortion in Web Gap

SYSTEM DESCRIPTION AND USE

A Campbell Scientific CR 9000 was selected as the base for the remote continuous monitoring data collection system. The system was set up in an environmental enclosure on Pier 2, which provided the system protection from the weather and vandalism as shown in Fig. 5. The system has an input capacity of 24 channels with further expansion possible. Thermocouples, strain gages, and displacement transducers were used in this study, but the system is capable of supporting many different sensor types. Instrumentation was focused at a diaphragm in the negative moment region near Pier 2 as illustrated in Fig. 6.

Four thermocouples were used to monitor the bridge steel temperature, the temperature in the enclosure, and the ambient temperature. These sensors were the standard K type thermocouples.

Two gradient strain gages were installed in the G1 and G2 web gaps as shown in Fig. 7. These gradient gages consist of five small, foil backed strain gages placed within the web gap to collect a detailed strain profile. For this study the strain in the web gap was the most important indicator of the effectiveness of the retrofit.

FIGURE 5. Photographs of Environmental Enclosure and CR 9000

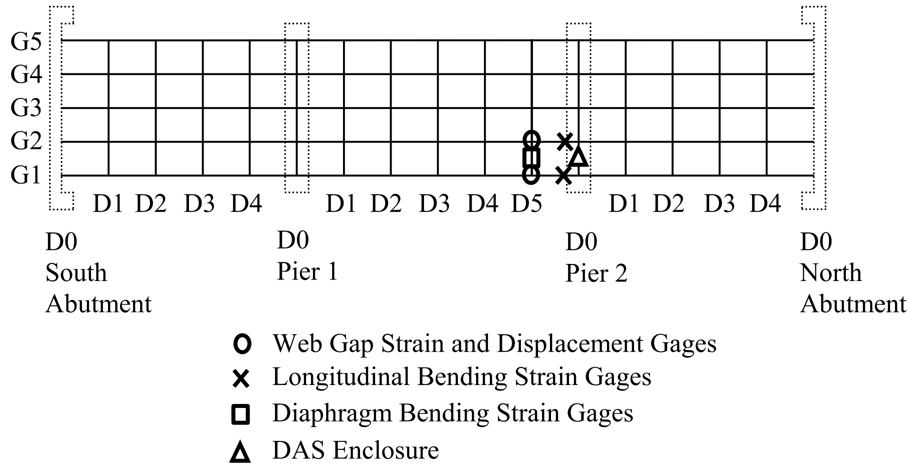

FIGURE 6. Illustration of Instrumentation Locations on the Superstructure

Three weldable strain gages were mounted on the diaphragm members between G1 and G2 as pictured in Fig. 7. The gages were encased in plastic to provide protection from the environment.

Four Direct Current Displacement Transducers (DCDT) were installed at the stiffener connection of D5. Two were placed on G1 and two were placed on G2 as shown in Fig. 8. One transducer measured out-of-plane displacement of the web gap and the other measured vertical movement of the flange relative to the stiffener. Out-of-plane displacement of the web was indicated by the change in horizontal position of the stiffener with respect to the top flange, which was integral with the deck. The out-of-plane displacement of the web is an indicator similar to the strain in the web gap of the effectiveness of the retrofit.

Two weldable strain gages were also used to measure the strain in the bottom flange of G1 and G2 at 0.9 m from the bearing at Pier 2 as pictured in Fig. 8. The strain in the girders reflects the global effect the retrofit has on the bridge and was also used as a trigger for the DAS as described below.

FIGURE 7. Photographs of Diaphragm and Web Gap Strain Gages

FIGURE 8. Photographs of Longitudinal Strain Gages and Out-Of-Plane Transducers

The DAS collected data in a unique manner. The system monitored the bridge continuously and stored approximately 20 minutes of data in temporary memory. These data were not used in the analysis. As more data was collected it recorded over the previously recorded data to update the temporary memory. In order for the system to record data to its permanent memory the data had to be considered useful to analysis. A trigger threshold, which reflected the magnitude of the load, had to be reached by a selected input channel. In this case, a strain gage on the bottom flange of G2 was used as the trigger channel. The system was programmed to recognize any value greater than 20 microstrain in the bottom flange to be a truck of notable size at which time the data was recorded to the permanent memory. Eight seconds of data prior to the trigger value being reached and eight seconds after were stored permanently, allowing the event to include all data involving the trigger load crossing the bridge.

The data stored in the permanent memory of the DAS were retrievable via modem connection. All parameters involving control of the unit were also accessible through the modem connection. Data collection programs could be uploaded to the unit and turned off and on remotely. The system was also capable of generating real time plots of data so that important channels could be visibly monitored if needed.

Data generated by Iowa DOT load trucks of known weight and dimension were recorded with and without the retrofit in place. These load trucks were used as base line data for evaluating the ambient data. The 20 microstrain trigger threshold was determined using the DOT load truck data so trucks of similar loading could be collected. Twenty microstrain is equivalent to a truck weighing approximately 22,680 kg, which is the loading of the DOT trucks.

Four months of ambient data were collected with the bolts in the tight condition (no retrofit) and another four months were collected with the bolts in the loose condition (retrofit installed). The four months of data were used to evaluate the effectiveness of the retrofit over an extended period of time following implementation.

RESULTS

Typical plots of the 16 seconds of data for the DOT trucks are shown in Fig. 9 before and after implementation of the retrofit. The DOT load trucks showed a substantial (nearly 90 percent) reduction in strain in the G1 web gap following implementation of the retrofit. Other sensors in the web gap region also showed similar reductions with the exception of the bottom flange strain in the girders. The girder strain showed very little change due to implementation of the retrofit, and because of this the bottom flange strain was useful as a trigger.

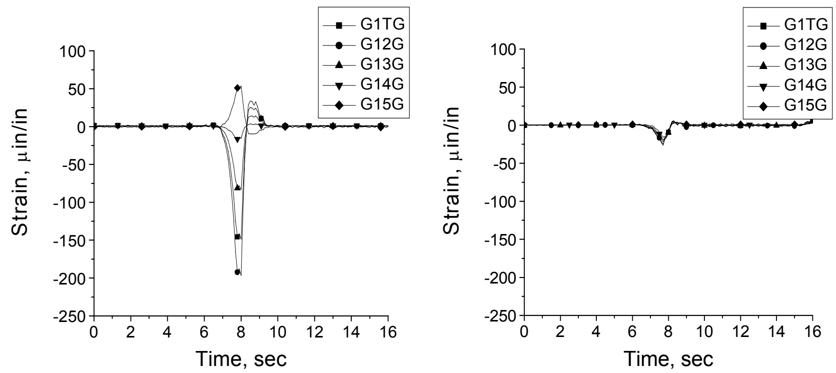

FIGURE 9. Web gap Strain Vs. Time Plots before and after the Retrofit

The girder strain showed very little change after implementation of the retrofit so that the bottom flange strain was useful as a trigger. The strain in the girders during the DOT truck loading was similar with and without the retrofit, about 20 microstrain as shown in Fig. 10, even thought the trucks were of slightly different weights. To correct for the difference in weight the plots from before the retrofit were normalized to represent equal loads of approximately 22680 kg in both tests. The longitudinal strain was used to identify load size of ambient trucks before and after the retrofit was installed.

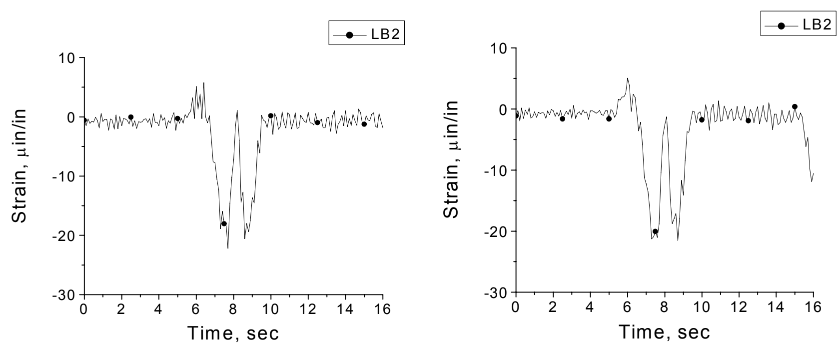

FIGURE 10. Longitudinal strain vs. time plots before and after the retrofit

A sample ambient truck similar to the DOT load trucks as far as causing similar longitudinal strain patterns and peak strains was selected for each of the four months before and after the retrofit was implemented. The longitudinal strain and web gap strain for each of these trucks is plotted in Fig. 11. Web gap strains for each of these trucks and the DOT trucks are similar both before and after the retrofit was implemented suggesting that the reduction in web gap strain does not change over time.

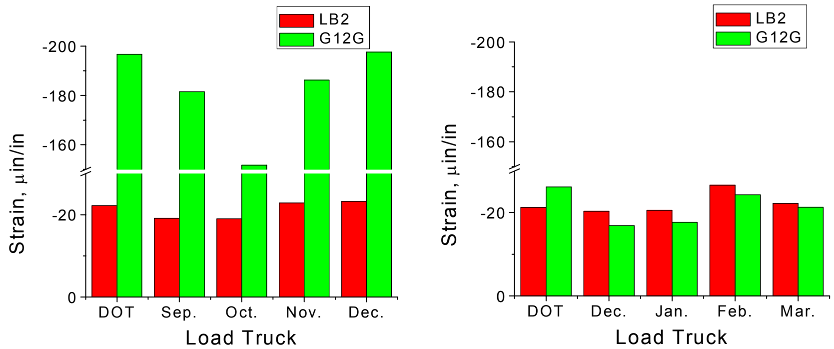

FIGURE 11. Long Term Web Gap and Longitudinal Strain before and after the Retrofit

CONCLUSION

The results of the testing show that the strain in the web gap is significantly reduced with the implementation of the bolt loosening retrofit. The exterior girder web gaps show the greatest reductions, but all web gap and diaphragm instrumentation showed reductions of over 50 percent.

The long term testing program revealed that the overall results (the peak strain and displacement associated with a loading) did not change over time when the retrofit was implemented. This suggests that the retrofit will be effective in reducing or eliminating fatigue cracking in the web gap and the effectiveness will not deteriorate over time.

The long term data from ambient loading also highlights the performance of the DAS. The system continuously recorded the needed data for analysis of the retrofit and withstood the rigors of on-site installation. The ability of the system to be controlled remotely saved time when data downloads or new program uploads were needed. The system segmented the truck loadings so the files could be input into a spreadsheet program as separate events. The system also counted the number of events since the last download and showed basic statistics on the data. The capabilities of the Campbell Scientific CR 9000 were instrumental in the completion of this project.

The DAS had other untapped uses that were not employed on this project. The software could be programmed to collect only peak values, instead of timed quantities of data surrounding a trigger. The peaks could be set up in a statistical analysis of traffic crossing the bridge by the system as it collects the data. This would provide an easily accessible overview of how many different size trucks crossed the bridge and the strain in the web gap, or damage, they caused.

ACKNOWLEDGEMENTS

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation.

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Asset Management and City Government

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ABSTRACT

Asset management is an emerging set of tools and skills that can help managers of transportation facilities make better maintenance and investment decisions. Since local governments own and operate 78 percent of the streets and roadways in the nation, most of the airports and most of the transit systems, some effort must be made to determine how local officials are progressing in the use of asset management.

To this end a telephone survey was conducted with 40 small- and medium-sized cities. Officials in these cities were asked about their practices in defining system performance goals; in planning and programming; in the collection, storage and use of data; in program implementation; and in program monitoring.

Cities reported the use of fairly advanced practices in the collection, storage, and analysis of data. They also use a wide range of contracting methods to secure the services of private contractors for many activities ranging from construction to data collection. They also generally have strong procedures for defining system performance objectives, using strategic planning and public involvement techniques.

On the side of concerns, many reported that their primary investment decision criteria was "worst-first," which would suggest the good efforts in the collection and analysis of data and the work in strategic thinking may not be fully utilized. Similarly, on the "soft" side of organizational issues, ensuring that the entire organization works toward the same goals, many cities continue to have challenges.

Key words: asset management—local government—strategic planning—transportation investment

INTRODUCTION

Asset management has been a major topic of discussion in the transportation community of the U.S. since the mid-1990s. A great deal of that discussion has been on states and what state departments of transportation could do to improve the management of state facilities, particularly state highways. What these discussions have sometimes missed is the huge role that local governments play in managing the transportation system of the country. Local governments own and operate 78 percent of the roadway miles in the country. They also own nearly all of the 5,300 public airports, and nearly all of the 6,000 public transit systems with more than 120,000 vehicles. Given the importance of local management in transportation, some attention to local adoption of asset management concepts is appropriate.

To gain a better understanding of the efforts of local governments, a telephone survey was done of a randomly selected sample of forty small- and medium-sized cities. Survey questions focused on the management practices used that are generally included under the broad heading of asset management. The overall conclusion of this survey is that cities are doing most of the things considered asset management: they are defining goals, inventorying and monitoring conditions, trying to bring together all parts of their organizations, and experimenting with innovative program delivery techniques. It is also clear from the survey that much confusion exists over the meaning of asset management and that no cities have put together all of the parts into a comprehensive asset management program. Like most states, cities are also struggling to define how to best proceed with managing their systems and improving their services.

WHAT IS ASSET MANAGEMENT

Since the survey used the term asset management in only one of twenty-five questions, it is important to understand how the authors defined the term as they structured the survey instrument. Following the model of the *National Cooperative Highway Research Program (NCHRP) Guide to Transportation Asset Management*, an asset management approach should have the following five components:

1. A strategic approach to defining the goals to guide the maintenance, operation and improvement of a transportation facility.

2. A planning and programming process that translates strategic goals into tangible actions that will result in the attainment of those goals.

3. An inventory, data and analytic system that measures the extent and condition of the facility, predicts the future condition of the facility, monitors progress toward defined goals, and supports the goal setting, planning and programming processes.

4. A program implementation process that maintains the strategic view of the facility, includes all relevant segments of the agency, and utilizes the most efficient and effective tools to implement programs.

5. A monitoring system that regularly measures the condition of the facilities, progress toward defined goals, predicts future conditions, and reports these findings to managers, professional staff and policy decision-makers.

Survey questions were designed to test each city's efforts in these five areas. Questions also allowed some assessment of how the cities defined asset management.

All the cities manage a wide range of asset types, as shown in Figure 1.

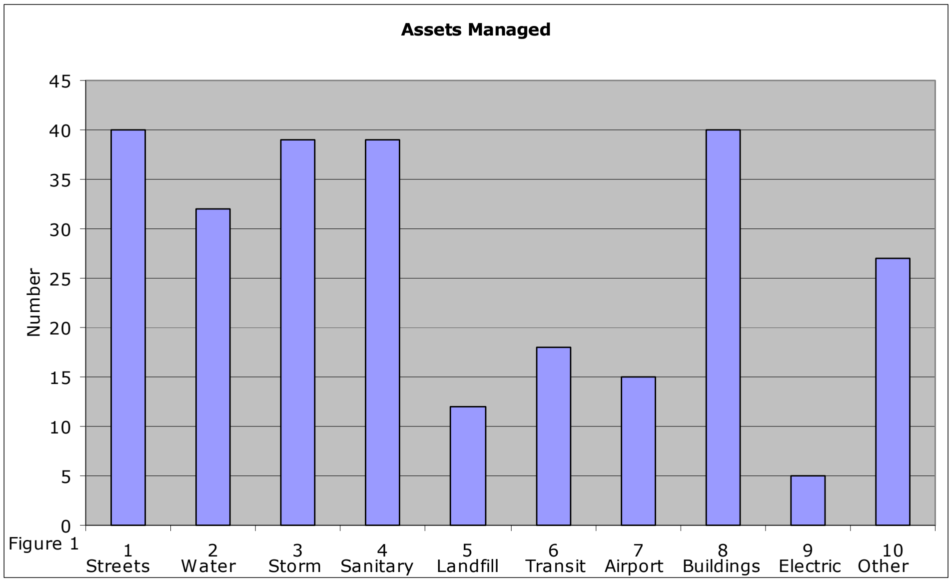

FIGURE 1. Assets Managed

When asked: Does your city have an asset management program? Thirty of the forty said that they did. Nine said they did not, and one was not sure. The activities of those who claimed to have an asset management program as compared to those who did not provides some insight into how the cities define asset management, although, given the size of the sample of those not claiming an asset management system (ten), the data should be used with some caution.

Those who claimed to have an asset management program were much more likely to look to non-transportation assets for their model. Ninety percent of those not using an asset management system chose streets as the asset to answer survey questions, as compared to only 55 percent of those claiming to have asset management systems.

Automated data collection systems were more prevalent in those cities claiming to have asset management systems. Nearly 60 percent of the cities that claimed to have an asset management system reported that they used automated data collection. Only 30 percent of those without an asset management system used such methods.

Just as data collection tends to be more sophisticated among those claiming to have an asset management system, so do the data storage systems. Cities claiming to have an asset management program were much more likely to report having some sort of database system, as opposed to paper or simple desktop computer applications.

In keeping with the previous two data issues, the cities professing to have an asset management program were also more likely to use more sophisticated methods for evaluating the condition of their assets. They tended to be less reliant on professional judgment and more likely to use defined standards and various devices for evaluating the condition of their assets.

The responses to the question of how the future condition of the system is predicted are somewhat reversed. Those claiming to have an asset management system were less reliant on professional judgment, more likely to use defined criteria, but less likely to claim the use of expert systems, as shown in Figure 2. One possible explanation for this apparent contradiction may be in the type of asset under review. Recall that those claiming to have asset management systems were more likely to select non-transportation assets for the detailed survey. Given the diversity of those assets, expert systems may not be as available as they are for highways, bridges or airports.

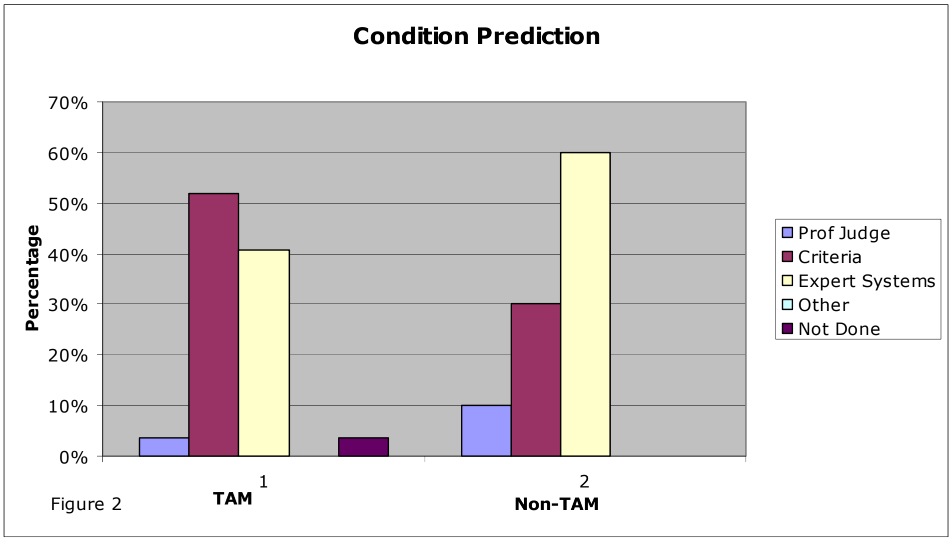

FIGURE 2. Condition Prediction

In all of those areas that might be considered "soft": goal setting, implementation, and involvement of the entire agency, no significant difference was found between those who claimed to have an asset management system and those who said they had none. This tends to confirm the fear that cities tend to define asset management as data systems, or analytic systems, as opposed to management processes.

SURVEY RESULTS

The survey measured city activities in each of the areas outlined above. The results are provided in the following sections.

Setting Goals, Planning, and Programming

Performance goals for facilities are generally established through sophisticated processes. Thirty-seven of the forty cited the use of a strategic planning process. Thirty-four of forty said they used results of performance measures. Thirty cited the involvement of political processes. Twenty-four said they generally used a process that involved the public.

More than half, twenty-three of forty, revisit facility goals at least annually. While this seems impressive, only twenty-nine of the forty choose to answer this question, which suggests that many may have no defined schedule for refreshing system goals.

System goals and conditions are translated into specific long-term needs in three ways: (1) the application of professional judgment (39 of 40); (2) based on variance from defined standards (34 of 40); and (3) through a political process (30 of 40 responses).

Actual investment decisions are made based on several factors. The primary of these is worst-first (34 percent). The advocates for asset management will cringe at this finding, since it tends to undermine the entire notion of asset management and strategic decision-making. Figure 3 contains the full answer to the question: How would you describe the approach that is taken to invest funds?

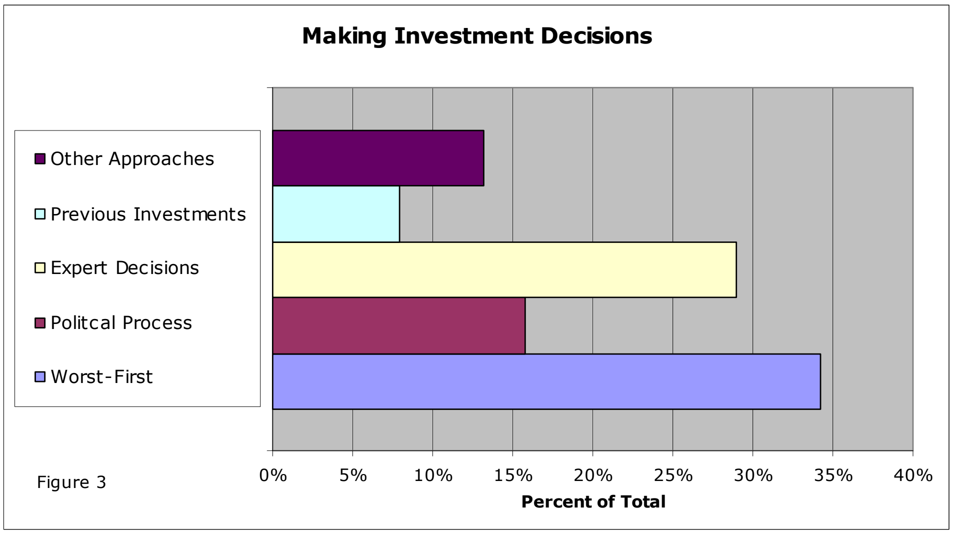

FIGURE 3. Making Investment Decisions

Three variables noted in the survey might be thought to influence how decisions are made: (1) the degree of professional management in the city; (2) the type of planning agency involved with the city; and (3) The presence or lack of an asset management system. Of these, the third, the presence of an asset management system, seems to have the most significant impact, but not the impact expected. Those cities claiming to have an asset management system were more likely to make investment decisions based on a worst-first approach. They were also more likely to follow a political decision making process, as shown in Figure 4.

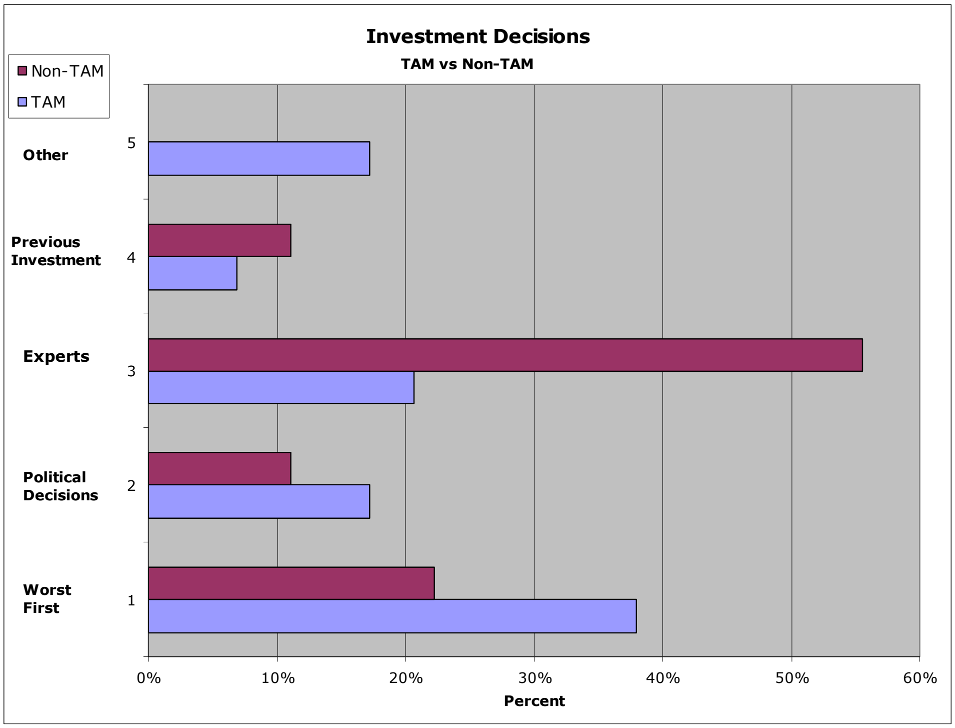

FIGURE 4. Investment Decisions

Inventory, Data Management and Analytic Systems

Inventory information is collected by all forty of the cities surveyed. Twenty reported using a manual process, and twenty, an automated process. Of those using an automated system, eighteen use laptop computers, or similar technology, twelve use digital or video photography, and eight use laser, or other sophisticated technology.

Data management systems vary significantly among the cities. Geographic information systems (GIS) are most commonly used, with twenty-three of forty reporting that they use them. Six cities use some other type of automated database. Five use desktop software, such a spreadsheets. Four still have paper files. And two had combinations of systems.

Predicting future facility conditions seems to be done some somewhat more consistently. Half of the cities said they used an expert system. Nearly half, nineteen of forty, reported that they used professional judgment. All but two of those relying on judgment had some type of defined standards against which they assessed and predicted conditions. Only one said that no forecasts were made.

Implementation

Carrying a strategic approach into program implementation seems to continue to be a challenge for cities. More than half of the cities rely on informal or no procedures to coordinate maintenance and capital programs, as shown in Figure 5.

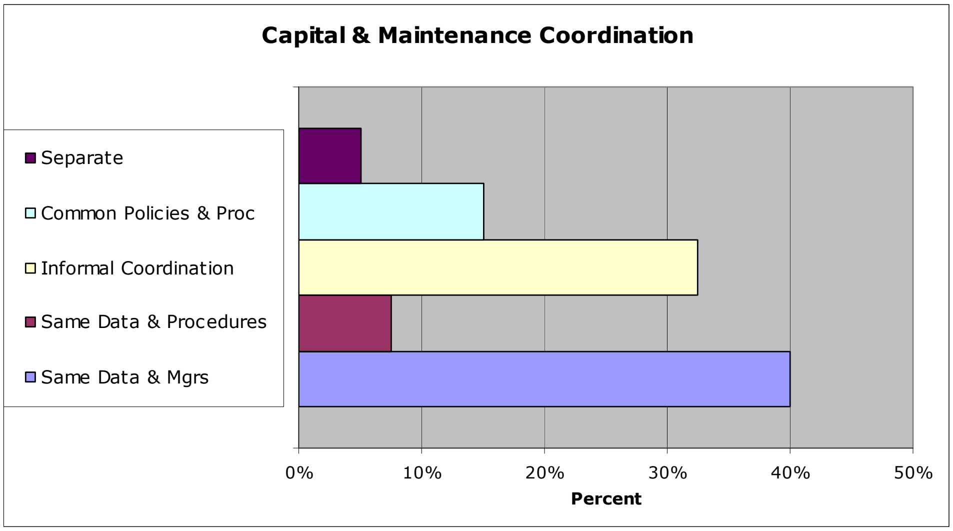

FIGURE 5. Capital and Maintenance Coordination

This separation continues in the implementation of investment programs. Seventeen cities say that the same people who develop programs implement them. Seventeen say there are different people in the two functions. Four report some of the same people in the two functions. And two do not know.

The private sector is involved in a number of ways in delivering the programs of cities. All use the private sector for construction. Nearly all use private providers of professional services. More than one-third said that private interests are used as program managers. And more than 60 percent use private firms for data collection.

Despite the wide and varied involvement of the private sector, more than a quarter of the cities said that the primary method used to deliver their programs was through public employees or arrangements with other municipalities.

The methods used to contract with the private sector also vary, perhaps reflecting experimentation, as shown in Figure 6.

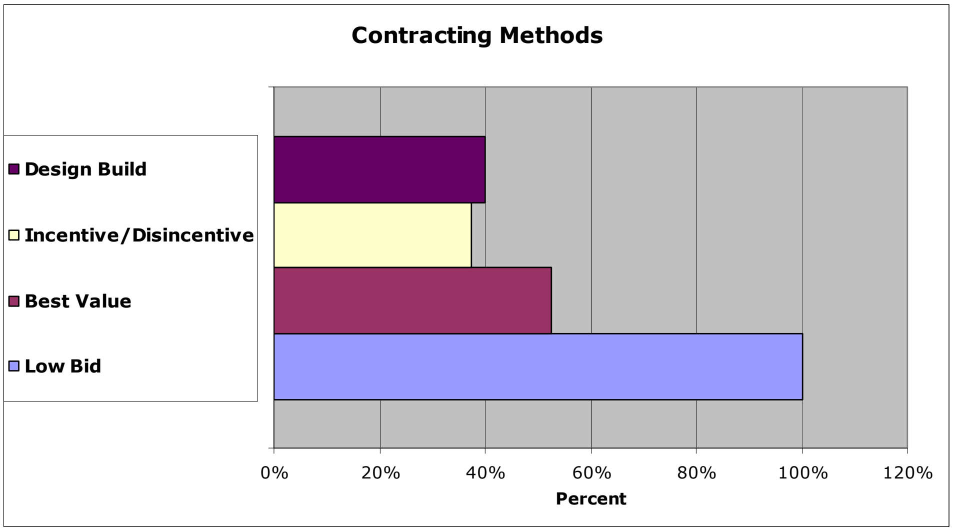

FIGURE 6. Contracting Methods

Program Monitoring

If the key to good program monitoring is good inventory information, the cities seem to be in fairly good shape. Ninety-five percent say that inventories are updated as needed or on a regular cycle.

Similarly, if the measure of program monitoring is the use of inventory information to evaluate condition, cities are doing well. All report that inventories are used in such a manner. Fifty percent say they monitor progress toward goals. Thirty five percent say they regularly evaluate current condition. And fifteen percent say they evaluate current condition on an irregular basis.

Overall Assessment

Based on the responses to questions about each of the areas of asset management, we can see something of a mix of successes and challenges. Successes include good approaches to defining strategic goals, good data systems and good use of data in evaluation and monitoring. On the side of challenges, strategic thinking may not be getting to the decision stage, as we see "worst-first" criteria still widely used and some weaknesses in the overall coordination of the agencies.

How city officials view their management processes seems to be more positive. Self-assessment questions provided insight in three areas:

First, if improvement strategy fitting the guidance of senior management is a measure of the success of the program, city officials feel they are doing well, with 80 percent saying that guidance is reflected very well or perfectly.

Second, only twenty-five of the forty said that they felt someone in their organization might want additional training on asset management. This suggests that fifteen are fairly content with their current levels of knowledge.

Finally, the cities were asked how efficiently their management systems work. Ninety percent of the respondents seem happy with the overall efficiency of their systems, as shown in Figure 7.

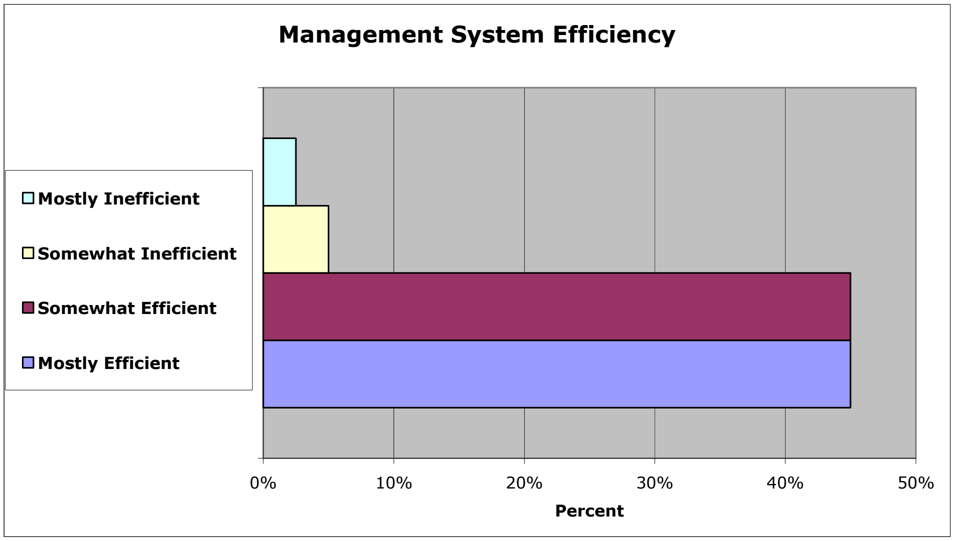

FIGURE 7. Management System Efficiency

CONCLUSIONS

Several conclusions can be drawn from the above. Most suggest the need for significant work to make asset management techniques a reality for local governments.

- Cities have made progress in the use of sophisticated data collection, storage and analytic systems. They also use some very sound process, strategic planning and public involvement, to define system performance objectives.

- At a most basic level, confusion exists as to what asset management really is or ought to be. Too many city officials seem to equate it with automated data collection and storage systems. While data is critical to effective asset management, the softer issues of how the organization is brought together to use the information provided by data systems seems not to be emphasized in most cities. Informal or no tie between the people who build and the people who maintain the systems seems common. Similarly, informal arrangements between those who plan the investments on the system and those who actually construct them seem to be the norm. In at least one city, asset management must be equated to the Governmental Accounting Standards Board (GASB) Statement No. 34, *Basic Financial Statements and Management's Discussion and Analysis for State and Local Governments*, since the respondent said that it was something housed in the finance department.

- While cities use the private sector in many ways—construction, professional services, data collection, etc.—and many use a range of contracting procedures, from low bid to best value, more than a quarter said the primary method of program delivery was through public employees or contractual arrangements with other municipalities. This suggests that many may not be seriously trying to find the most efficient delivery methods available.

- Overall nearly 35 percent of respondents characterized their investment decision-making process as “worst-first.” Nearly 40 percent of those who said they had asset management systems claimed "worst-first" as their decision-making process. This suggests that the strategic or systems approach to investment decision-making is not well understood or used.

- Ninety percent of the respondents characterized their asset management approach as either "mostly efficient" or "somewhat efficient." In addition only twenty-five of forty said that they thought anyone in their organization would want training in asset management. These statistics suggest a degree of satisfaction that will make change very difficult.

Demonstrating the Use of Pavement Management Tools to Address GASB Statement 34 Requirements

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ABSTRACT

The field of asset management has emerged in the last few years as an important decision-making framework that incorporates economic and engineering factors and considers a broad range of assets. Its importance to government agencies has increased with the recent initiative by the Government Accounting Standards Board (GASB) recommending that States and local governments improve their annual financial reporting by incorporating the value of capital assets (including transportation infrastructure) into their financial statements. This initiative, referred to as GASB Statement 34, has established guidance in how transportation assets should be reported on financial statements and has established a schedule for compliance with its recommendations (Federal Highway Administration 2000).

GASB recommends that government agencies use a historical cost approach for capitalizing long-lived capital assets; however, if historical information is not available, guidance is provided for an alternate approach based on the current replacement cost of the assets. A method of representing the costs associated with the use of the assets must also be selected, and two methods are allowed by GASB. One approach is to depreciate the assets over time. The modified approach, on the other hand, provides an agency more flexibility in reporting the value of its assets based upon the use of a systematic, defensible approach that accounts for the preservation of the asset.

Agencies that have implemented pavement management systems are well positioned to use the modified approach to address the GASB Statement 34 guidelines. This paper illustrates the use of a pavement management database to determine the replacement value of an agency's pavement assets using both the depreciation and modified approaches. The advantages and disadvantages of both approaches are also presented.

Key words: asset management—GASB Statement 34—pavement management

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INTRODUCTION

The recent introduction of the Governmental Accounting Standards Board's (GASB's) Statement 34 is having a dramatic impact on the financial reporting requirements of state and local governments. Introduced in June of 1999, this provision recommends that governmental agencies report the value of their infrastructure assets in their financial statements. The implementation of this provision is being conducted in phases, with the reporting of all infrastructure assets receiving major improvements since the effective date of the Statement being required first and then the remaining assets improved after June 15, 1980 being included in later phases.

To satisfy GASB, the value of this infrastructure includes the initial cost of construction and further capital improvements. Additionally, the cost of using the asset should also be included in the financial reporting. GASB Statement 34 allows these costs to be reported using either a depreciation method or a modified/preservation approach. Alternatively, a combination of the two approaches can also be used. For instance, an agency may choose to use the modified approach for its pavement network and the depreciation approach for its storm water system.

OBJECTIVE

The concept of this paper is to investigate the use of information from a standard pavement management database to support the requirements of GASB Statement 34 for valuing the pavement asset. Further, the overall objective is to use the data contained in a pavement management database to determine the replacement value of an agency's pavement assets using both the depreciation and modified approaches. Before determining replacement values, it is advantageous to examine the details of the two valuation approaches and the use of pavement management information to support the chosen valuation approach. With a portion of a sample City's pavement management database selected for use in examining the valuation approaches, the depreciation and modified valuation approaches are applied. After the calculation of replacement value using both approaches, the advantages and disadvantages of each approach are discussed.

Valuation Approaches

The depreciation approach follows a more traditional accounting approach to reporting the annual cost of using capital assets. Under this approach, the historical cost of an asset is reduced each year by an amount equal to the total cost divided by the service life to determine its book value. Alternatively, if the government agency can show that the assets are being managed using asset management tools and the government can document that the infrastructure assets are being preserved at, or above, a condition level originally established for the assets, a different approach may be used. To be considered for the alternate approach, referred to as the modified approach, the asset management system should include the following (*1*).

- An up-to-date inventory of its assets.
- The conduct of an assessment of asset conditions at least once every 3 years using a repeatable procedure.
- An estimate of the annual amount required to maintain and preserve the infrastructure assets at the condition level originally established for the assets.
- An ability to demonstrate that the assets are being preserved at the level predetermined by the government.

The advantages and disadvantages of using the modified approach versus the depreciation approach will be discussed later.

Use of Pavement Management Information to Support GASB Statement 34 Requirements

Pavement management systems, such as Micro PAVER, provide a variety of information needed to conduct a pavement valuation. The information available from the pavement management system database typically includes inventory information such as section identification, surface type, pavement quantity (in terms of area), and functional classification. Also commonly provided in the database are work history and age information in terms of last construction date and construction cost. Condition data are also available.

All of this information is utilized in determining a value for the pavement network when using either the depreciation or modified approach. Further customized portions of the database such as performance curves, treatment rules and cost are used when implementing the modified approach in order to predict needed investments for maintaining the network at a predefined condition level.

Network Subsections

In this assessment, two subsets of the city's road network are examined in order to exemplify the depreciation and modified methods for pavement valuation. The first subset is a portion of Main Street, which is detailed on a section-by-section basis. This subset was chosen to illustrate the level of detail that can be used in calculating pavement valuation. The second subset, which is less detailed, includes only pavements listed as collectors with last construction dates after June 15, 1980.

Main Street Subsection

The portion of Main Street that is examined is detailed in Table 1. As shown in the table, the sections on this portion of road have a variety of construction dates and work types. This will help illustrate the detailed valuation of the pavement.

TABLE 1. Main Street Section Information

Branch	Section ID	Last Construction Date	Construction Type	Surface	Area (ft^2)	PCI at Last Inspection
Main Street	100	11/04/1978	Initial	PCC	16,880	57
Main Street	200	11/04/1986	Initial	PCC	8,480	64
Main Street	300	8/21/1995	Rehabilitation	AC	44,640	95
Main Street	400	10/1/1996	Rehabilitation	AC	30,840	96
Main Street	500	8/21/1996	Rehabilitation	AC	21,880	93
Main Street	600	10/12/1999	Rehabilitation	AC	30,600	99
Main Street	700	10/12/1999	Rehabilitation	AC	17,120	98

Collector Network Subsection

From the Micro PAVER database, the weighted average condition of the second subset, the collector network, was determined. The condition values associated with each age category of the collector network are shown in Table 2.

TABLE 2. Age and Condition of the Collector Network

Age at Inspection	Pavement Area (ft^2)	Weighted Age	Weighted Average Age at Inspection	Weighted Average Condition	Average Condition at Inspection	Average Condition (Dec. 2002)
0–2	653,427	5.4	1.2	90.3		
3–5	500,633	6.4	4.1	89.9		
6–10	1,978,640	10.7	8.4	77.1	71	68
11–15	2,359,289	14.9	12.9	65.4		
16–20	1,589,770	19.1	17.2	64.3		

These portions of the pavement network can be used to illustrate the depreciation and modified approaches for valuing the roadway system. Both methods are displayed in the following sections.

METHODOLOGY

Depreciation Approach

Example: Main Street Subsection

The first example of the depreciation will be shown using the Main Street subsection. Of the seven sections of Main Street that are shown in Table 3, section 100 will not be included in the depreciation example because its construction date is prior to the June 15, 1980 GASB cut-off date.

The first step to finding the value of these assets is to compute the cost of replacing the assets in current dollars. It is important to note that only the work done since 1980 is included in the replacement cost. If a section has been reconstructed its full reconstruction cost is calculated as the replacement cost. However, if the section was simply overlaid since 1980, according to GASB only the price of the overlay is calculated in the replacement cost.

The assumed unit replacement cost of each section as shown in Table 3 was chosen for ease of calculation. The cost of reconstruction has been valued at \$3.00/ft^2 for the concrete section. The replacement cost for the remaining sections is shown as \$0.50/ft^2, assuming a substantial mill and inlay was used as the preferred replacement treatment. The total replacement cost for each section is computed and shown in Table 3.

TABLE 3. Depreciation Approach Applied to Main Street Subsection

Branch	Section ID	Last Construction Date	Area (ft^2)	Replacement Cost per ft^2	Total Replacement Cost (\$)
Main Street	100	11/04/1978	16,880	N/A	N/A
Main Street	200	11/04/1986	8,480	\$3.00	25,440
Main Street	300	8/21/1995	44,640	\$0.50	22,320
Main Street	400	10/1/1996	30,840	\$0.50	15,420
Main Street	500	8/21/1996	21,880	\$0.50	10,940
Main Street	600	10/12/1999	30,600	\$0.50	15,300
Main Street	700	10/12/1999	17,120	\$0.50	8,560

The next step to the depreciation method is to determine the historical cost, which converts the current cost of replacement to the costs at the time the construction took place. The historical cost is calculated using equation (1).

$$Cost_{historical} = Cost_{replacement} * CCI_{year\ of\ construction} / CCI_{year\ of\ calculation}\ , \quad (1)$$

where

$Cost_{historical}$ = historical cost
$Cost_{replacement}$ = replacement cost
$CCI_{year\ of\ construction}$ = construction cost index for the year the pavement was constructed
$CCI_{year\ of\ calculation}$ = construction cost index for the year in which the costs are calculated

Table 4 displays the calculation of the historical cost. Included in the table are CCI values for each section. These values are based upon a yearly average as reported by *Engineering News Record* (ENR).

TABLE 4. Historical Cost of Each Section of Main Street

Branch	**Section ID**	**Total Replacement Cost ($)**	$\mathbf{CCI}_{year\ of\ construction}$	$\mathbf{CCI}_{2003}$	**Historical Cost ($)**
Main Street	100	N/A	N/A		N/A
Main Street	200	25,440	4,295		16,323
Main Street	300	22,320	5,471		18,242
Main Street	400	15,420	5,650	6,694	13,015
Main Street	500	10,940	5,650		9,234
Main Street	600	15,300	6,060		13,851
Main Street	700	8,560	6,060		7,749

After the historical cost of each section is calculated, the book value of each section can be determined using straight-line depreciation. This approach is based on the assumption that a road is expected to last a certain length of time and that the value of the asset depreciates equally each year that the pavement is in service. The equation used to calculate the book value is shown in equation (2).

$$Cost_{book} = Cost_{historical} * (Life - Age) / Life \quad (2)$$

where

$Cost_{book}$ = book value
$Cost_{historical}$ = historical cost
Life = expected life of the pavement
Age = actual age of the pavement

Table 5 provides all the details used in the calculation of the book value of each section of Main Street.

TABLE 5. Book Value of Each Section of Main Street

Branch	Section ID	Historical Cost ($)	Expected Life	Actual Age	Book Value ($)
Main Street	100	N/A	N/A	N/A	N/A
Main Street	200	16,323	30	16	7,617
Main Street	300	18,242	15	7	9,729
Main Street	400	13,015	15	6	7,809
Main Street	500	9,234	15	6	5,540
Main Street	600	13,851	15	3	11,080
Main Street	700	7,749	15	3	6,199

The Main Street example illustrates the value calculation when all construction details (such as last date of construction, last treatment type, and cost of last treatment type) are known. However, it is possible that an agency might have information that is somewhat incomplete or inaccurate, so, it is more likely that a less detailed analysis will be used if the depreciation method is chosen. The same principles apply if only average values are used to calculate the pavement valuation instead of completing detailed section-by-section calculations, as illustrated in the next example.

Example: Collector Network Subsection

This example includes only those pavements ranked as collectors in the sample City's database that have last construction dates from June 15, 1980 to the present. If no detailed construction history is available, an assumption concerning the construction date, type, and cost must be made. For this example, it is assumed that all sections have been reconstructed in the past 23 years and have a unit replacement cost of $2.00/ft^2. Table 6 provides the details of the collector network used to determine the book value using the depreciated value at constant dollars. With the total replacement cost calculated, the historical value can be easily calculated by multiplying the replacement cost by the ratio of the CCI of the construction year to the current year. Then a straight-line depreciation is applied to the historical value in order to calculate the book value. Note that pavements older than 20 years are not considered since they were constructed before 1980.

TABLE 6. Book Value of Collector Network Using Depreciated Value

Age at Inspection	Pavement Area (ft^2)	Weighted Age	Total Replacement Cost ($)	$CCI_{\text{year of construction}}$	Expected Life	Book Value ($)
0–2	653,427	5.4	1,306,854	5,825	25	891,566
3–5	500,633	6.4	1,001,266	5,620	25	625,422
6–10	1,978,640	10.7	3,957,280	4,835	25	1,634,947
11–15	2,359,289	14.9	4,718,578	4,406	25	1,254,733
16–20	1,589,770	19.1	3,179,540	4,066	25	455,782
			Total			4,862,450

The total book value of the system using the depreciated value is therefore determined to be $4,862,450. This approach illustrates the use of network-level information to approximate a book value for reporting purposes. As stated earlier, this approach might be used if complete historical records are not available in the pavement management database.

Modified Approach

The modified approach allows the City the opportunity to set its own benchmark standards for maintaining its pavement network and preserving the value of the pavement assets. However, these standards not only have to be met but they must also be disclosed to the public and therefore warrant careful consideration. With this approach, the book value is preserved rather than depreciated.

To illustrate the modified approach, the collector network is again utilized. The first step in using the modified approach is to set the benchmark condition standards. Table 7 displays the current condition of the network according to Micro PAVER and it also displays the condition standards established by the agency for the collector network. The City has complete flexibility in setting the condition standard and should target a condition that is high enough to be maintained at the current and anticipated funding levels. In some cases, agencies have chosen to establish different benchmarks for different functional classifications of roads. In this example, a condition standard of 65 was set for the entire network to illustrate the use of these concepts.

TABLE 7. Condition Benchmarks for the Collector Network

Functional Class	Pavement Condition Index for December 2002	Proposed GASB Benchmark Condition Standard
Collector	68	65

Using the information that is customized in Micro PAVER such as performance curves, treatment sets, and corresponding maintenance and rehabilitation (M&R) rules, the modified approach can be quickly applied. Using the M&R planner in Micro PAVER 5.0, the portion of the network that is going to be analyzed can be defined, the timing of the analysis can be specified, and the mode for creating the plan can be detailed. Micro PAVER 5.0 has the ability to specify a maximum number of iterations to reach a desired condition level. Therefore, applying the GASB modified approach is as simple as having the Micro PAVER database customized to meet your needs and applying the M&R planning module to reach the desired benchmark condition.

Using the M&R planner, the funding level needed over the next 5 years to achieve the targeted condition level is determined. For this analysis, the approximate amount of money that must be spent on the collector network to maintain a condition of 65 is $450,000 per year. By comparing this figure to the agency's expected budget levels, the agency can quickly assess whether sufficient funding is being allocated to meet the desired conditions.

In comparison, if the targets had been set to maintain a condition of 68 over the next 5 years, the budget requirements are $585,000 for each year. Therefore, an additional cost of $135,000 per year is necessary to raise the condition to 68 from 65. Micro PAVER easily allows for the comparison of various condition scenarios to determine funding levels.

In addition to determining the budget requirements needed to maintain the desired condition of the collector network, Micro PAVER can be used to report the details of the estimated condition during each of the years of the analysis. This result for maintaining a condition of 65 is displayed in Figure 1. This type of report demonstrates that the targeted conditions will be achieved if the budgeted amount is expended on the collector network. An agency could superimpose the actual conditions based upon their current expenditure levels to illustrate their success at meeting or exceeding the targeted pavement condition.

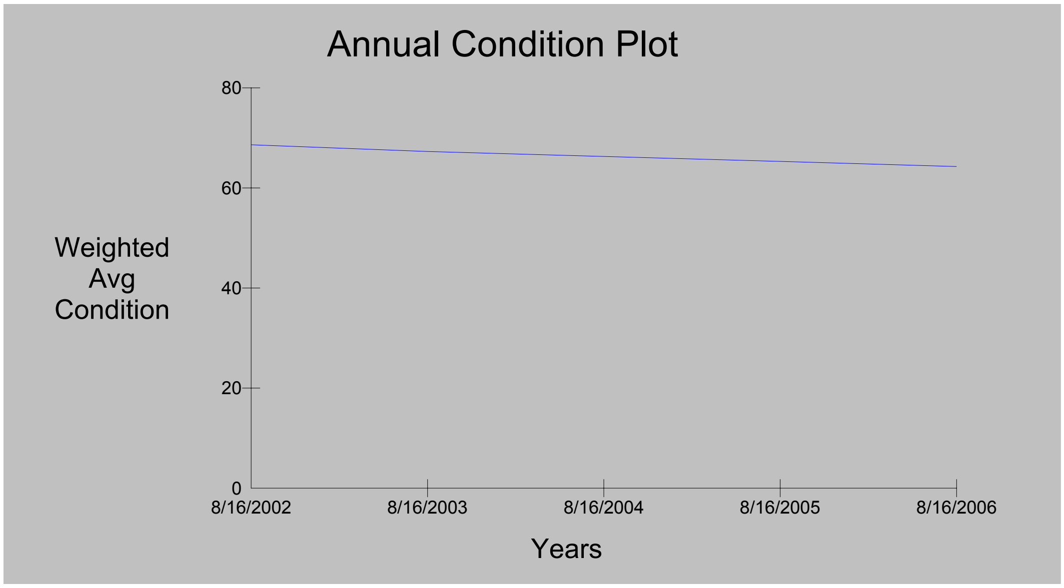

FIGURE 1. Annual Condition of the Collector Network for the Analysis Period

When determining the book value for the collector network using the modified approach, the City is free to use any reasonable and rational approach as long as the method is documented and is consistent. For example the book value may be listed as simply the replacement cost of the collector network, which was determined to be $4,862,450 as shown in Table 6.

RESULTS

Using the depreciation and modified approaches, portions of the example City's pavement network were valued. These examples were highlighted to show the ease of using data from a pavement management system to comply with GASB Statement 34 requirements. By demonstrating each of the approaches that can be used, the advantages and disadvantages associated with each approach can be determined. A summary of key advantages and disadvantages to the use of the modified approach is provided.

Advantages

There are a variety of advantages for an agency in using the modified valuation approach over the depreciation approach to meeting GASB Statement 34 requirement. Some of these advantages are described in the following (*2, 3*):

- The modified approach can be easily used to satisfy GASB Statement 34 requirements if an agency has a fully functioning pavement management system because the system is likely to contain all the needed information in its database. The pavement management system can also be used to easily supply supporting documentation of condition and funding information in the form of reports and tables.
- The modified approach shows the value of maintaining the roadway asset. Using this approach further promotes the importance of preventive and routine maintenance.

- The modified approach indicates the cost of actually using the asset in a given time frame. It provides the replacement cost, whereas, the depreciation approach provides the remaining cost of an asset which doesn't reflect the actual cost of using the asset.
- The modified approach provides an additional use of the pavement management system, which further justifies the need for the necessary staffing and funding to maintain the pavement management system and its data elements.

Disadvantages

The modified approach is not without disadvantages. Some of the disadvantages of choosing the modified approach over the depreciation approach are provided in the following:

- The depreciation approach follows a more traditional accounting approach than the modified valuation approach.
- The depreciation approach may be easier to implement than the modified approach.
- The deprecation approach allows for the value of an asset to be reduced over time; whereas, the modified approach allows the asset to retain its value if it is maintained at the agency specified level. An agency must, therefore, account for this maintained condition in the form of maintenance money and corresponding condition assessments of the pavement sections.

Recommendations

For those agencies deciding to implement the modified approach, there are still a variety of issues that must be addressed. Listed are additional considerations that must be made prior to implementation.

- With a pavement management system in place, is it advisable to make your Financial Officer aware of that your pavements are being managed in this system?
- For your pavement data, how complete is the inventory? What checks and corrections need to be made to the database?
- What is the information flow between the Public Works department and the Finance Department? How can this flow be improved?
- GASB only requires the reporting of assets having major improvements since 1980. Would the City want to consider reporting all pavement section information?

This is only a partial list of potential questions that must be addressed before utilizing the modified valuation approach. There are additional concerns that will arise during the valuation process.

CONCLUSION

This study illustrated the use of a standard pavement management system to support either the depreciation or modified valuation approaches to satisfy the GASB Statement 34 requirements. As shown, information from a fully functioning pavement management database can provide the necessary data to value a pavement network. The advantages and disadvantages of using the modified approach over the depreciation method are also discussed. Special consideration to the multiple issues associated with the chosen methodology must be given before selecting the approach that best meets the needs of the implementing agency.

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Travel Demand Modeling of Automated Small Vehicle Transit on a University Campus

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ABSTRACT

This study assesses the impact of an Automated Small Vehicle Transit system on the mobility practices of students, faculty, and staff on the campus of Kansas State University (KSU) in Manhattan, Kansas. Automated Small Vehicle Transit (ASVT), also known by the name of Personal Rapid Transit (PRT), is a subclass of Automated People Movers which utilizes small vehicles, typically holding 2 to 4 passengers, on a closed network to transport people directly from their origin to destination without stopping at intermediate stations. Conceptually the system can be pictured as automated taxis or horizontal elevators.

A system to transport people in and about closely spaced activity centers continues to be an unmet need in the North American transportation infrastructure. The concepts of ASVT have been proven in earlier theoretical studies to provide an efficient and effective distributor system in urban areas. Since most universities are laid out to optimize pedestrian travel, most universities are an example of a region of closely spaced activity centers. The density incurred from enabling pedestrian travel results in mobility problems similar in nature to those found in larger urban areas. Parking, pedestrian, and vehicle congestion are common, hotly debated issues. As a test of the theory, ASVT is modeled on Kansas State University to determine what impact, if any, a deployed system would have.

A methodology for comparing the before and after impact of a proposed installation of a new transit system is developed based on concepts derived from traditional highway travel demand modeling. The methodology was extended to include pedestrian, parking, and transit networks. Using this model the existing mobility practices at KSU is compared and contrasted with predicted mobility practices with an operational Automated People Mover as described above. The methodology allows vehicles, pedestrians, and transit to interact and compete for mobility choices in a manner similar to traditional Highway Travel Demand Models. A transportation demand model is constructed using data provided by the University Registrar, University Parking Authority, Fiscal Office, and building usage statistics for major attractions such as the library, student union, etc. The multi-mode model includes a network that reflects the necessary interaction between vehicles, pedestrians, and parking as well as transit components. The model

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was iterated over a range of specifications such as extent of system (geographical and node spacing), speed, and capacity. The primary objective was to report on the qualitative impact that a demand-responsive, small, automated transit system could provide to relieve parking congestion, increase campus mobility, and make remote portions of campus more accessible.

Key words: mobility practices—people mover—transit

INTRODUCTION

The objective of the study was to assess how the implementation of a Personal Rapid Transit (PRT) system would affect the mobility practice at Kansas State University (KSU) in Manhattan, Kansas. The study was funded through a research grant from the Kansas Department of Transportation, the primary highway authority in the State of Kansas. The objective was not to determine the technical viability of such a system, but rather, assuming that such a system were viable and commercially available, we wanted to assess PRT's potential for enhancing transportation on campus by resolving many of the long standing deficiencies of the current system. Additional information about the PRT mobility concept is provided in 'Supplemental Information' at the end of this report.

Although tools exist to simulate the movement of people within various travel modes, modeling tools to predict changes in mobility practice before and after a transportation system is enhanced with a new travel mode are lacking. Mode choice decisions are rarely based on a single link, but rather in light of the entire day's travel schedule. Through the implementation of concepts from traditional highway travel demand modeling extended for pedestrian, parking, and transit networks, the existing mobility practices of faculty, staff, and students at KSU are compared with predicted mobility practices with an operational automated people mover (APM) integrated into the transportation system. The methodology allows for various travel modes such as vehicles, sidewalks, and transit to interact and compete for mobility choices in an integrated fashion.

In this study a network model and trip tables are constructed using data provided by the University Registrar, Human Resources, University Parking Authority, and Fiscal Office, and building usage statistics for major attractions such as the library, student union, etc. The multi-mode model includes a network that reflects the necessary interaction between vehicles, pedestrians, and parking as well as transit components. The model is iterated over a range of APM specifications including extent (geographical and station spacing), speed, and boarding delay. The primary objective was to report on the qualitative impact that a demand-responsive, small vehicle, automated transit system could provide to relieve parking congestion, increase campus mobility, and make remote portions of campus more accessible. This in turn would determine if the PRT concept provided substantial enough benefits to warrant additional study.

The last section of this paper entitled 'Model Results and Analysis' is a discussion of the model output. This section presents the base level data comparison of the before and after scenarios and also tries to capture aggregate measures of system effectiveness. One such measure draws from the 'Just-In-Time' delivery concept in which both mean and variance of travel time are considered. Another measure examines accessibility using iso-chrono plots to reflect enhanced mobility.

STUDY AREA

Manhattan, Kansas, home of Kansas State University, is located in the mid-west United States. The main campus of Kansas State University has a land area of 664 acres. The total student population enrolled (on campus only) in Fall 2001 was 21,929. The total number of faculty and staff in the fall of 2001 was approximately 2,200. On-campus population including students, faculty and staff accounts for approximately fifty percent of the population of the city of Manhattan. The Manhattan community encompasses approximately 11 square miles and has an estimated population of 44,800 as of the year of 2001. The City's population for the year 2010 is projected to be 56,539. A map of the KSU campus is shown in Figure 1.

University communities possess unique travel characteristics. Highway planners have long acknowledged that many of the trip generation formulas that rely on socio-economic data do not produce valid results in university settings. Level of income, number of vehicles, and employment status cease to be valid indicators of the type of travel behavior. Additionally, communities of approximately 50,000 residents, such as Manhattan, rarely possess the housing density to consider any type of fixed route public transit. However, university policies intended to sustain a pedestrian campus atmosphere combined with the general propensity of students to live in group settings have resulted in population densities in these relatively small university communities that are reflective of densities seen in larger urban cores. These higher housing densities in turn produce strain on the transportation system disproportionate to the community's size.

Often cited deficiencies in the university's transportation system include the proximity and amount of parking, parking policy, pedestrian congestion, pedestrian and vehicle interaction, and the need to maintain an easily accessible campus. Pedestrian oriented campuses typically try to maintain a ten-minute access rule to accommodate class schedules. This access policy infers that the distance between any two places on campus can be traversed by foot in ten minutes or less. As campuses grow and expand, the ten-minute access rule becomes increasingly difficult to accommodate, causing difficulties in class scheduling.

When this study was initiated in 2001, KSU was in the midst of a bussing study with the intent of initiating a new bussing system. It was in this atmosphere that we approached KSU with the concept of modeling an APM system on campus to assess the qualitative impact to campus mobility. We were both surprised and pleased by the warm reception shown by many campus officials. Many officials readily volunteered insight into parking issues, the conflicting requirements between a pedestrian campus and a vehicle oriented society, and the various strategies that the university has considered to meet these conflicting objectives. As a result of their openness an eagerness to explore alternative solutions, much of the data needed for the construction of the model was easily obtained.

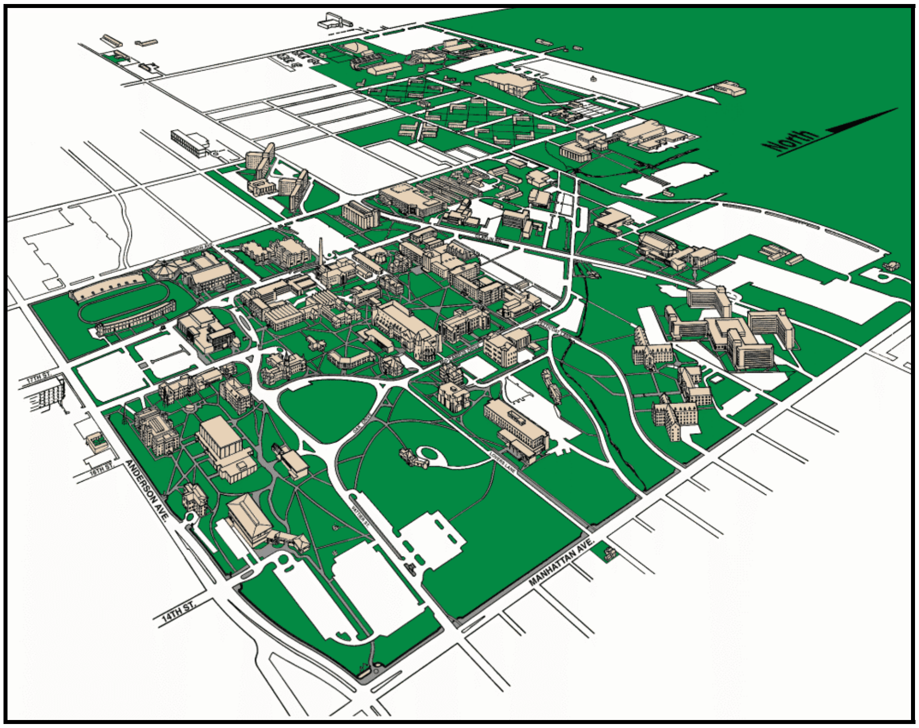

FIGURE 1. A Map of the Kansas State University Main Campus

MODEL CONSTRUCTION AND METHODOLOGY

The modeling process consisted of the construction of a link and node network to represent the various transportation modes, construction of trip tables that represent the travel demand of students and faculty and staff to and from and among the campus activity centers, and the assignment of those trips onto the network. Each of these processes is described briefly below.

Network Construction

The basis for the model is a simple link-node network. Each link represents some type of travel way, and each node is a type of intersection. In our model a single link-node network is used to represent all types of travel modes. Links are used to represent sidewalks, transit, and parking as well as streets. Each link has attributes that determine the cost, measured in time, for traversing the link. Some links have a designated speed, such as streets and sidewalks, while others have a fixed delay attribute such as for parking or a boarding delay for transit. Speed on street sections range from 15 to 30 MPH. Pedestrian links are assigned 4 MPH. The APM transit links representing PRT are assigned 20 to 30 MPH, consistent with published PRT system characteristics. Boarding and deboarding PRT links (representing PRT stations) were assigned fixed delays. Links can be either one way or two way. Both are incorporated into the model as needed. One way links are used extensively in representing the PRT system. Nodes can also be assigned delay characteristics, but in the current model all node delays are set to zero.

Parking lots are challenging to represent in a simple link-node network. The delay times associated with parking links are a function of its volume-to-capacity ratio. For vehicles entering a parking lot, the delay is at a minimum when the parking lot is empty. Once capacity is reached, a large delay is assigned to deter any further vehicles from entering. The delay function for vehicles exiting a parking lot works in a similar fashion, but in reverse. Parking links are unique in that volume is accumulated in both directions in order to calculate appropriate delays throughout the day related to turnover that occurs. Actual parking on campus is assigned using three basic permits: student, faculty/staff, and open parking. In the model, each parking lot is modeled as a parallel three link system, one for each type of permit. The capacity of each type of parking within the lot was encoded into the appropriate link. A basic link-node representation of a parking lot is shown below.

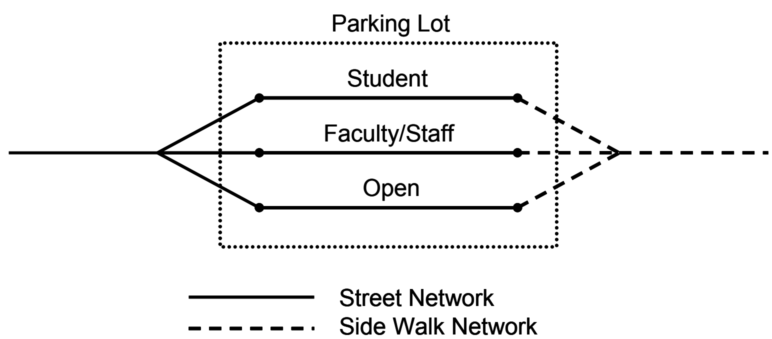

FIGURE 2. Basic Layout of a Parking Lot in the Link-Node Network

The network model includes streets, sidewalks, parking lots, and transit alternatives. A modeled trip consists of the necessary combination of walking, driving, and riding as in a real trip. Although the type of link (street, sidewalk, parking lot, or transit) is recorded in the link's attribute fields, the trip assignment methodology considers only the minimum travel time between a trip origin and destination. Street and sidewalk links were interconnected via parking lots. The final link to all activities centers on campus was a sidewalk, as is the case in a real environment. Similarly, the connecting links to any PRT station were also sidewalk links. A typical PRT station network layout is shown below. The diamond interchange configuration shown in Figure 3 allows vehicles that are not providing service at a particular station to bypass the station without stopping. This is consistent with the off-line station property inherent to the PRT concept.

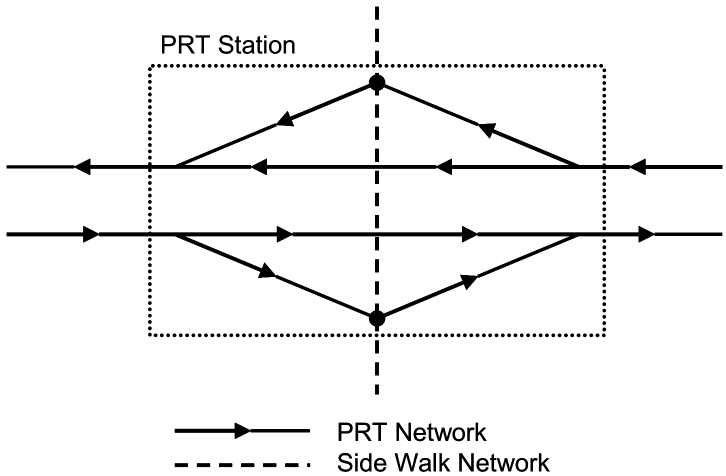

FIGURE 3. Basic Layout of a PRT Station in the Link-Node Network

Trip Generation and Distribution

The fall 2000 line schedule served as the basis to construct student home-to-school and school-to-home trip matrices. The line schedule was cross referenced to student residential files in order to construct trips from a student's residence to their first class, and from their last class back to their place of residence. A complete weekly schedule was averaged to obtain a trip matrix for an average day. A faculty/staff trip table was constructed by matching home addresses to their campus mail drop.

The line schedule was also used to generate building-to-building trips for students throughout the day. If two classes were scheduled back-to-back, a single trip connecting the corresponding classroom buildings was created. If there were one or more hours between scheduled classes, the student was assumed to be likely to make another trip. These student intra-campus trip tables were constructed using a gravity model in which major activities centers such as the library, student union, classroom buildings, etc. competed for student travel options. The amount of time between successive classes was the primary variable in determining likely intra-campus trips. Depending on the length of the gap between classes, the desirability of short versus long trips in the gravity model was adjusted. The longer the time gap between classes, the more likely a student would travel a further distance to another activity center such as a library, recreation center, his or her residence, or place of employment. For instance, suppose a student has a class that ends at 9:20AM and the next class does not start until 1:30PM. During this four-hour gap the model would create a trip to a local attraction such as a major campus activity center or off campus to a work location. Although the authors acknowledge that that there may be errors in this method, the efficiency and reliability of using the class line schedule outweighed the cost and sample errors associated with an origin-destination interview process.

In all, over 72,700 trips were created for a typical day on the University campus. These trip tables served as the basis for assignment onto the network. Similar to other network models, the shortest path (measured in time) is calculated during each iteration of assignment, and trips from the trip tables are loaded onto the network accordingly.

Trip Assignment

Trips are loaded incrementally onto the network in short time increments in order to adjust delays as parking lots fill and empty. The time increments range from a minimum of 1 minute to a maximum of 5 minutes. The model tracks movement of students, faculty, and staff from 6AM to 10PM. Since the trip tables defined travel demand for common class starting times and/or ending times for students and simply journey to and from work for faculty and staff, probability density functions were used to distribute trips from the tables to selected time increments. For example the faculty & staff (F/S) trip table consists of one table for the AM journey-to-work, and one table (identical to the AM except the trip ends are swapped) for the PM journey back home. Since no ancillary information was readily available to guide the assignment of the trip to a particular time, a Gaussian probability density function (PDF) curve with a mean arrival time of 9AM and a 1 hour standard deviation was used to distribute F/S trips to campus in the morning. Similarly, a Gaussian PDF with mean of 5PM and standard deviation of 1 hour was used to distribute the evening F/S trips back to residences. The distribution curve for F/S is shown in the figure 4. For example, the portion of faculty/staff trip table assigned to the network between 8:21AM and 8:26 AM is found from integrating the curve in Figure 4 for the respective times.

The distribution of student trips about the beginning or ending of class uses a non-symmetric distribution. Figure 5 shows the distribution of student trips prior to the beginning of class. In this chart the area under the curve is unity. For example, if the trip assignment for the 7:18 to 7:21 time frame is needed from a trip table showing student home-to-school trips for a class beginning at 7:30, the integral of the curve in Figure 5 from -12 minutes to -9 minutes is applied to the trip table.

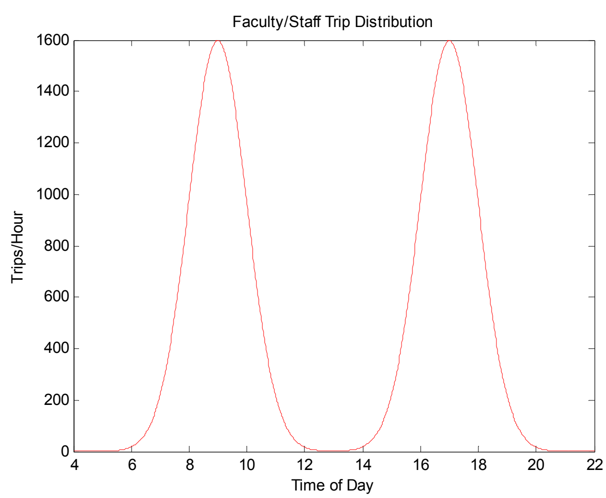

FIGURE 4. Probability Distribution Curve Used to Distribute Faculty and Staff Trips

For any particular time period, all possible trip tables and their respective distribution curves are referenced and accumulated into two tables. One table is for student trips, and the other for faculty/staff trips. These two trip types must remain separate because of the distinction in parking policy. Time increments varied between 1 to 5 minutes. During non-peak travel periods such as early in the morning and later afternoon, five minute increments were used. During peak travel periods the increments were reduced so as not to overload parking lots. If too many trips are assigned in any one-time increment, parking lots can severely overshoot capacity.

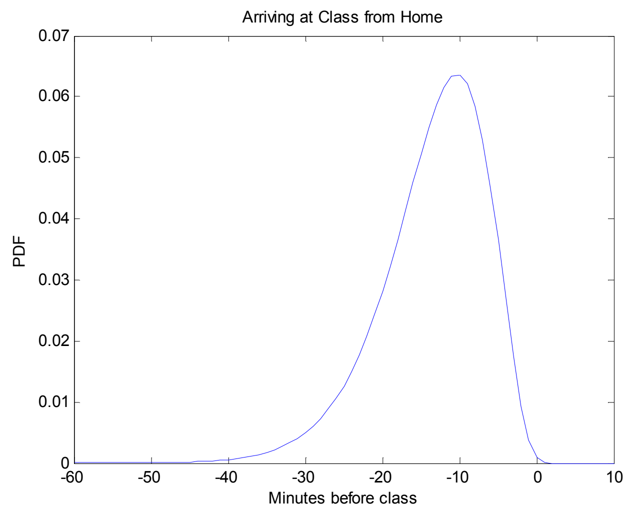

FIGURE 5. Probability distribution curve used to distribute student trips prior to the beginning of class

MODEL RESULTS AND ANALYSIS

The first and most basic PRT network tested was a radial crossing network as shown in Figure 6. It connected remote parking lots to the central core campus where the two lines intersected. Although other PRT networks were tested with more conventional loop configurations, the basic results varied little from the results of this radial network. This reflects that the critical aspect of any APM is the linking of remote parking to central campus activity centers, which was the basis of the initial radial layout.

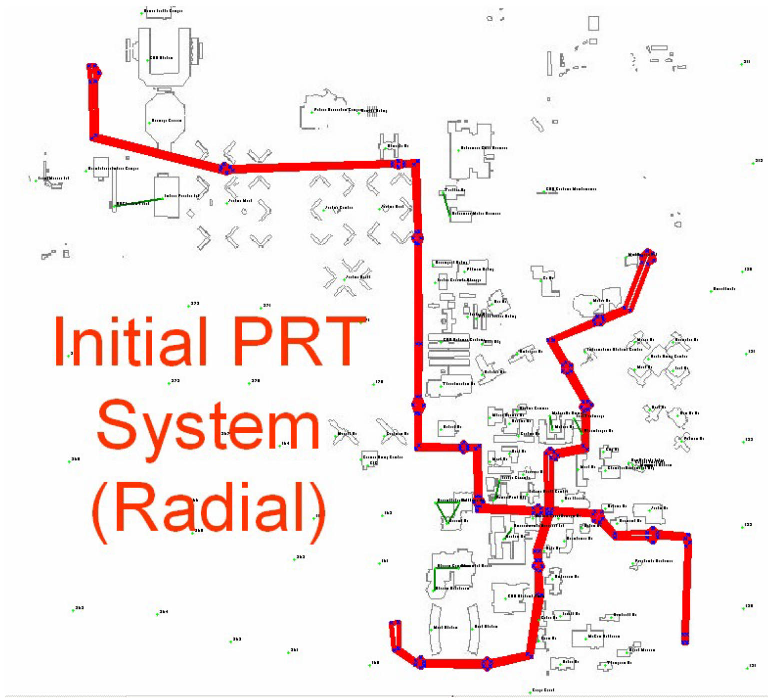

FIGURE 6. The above picture depicts the layout of the radial crossing PRT network

The basic comparison of the transportation system with and without any PRT is shown in the table below. Three transit scenarios were run, each with successively less boarding delay. The three scenarios shown have PRT boarding delays of 6, 3, and 2 minutes respectively. All transit scenarios assume a de-boarding delay of 1 minute. As shown in the chart, as the responsiveness of the PRT increases, about 10,000 person-miles per day are drawn onto the PRT system. Of the 10,000 person-miles drawn onto the PRT system, about half, or 5000 person-miles per day, are vehicle miles drawn off the street network. The removal of 5000 vehicle miles, primarily on the streets adjacent to the campus, would significantly alleviate parking/pedestrian congestion issues.

TABLE 1. Base Level Results for the 't' PRT Network

Person Miles per Day		No PRT	6 Min	3 Min	2 Min
	Driving of the Road Network	40131	38352	36621	35807
	Walking on the Sidewalks	20216	18814	16770	15681
	Riding the PRT	0	3604	8169	10279
	Total	60347	60770	61560	61767

Person Hours per Day		No PRT	6 Min	3 Min	2 Min
	Driving on the Road Network	2014	1927	1842	1805
	Walking on the Sidewalks	5037	4688	4186	3923
	In the Parking Lot	1877	1686	1532	1497
	Riding the PRT	0	458.2	842.6	1035
	Total	8928	8759.2	8402.6	8260

A major challenge in this study was to develop appropriate system metrics. Many of the deficiencies in the current transportation system had been previously analyzed and studied in isolation. For example, specialized parking studies, pedestrian studies, and shuttle system studies have been conducted for KSU. Each study concentrated on a mobility sub-system and provided metrics that are specific to the sub-system in question. The challenge of this study was to assess the transportation system as a whole. Simply showing 'ridership of a transit option' or 'reduction in VMT', though valid metrics, do not reflect well the overall quality of the transportation system in providing mobility service to its patrons. In order to reflect the impact of an APM on mobility practice, a concept from 'Just-In-Time' delivery is used in which not only speed of delivery was important, but also predictability of delivery time was also assessed.

Using a no PRT scenario and a Crossing PRT scenario with a two-minute boarding delay, the mean and variance of the travel time are tracked between major travel nodes in the network. Assessing the variance of travel time throughout the day would indicate any significant decrease in the variability (or increase in predictability) of anticipated travel time. This analysis can be performed between any two locations in the model. For demonstration purposes, a trip between a node on the western edge of the network and Hale Library is chosen. Hale Library is located in the middle of the campus academic area and is considered the academic center of undergraduate education. The trip is representative of travel demand from an off campus location to the campus core. Figure 7 tracks the travel time between these two nodes throughout the day. As seen in chart below, the primary benefit of a PRT augmented transportation system is the reduction in variability of travel time throughout the day, avoiding excessive delays during peak travel periods.

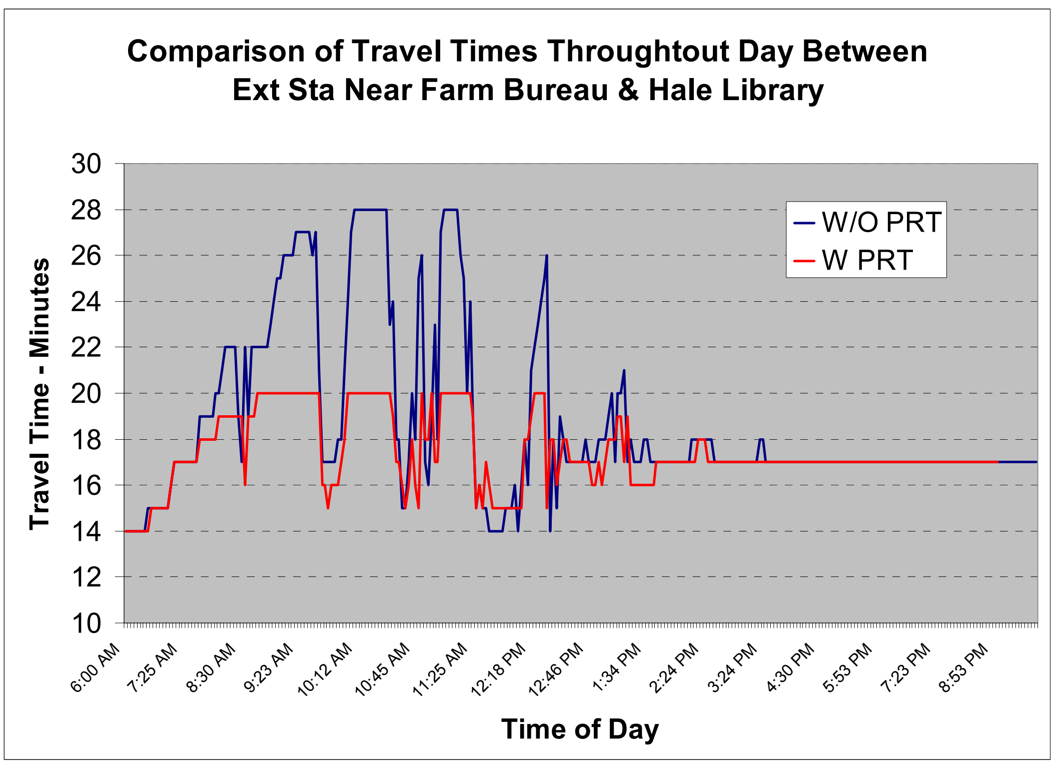

FIGURE 7. The Travel Time Between an External Station and Hale Library Before and After a PRT System is Added to the Network

A simple mean calculation between the scenarios results in 18.4 and 17.1 minutes respectively for the 'No PRT' and 'PRT' options shown in Figure 7. However, the standard deviation of the same data results in 3.97 minutes and 1.84 minutes respectively, a reduction of over a factor of two in variability.

Travel demand varies throughout the day. Figure 8 graphs the rate at which trips are loaded onto the network versus time. This is similar in concept to the AM and PM peak in highway travel demand, but as the graph shows, peak times are distributed throughout the day on a university. The peaks generally coincide with the beginning and ending of class times.

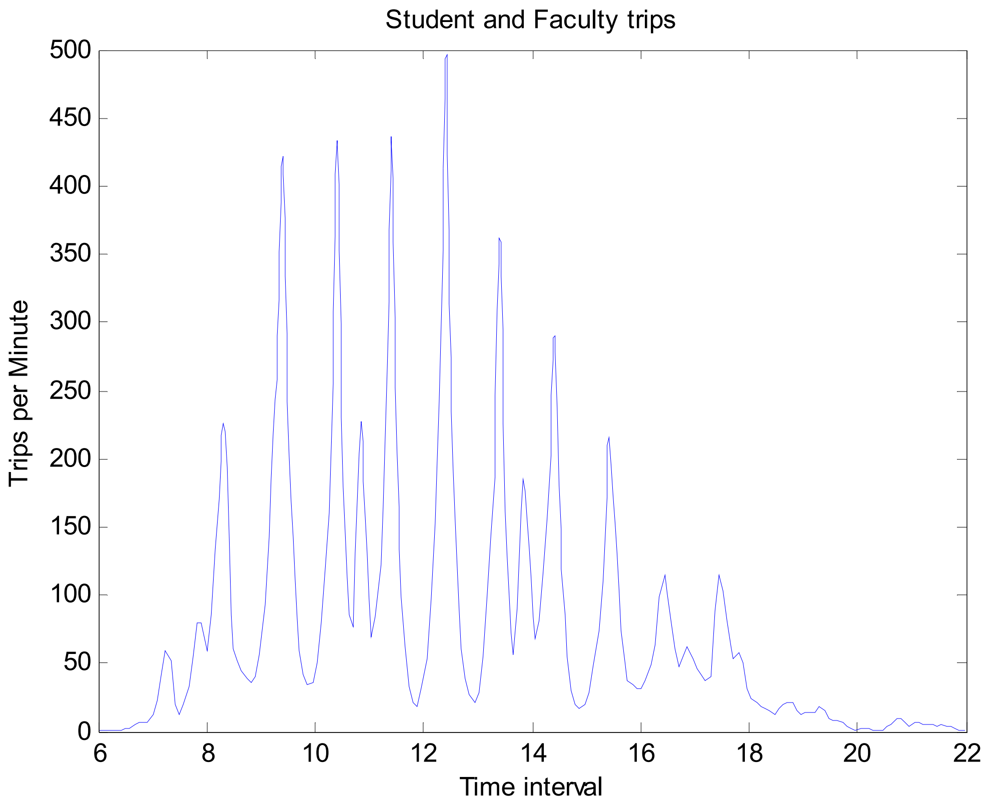

FIGURE 8. The Rate at Which Trips are Loaded Onto the Network Versus Time is Shown Above The Peak Times Coincide with the Beginning or Ending of Regular Class Periods

Trip generation rates, as shown in Figure 8, are used as weighting constants in the calculation of mean and standard deviation. Applying these weighting constants to the calculation of mean travel time between an external node and Hale Library results in a mean of 19.7 and 17.8 minutes respectively for the 'No PRT' and '2 Minute PRT' options, a difference of about 2 minutes. Notice that in both cases the mean increased, but the PRT option increased less. Applying the same weighting concept to the calculation of standard deviation results in 4.08 and 1.06 minutes for the two options, respectively. The standard deviation for the 'No PRT' option increases, while the standard deviation of the PRT option actually decreases when the calculation is based proportionally on travel demand. This results in a 4:1 ratio of variability (or predictability) in travel time between the two scenarios.

Another method that portrays the accessibility of campus is an isochronal (constant time) plot. Again using Hale Library for demonstration purposes, a series of three plots are shown in Figures

9 through 11. Each ring encircles all the nodes in the model that can access Hale Library within the time frame as labeled. In each plot, the outline of the University campus is shaded in light blue.

Figure 9 shows the accessibility of the Library during that early morning hours. At this time, any parking facilities immediately adjacent to the library are available for use. As a result the library can be easily accessed within 10 minutes from an area that encompasses a good portion of the Manhattan community. In contrast, Figure 10 portrays the accessibility of Hale Library at approximately 10AM in the morning, one of the highest peaks in travel demand. At this time the parking facilities adjacent to the library are full and not available for use. As a result, anyone trying to access the library will have to either walk the entire distance, or park at a distant lot and walk a considerable distance across campus. The 5, 10, and 15-minute isochronal lines are shrunk down to the area immediately around the University, depicting primarily what is accessible within walking distance of the library.

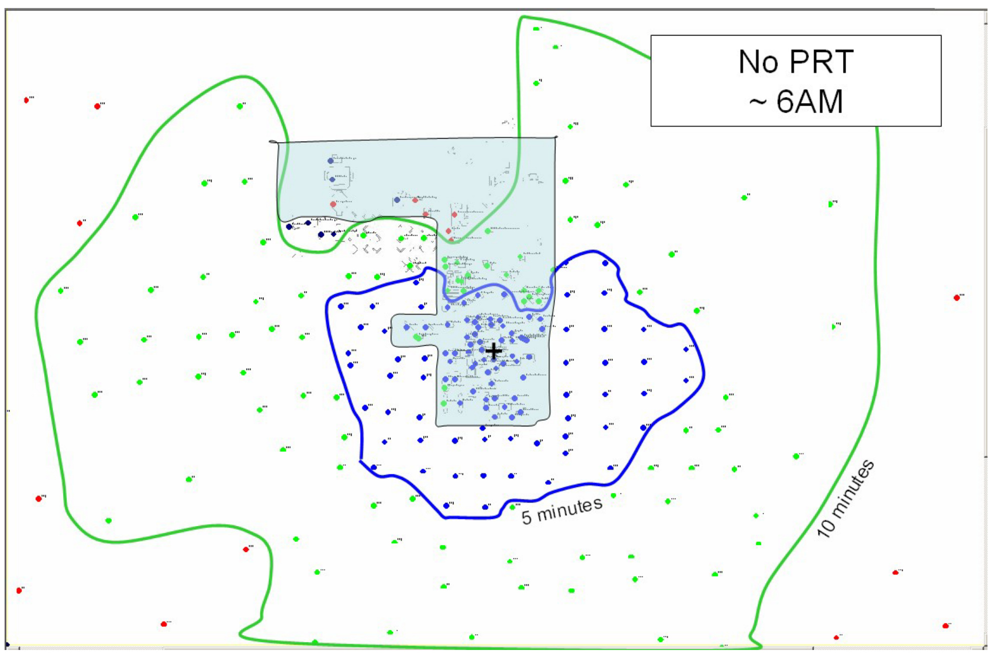

Figure 9. Isochronal Plot Showing Accessibility of Hale Library at 10AM with no PRT

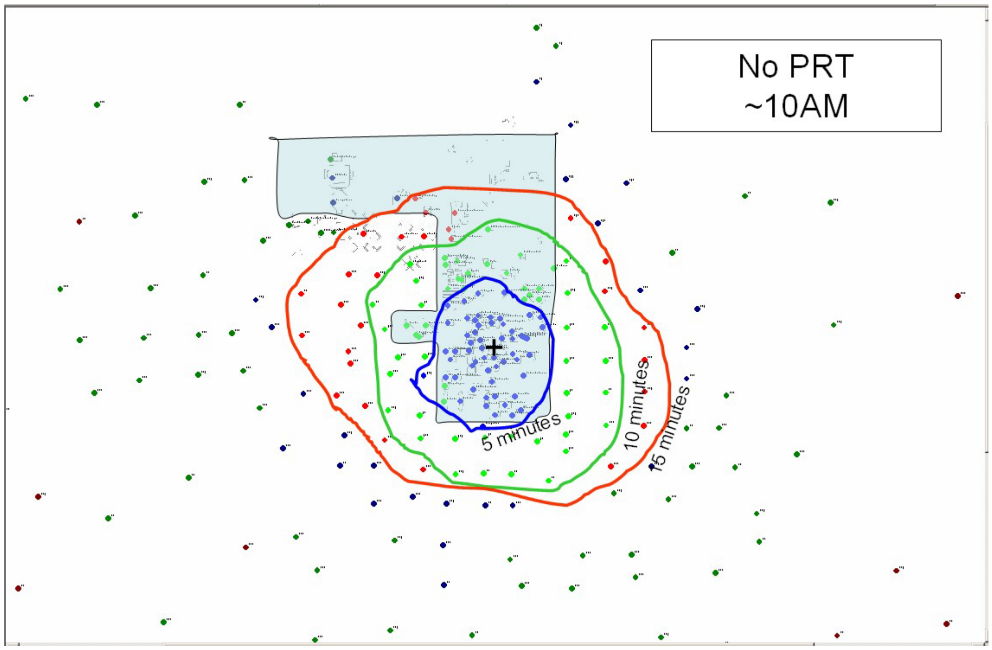

FIGURE 10. Isochronal Plot Showing Accessibility of Hale Library at 10am with Crossing PRT

The last plot, Figure 11, depicts the accessibility of Hale Library near 10AM with the crossing PRT. Highlighted in red are the 15-minute iso-chrono accessibility lines with and without a PRT system. The 'Crossing PRT' analysis shows that the geographical area that can access the library within 15 minutes during peak travel demand is approximately six times greater than the current transportation (no PRT) system. Other PRT scenarios that were tested resulting in area ratios anywhere from 2:1 up to 6:1.

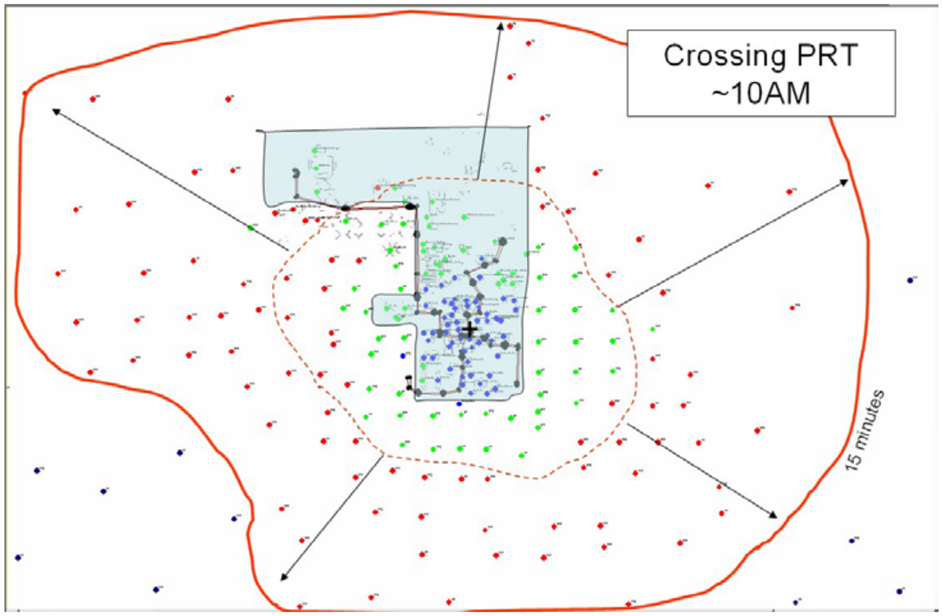

FIGURE 11. Isochronal Plot Contrasting Accessibility with and without a PRT System The Plot Depicts The Expansion Of The 15-Minute Iso-Chronal Line At 10am with the Implementation of a Crossing PRT System

CONCLUSION

The objective of the study was to assess the impact on mobility that an APM would have on a University campus. Although the technical viability of PRT systems has been debated for decades, appropriate tools and analysis techniques to assess the impact of a functional PRT are lacking. Assuming that a PRT is technically viable and operates within the specifications as reported for Taxi2000 and Ultra, our goal was to assess whether a transportation system with an integrated PRT substantially impacts mobility practice. The most notable deficiency in campus transportation systems is summarized in the frequent citation of the 'amount and proximity of parking' being the number one transportation concern. One of the tests of an effective APM is to make parking anywhere on campus equally attractive, or in essence remove the 'proximity' clause from the parking equation.

Using an innovative hybrid, multi-modal network model, the impact of an integrated APM was assessed using regional highway travel demand modeling philosophy. Trip matrices were created using data provided by the University for the fall 2000 semester. Trips were assigned to the network using the shortest path algorithms. Unlike traditional travel demand models that use a modal-split procedure that predicts transit ridership based on socio-economic data, the current model include all modes (streets, sidewalks, parking, and transit) in its base network. Trips are not pre-segregated into distinct modes, but rather assigned to the network links that minimizes overall travel time on a pure utility basis. As a result trips are modeled with the necessary combination of driving, parking, walking, and riding, similar to real-world experience.

System metrics were introduced to assess a transportation system's ability to provide predictable, effective service. Similar to JIT delivery concepts, the first method examines both the mean travel time as well as the variability of travel time throughout the day, weighted by relative travel demand. In these regards, a PRT system was found to reduce variability of travel time by a factor of 4. Accessibility was analyzed using isochronal plots which depicted the relative area that can access a facility within a given timeframe. Using Hale Library in the center of campus, the PRT scenarios that were tested showed an increase in accessible area between two and six times greater than the current transportation system. For the KSU campus the linking of remote parking lots to central activity centers is the primary role of the APM or any other transit option. Because both the radial PRT network and loop concepts all accomplished this primary objective, the results of the analysis did not vary significantly for the PRT options tested.

APPENDIX 1: SUPPLEMENTAL INFORMATION ON PERSONAL RAPID TRANSIT

Background Information

Of the deployed transit systems in the United States over the past 30 years, very few advancements have been made in the basic modes of either bus, light rail, or trains. Although technology has been added to enhance and automate these systems, they remain uncompetitive with automobile travel due to inherent limitations associated with each mode. Personal Rapid Transit (PRT) is a different form of automated transit that goes beyond a "simple electrification" of an existing mechanical concept. Rather PRT starts with a fully automated vehicle and uses communication and computing technology to provide an on-demand, personalized transit service.

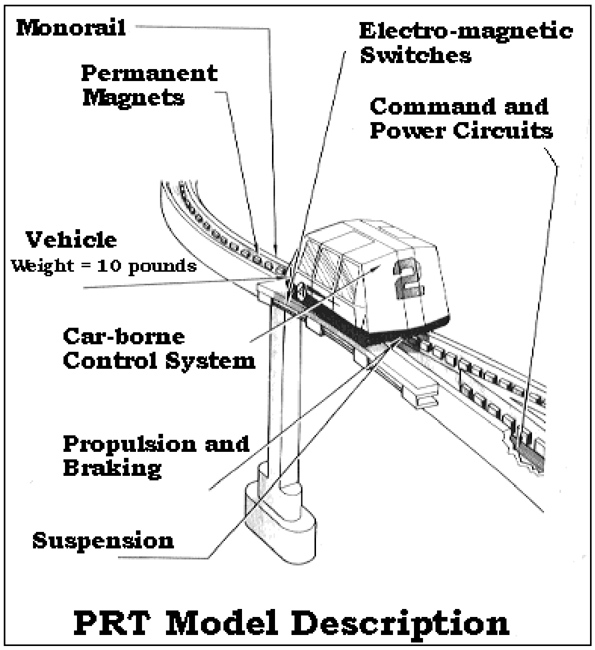

FIGURE 1: PRT Concept Drawing, Aerospace Corporation, 1969

The basic concept of PRT is on-demand service to your destination (without any intermediate stops) using small, completely automated vehicles capable of carrying 2 to 6 people. PRT systems can use rail technology, rubber tires, magnetic levitation, or any other type of transport mechanism. A PRT system is defined by its concept of service, not the technology used to implement it. In many respects the service concept of a PRT system is similar to that of an automated highway system. Others have likened it to a horizontal elevator or an automated taxi service. A concept drawing of one possible PRT system is shown in Figure 1.

Various architectures and design concepts for a PRT system have been proposed over the last 25 to 30 years. A few have made it to prototype stage. The only deployment of a system that attempted to achieve the basic properties of the PRT was on the Morgantown campus of the University of West Virginia.

Constructed in 1970, the Morgantown system is an automated, on-demand system that transports people between five different areas on campus. The vehicles have seats for eight people with room for standees. The Morgantown system has been dubbed as Group Rapid Transit (GRT) due to its vehicle size and limited extend (only five stations). The system can route a vehicle directly from origin to destination without intermediate stops, but operates in this mode only during certain times of the day. The system is still in operation to this day providing for the basic mobility needs of the campus and surrounding community.

In the spring of 2001 the Kansas Department of Transportation approved funding for a study to assess the impact of a Personal Rapid Transit system on a University Campus and surrounding community. The study was timely because both major Kansas Universities were undergoing studies to either expand or create a bussing system. It is generally acknowledged that the mobility needs in and about the campuses are not fully met with existing systems. However, there is great hesitation to expand existing road networks and parking facilities within the existing campus due to adverse societal, visual, environmental, and cultural impacts. This is especially true on the KSU campus where adequate real estate exists to create such structures.

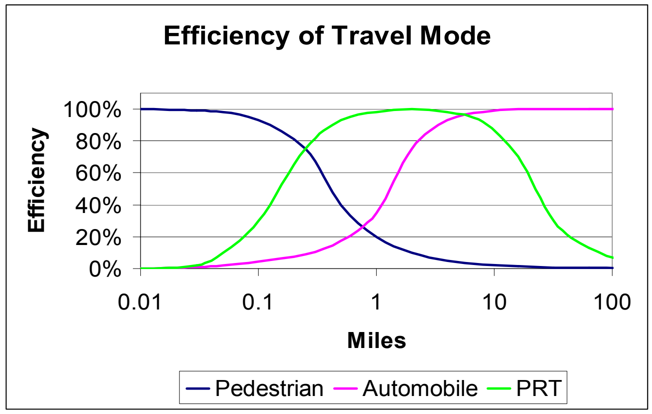

FIGURE 2

The concept of PRT applied to a University Campus is not necessarily to replace automobiles, but rather augment vehicle and pedestrian travel by providing a third mode of transit. Automobile travel is efficient for distances from 2 miles up to about 200 miles. When the distance is less than a couple of miles, the terminal times associated with automotive travel - namely parking - begin to significantly impact the efficiency of the trip. Pedestrian travel is efficient for trips up to about 1/4 of a mile, more or less depending on the person. Trips greater than a 1/4 are time inefficient by foot, plus depending on the environment may produce undesirable exposure to the elements. For University campuses it is typically the distances from 1/4 of a mile up to two miles that automotive travel and pedestrian travel fail to provide a desirable alternative. This is the area that PRT holds exceptional promise to reduce parking, automotive, and pedestrian congestion in and around Universities.

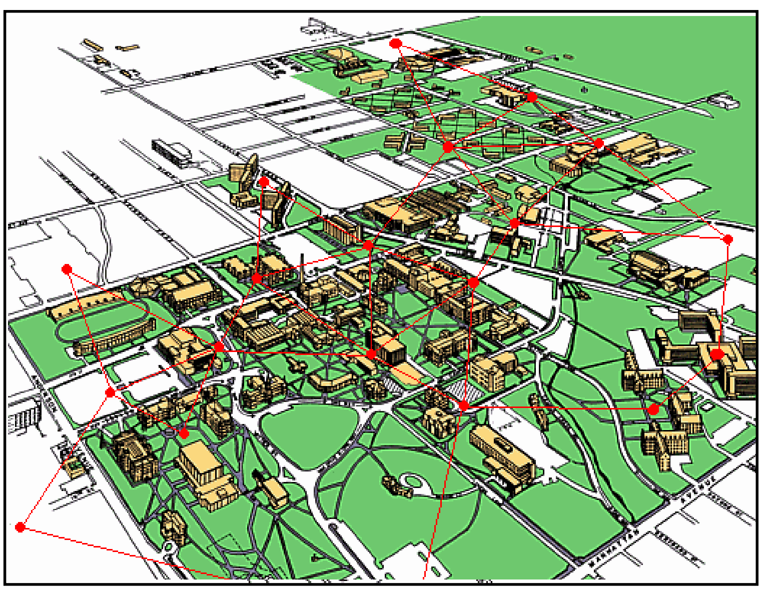

FIGURE 3. Conceptual PRT Network for KSU, Manhattan

The study of PRT for the Manhattan campus emphasizes the potential to increase the mobility in and around campus with the secondary benefits of discouraging students from driving and reducing the link times between campus centers spaced far apart. In this study a PRT system is modeled according to its basic principles of on-demand service between activity centers using small vehicles capable of carrying two to four people. No specific technology is identified. The parameters within the model that determine the capacity and throughput of the PRT system are not specific to any particular technology, but rather chosen to reflect the expected capability of any mature PRT system. A sample PRT network for the KSU Manhattan campus is illustrated in Figure 3.

Fiber Optic Implementation of a Cumulative Momentum Model for Natural Urban Intersection Traffic Management

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ABSTRACT

We propose an agile and cost-effective urban intersection traffic management model based on the competitive cumulative momentum (CCMV) of vehicles. It is to be implemented by an in-ground fiber optic load sensor system. The model will address congestion relief, bus and truck signal priority in order to reduce excessive bus and truck idling and delays, and the prevention of red light running. The cumulative momentum (CMV) values of traffic in cross streets at an intersection are compared, and the higher CMV value commands the GREEN. A related static cumulative mass or weight (CM) model will be employed to manage vehicle traffic in left turn lanes at intersections with two-way traffic, and also pedestrian crossing traffic. The combined CCMV and CM model replicates precisely the way a traffic cop manages intersection traffic. The GREEN will lower the CMV of a street, while traffic in the cross street continues to build. Naturally, when those two discrete values are cross then so will the command of the Green. This competitive process allows traffic flow to alternate at an intersection based on traffic volume. The CM model will determine when cumulative weight in left turn lanes and at the curb of pedestrian crossings should trigger the Green. We also propose a kinetic energy ($KE = MV^2/2$) model to emphasize speed-controlled traffic signal in the case of speeding vehicles that may run a red light. The proposed fiber optic load sensor systems are commercially available. They are capable of performing axle detection and counting, speed measurement, gap measurement, vehicle classification and recognition. These in-ground systems can be readily installed flush with road surface, so as to protect against road hazards. The embedded fiber sensor is vibration proof; has high sensitivity, high elasticity, flexibility and durability, and is non-metallic and thus EMI proof.

Key words: fiber optics—traffic management—traffic signal

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PROBLEM STATEMENT

The inevitable increase in traffic density with urban growth is demanding new ways to monitor, manage, control and optimize traffic flow, and to obtain information for long term planning of transportation infrastructure. Many impact studies have been reported on urban traffic congestion, fuel consumption, pollution, noise and public transit rider-ship woes. Advanced traffic management and intelligent transportation systems have been proposed and developed. Urban Traffic Intersection Management is the essential issue. The task lies in accurate and quick data acquisition and processing of critical parameters at the intersections of traffic arteries. We have proposed to specifically address urban intersection traffic control.

Current traffic signaling technology at intersections in the Triad cities is mainly based on inductive loops, embedded in the pavement at intersection approaches. The loop creates a magnetic field, and when disturbed by a vehicle moving through it, sends a signal to the control box. The system has a short range, and performs mainly vehicle counting. In terms of software, these traffic signals run on fixed clock cycles, so that often traffic in a busy lane is being held up by the "Red", while the low traffic lane has the "Green", creating a potential red light running situation by frustrated drivers.

It appears that public transit in cities of the size of Winston-Salem and Greensboro faces steady decline in rider-ship, to the point that such city service is constantly running in the red. The cause lies in the glaring disparity in convenience between personal and public transportation. In addition, due to frequent stops, public transit buses often miss the green at clocked intersections, leading to excessive travel delays. This further discourages rider-ship. Idling of city and school buses and heavy vehicles at urban intersections also contributes to air pollution and waste of fuel. North Carolina ranked fourth nationally in the number of smog days in year 2000 among industrial states, such as Pennsylvania, Ohio, Maryland, New Jersey. The EPA has just announced a National Clean School Bus Act to reduce pollution of diesel engine emissions.

PROPOSED APPROACHES

To improve urban intersection traffic management, we have proposed an "intelligent" traffic signaling system actuated on demand, patterning after the traffic management style of a "traffic cop" at an intersection. The "cop" must have sufficient vision to view approaching traffic for a rough estimate of traffic flow in cross streets. He/she then made a decision normally to let the lane with heavier traffic volume go first. This process would then alternate according to land demand. Long range detection can be above ground, or in the roadbed. In addition, the algorithms for traffic management must be cost effective, and hence "smart", requiring minimum data acquisition and processing. Much deliberation has been given to the selection of traffic flow parameters to be monitored so as to arrive at a high performance and cost effective system. It must also include Green Preference for public transit and heavy vehicles. Several of these algorithms are now commercially available. However, all these must be preprogrammed into the systems, and hence differ significantly from the on- the-spot free management style of the traffic cop. We have proposed a natural algorithm that mimics the mind of the cop as he/she views the traffic picture at an intersection and makes appropriate decisions. This has resulted in a "Claim to Green" formula. It was an attempt to quantify the traffic cop's thought process. Initially, the formula was based on vehicle length, representing vehicle weight, vehicle speed and vehicle distance from the intersection. The determining factor used for claim to green was the kinetic energy or vehicle length x $(\text{speed})^2$.

Green Demand algorithm raises priority as standing vehicle waiting time increase. For moving vehicles farther than 50 ft from the intersection, Green Demand is defined as the ratio of the kinetic energy of each vehicle and its distance from the intersection. The quantity is then summed over each lane. The nearer, faster and heavier the vehicle group, the higher the Green Demand priority. The algorithm will continuously update the Claim-to-Green in each lane, based on cumulative kinetic energy and cumulative elapsed waiting time on approach.

The Green Demand algorithm as postulated above resembles the mindset of the traffic cop at an intersection. The formula has simply quantified his/her thought process. For instance, vehicles far from the intersection would automatically command a lower Green priority due to the limited vision of the cop. He/she can also estimate vehicle weight by its size, although not exactly. Nevertheless, vehicle size does command his attention, thus increasing the Green Claim. It is also unlikely that a human can readily compute kinetic energy mentally, although a sense of vehicle speed comes rather naturally. Furthermore, squaring the speed so as to compute kinetic energy unduly emphasizes vehicle speed more than weight. For instance, a sedan weighing one quarter that of a bus, will have the same kinetic energy if its speed is twice that of the bus. However, if we were to measure momentum, instead of kinetic energy, then both weight and speed are treated equally. In this case, a bus four times the sedan weight traveling at half the speed will still have more momentum than a light sedan, traveling at twice the bus speed.

The momentum concept is taken one step further to include cumulative momentum of all vehicles measured over a designated time period. This departs form the standard vehicle counting systems. As long as the load sensor registers the weight of the passing vehicle or vehicles. Hence, even two or more vehicles pass over the same load sensor; the cumulative weight will be recorded and counted as a single vehicle. If they have different speeds, then they will not pass the second load sensor at the same time. This scenario clearly illustrates the beauty of the system. It needs to monitor only a minimum of traffic parameters, and should thus be most economical and robust. This system allows the cumulative momentum of cross streets to compete: The higher value automatically commands the Green. We have thus arrived at an intersection traffic signal system that implements the claim to Green algorithm naturally.

We therefore propose an agile and cost-effective urban intersection traffic management model based on the competitive cumulative momentum (CCMV) of vehicles. It is to be implemented by an in-ground fiber optic load sensor system. The model will address congestion relief, bus and truck signal priority in order to reduce excessive bus and truck idling and delays, and the prevention of red light running. The cumulative momentum (CCMV) values of traffic in cross streets at an intersection are compared, and the higher CMV value commands the GREEN. A related static cumulative mass or weight (CM) model will be employed to manage vehicle traffic in left turn lanes at intersections with two-way traffic, and also pedestrian crossing traffic. The combined CCMV and CM model replicates precisely the way a traffic cop manages intersection traffic. The GREEN will lower the CMV of a street, while traffic in the cross street continues to build. Naturally, when the two values cross so will the command for the GREEN. This competitive process allows traffic flow to alternate at an intersection based on traffic volume. The CM model will determine when cumulative weight in left turn lanes and at the curb of pedestrian crossing should trigger the Green. We also proposed a kinetic energy ($KE = MV^2/2$) model to emphasize speed-controlled traffic signal in the case of speeding vehicles that may run a red light.

The CMV or cumulative momentum (MV = product of vehicle mass or weight and its velocity or speed) model sums the momentum of N vehicles counted within a certain zone of a street over a certain time period. The CMV's of traffic of all cross streets at an urban intersection are compiled and compared or "allowed to compete". The street with the higher CMV commands the Green, hence named the competitive cumulative MV or CCMV model. The CMV model automatically favors heavier vehicles, such as buses and trucks, which normally ply the main streets. Hence, main street traffic commands the Green more frequently. Left-turn lane, traffic in two-way streets and pedestrian crossings are included by adding a static cumulative mass (weight) or CM model programmed within the CCMV model. The combined CCMV and CM model replicates precisely the traffic cop, directing traffic at an intersection. The street with the Green will naturally have its CMV lowered, while that of the cross street continues to build. At some point, their CMV numbers will cross, and the Green command changes hand. This competitive process causes the Green signal to alternate naturally and intelligently, since no Green threshold is preprogrammed.

ADOPTED DETECTOR TECHNOLOGY AND ITS IMPLEMENTATION

A portable IRD fiber optic load traffic sensor system will be purchased and tested first at a test site on campus under well-controlled conditions. This is aimed at checking out the system hardware and software for axle counting and speed measurement. System software and hardware must then be modified or reconfigured for momentum measurement. Known test conditions and parameters are established: number and type of vehicles, their speeds and weights and wheel and tire sizes, site layout, and sensor installation issues. System capability must be further upgraded for cumulative momentum or CMV data acquisition and processing. This may involve integration and modification of the IRD fiber optic sensor hardware, and writing our own software or algorithms. After these preliminary tests are completed successfully, then additional components will be purchased to constitute two pilot systems for CCMV model testing at the A & T Intersection or ATI site. This site is selected for pilot system testing since we will incur minimum disruption to city traffic due to remoteness of site from downtown, and proximity to the campus. The IRD optoelectronic interface TCC 540 is capable of binning data by number of axles and spacing classification, binning individual vehicle speed, and overall length, gap between vehicles from the tail of the leading vehicle to the nose of the following vehicle, and binning headway by the time of vehicles going in the same direction, from the nose of the leading vehicle to the nose of the following vehicle. The interface must be integrated with current traffic control systems. Existing control systems run mainly on clocks, and are not intelligent systems. Substantial collaboration is planned between this group and the twin- city DOT's in interfacing sensors with existing traffic signal controls.

TEST SITE DATA AND DESIGN

To implement this model, we have chosen 10 blocks of downtown Greensboro along Friendly Avenue and Market Street and the East Market-Bennett intersection (henceforth named the A & T Intersection or ATI) as the two test sites. Downtown with a major city bus hub and daily traffic volume of 20,000 per intersection is ideal for testing the impact of bus signal priority on busy intersections along the two major thoroughfares. The ATI site has two-way traffic and left turn lanes. Pedestrian crossings are present at both sites.

Bus route preliminary analysis has identified 10 intersections with 10 traffic lights along the 10 blocks. There are three bus tops along Friendly, as buses depart the hub, and 10 stops along Market, as buses return to the hub. Bus route schedules and delays due to frequent stops must be analyzed and actual data gathered. They will be to arrive at average bus speeds between intersections and the impact of bus signal priority on overall traffic flow in downtown Greensboro during rush hours. The CCMV + CM Intersection traffic management model will be first simulated for the chosen test sites using the Synchro 4 software package.

An International Road Dynamics (IRD) portable fiber optic load sensor + IRD TCC 540 multi-lane Traffic Counter/Classifier will be purchased and installed for preliminary proof of concept tests on campus grounds. It will be tested for ease of installation, ruggedness against most road hazards, durability, susceptibility to roadbed vibration noise, sensitivity to vehicle weights, calibration of load dependent signals, signal processing and optical to electronic interface for traffic signal control. The system will be tested for MV, CMV, and combined CM + CCMV model implementation.

To implement and test the CCMV model properly, actual test site traffic volume data must be established. Thus, in addition to publicly available traffic data about the sites, more precise and specific data must be sought from the Greensboro DOT. Collaboration similar to that established with Winston Salem DOT is planned. Statistical data on the traffic volumes at each of the 10 intersections along Friendly and Market, especially, during the two rush hour periods, are essential for the design of the capacity of the CCMV model. The amount of bus traffic and the severity of bus delays due to frequent stops for passenger pickup and drop -off along the two main thoroughfares must be known statistically. These numbers will aid us in assigning CCMV capacities to different intersections for optimum traffic flow control along the 10 blocks. Traffic flow data will first be input into the Synchro 4 simulation tool for a direct diagramming of the 10 intersection traffic patterns. This will enable us to have a clearer picture of traffic along the main and side streets and bus traffic, as well. A table of the 10 downtown blocks and bus schedule and potential delays due to stops are tabulated below.

TABLE 1: Test Beta Site

East Bound	Miles	North/South Bound	West Bound
Market St.	0	Murrow Blvd.	Friendly Ave.
Market St.	0.12	London Street	Friendly Ave.
Market St.	0.1	S. Church St.	Friendly Ave.
Market St.	0.12	N. Davie St.	Friendly Ave.
Market St.	0.08	N. Elm	Friendly Ave.
Market St.	0.07	N. Green St.	Friendly Ave.
Market St.	0.06	Commerce Place	Friendly Ave.
Market St.	0.08	N. Eugene	Friendly Ave.
Market St.	0.12	N. Edgeworth	Friendly Ave.
Market St.	0.07	N. Spring St.	Friendly Ave.
Total Miles	**0.82**		

Source: *2002 Yellow Pages*

Friendly and Market along the 10 downtown blocks have only one way traffic without left turn lanes, but do have pedestrian crossings, This test site is ideal for the demonstration of the CCMV model. On the other hand, the ATI test site has two-way traffic on both East Market and Bennett/Yanceyville, four-left turn lanes and pedestrian crossings. However, since this is an isolated intersection in this area with sufficient traffic volume and proximity to A & T, it serves as an ideal site for testing the CCMV + CM model.

DEFINING THE BETA SITE

Physical Layout

Figure 1 (Dudley St. and Market St.) shows a complete layout of the signal application for the Dudley and Market intersection. There are a total of 4 push button devices, 4-induction loop detectors for all 4 left turning lanes, and induction loop detector for all lanes approaching the intersection on Dudley Street. There are no induction loop detectors for through traffic detection approaching Market Street, at the intersection. The detectors are located 180-feet from the stop bar on Market Street. The detectors record vehicle's presents while approaching the intersection, and during idle traffic mode or while traffic build up.

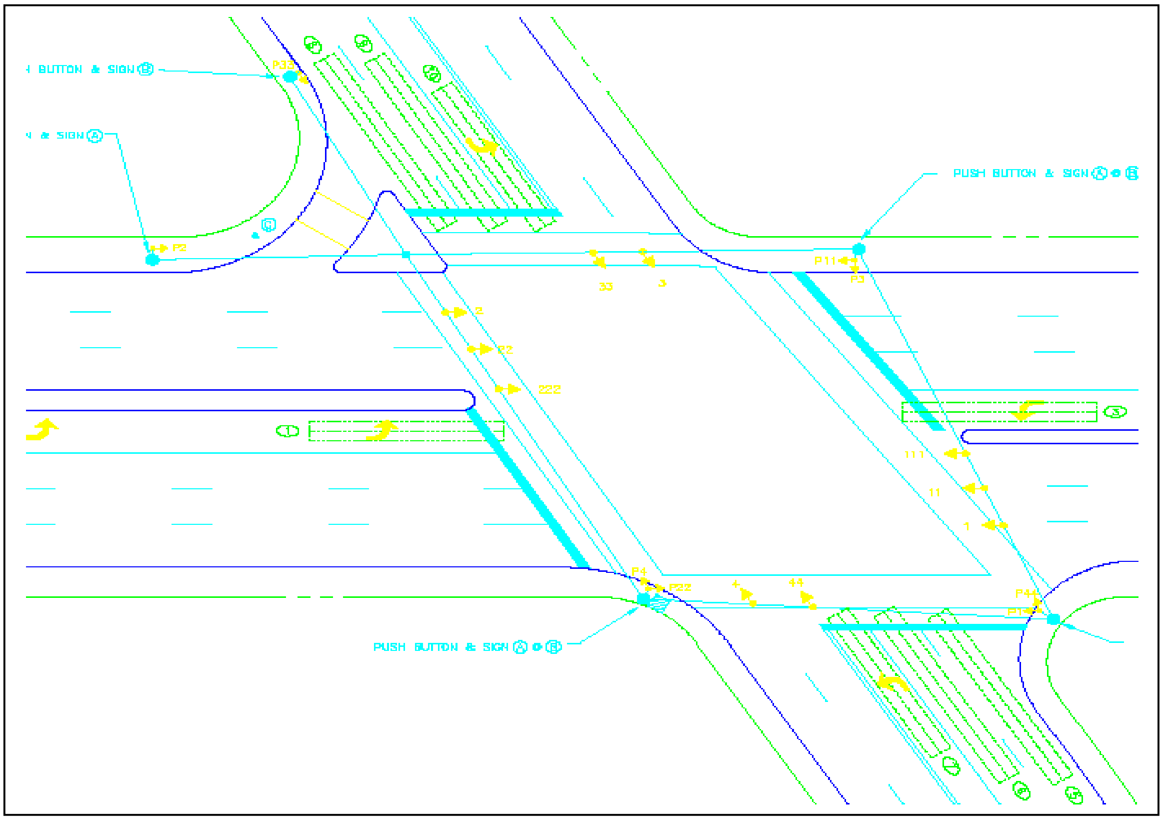

FIGURE 1: Dudley St. and Market St. Intersection (ATI Site)

Data Source: *Greensboro, NC DOT*

The in-ground induction loops are documented as having 3 different lengths: 6' X 30', 6' X 50' and 6' X 60'. The significance for the variance in length can be an attribute for the need to monitor over a specific distance. There are no initial plans to change the layout for this site. Comparison data will be gathered simultaneously to determine the optimal layout. We may determine that we need a longer detection zone than what the loop detectors provide. This will increase the accuracy for forecasting priority and signal time duration. The fiber optic detector will provide the accuracy and repeatability of similar or better quality, compared to the loop detector.

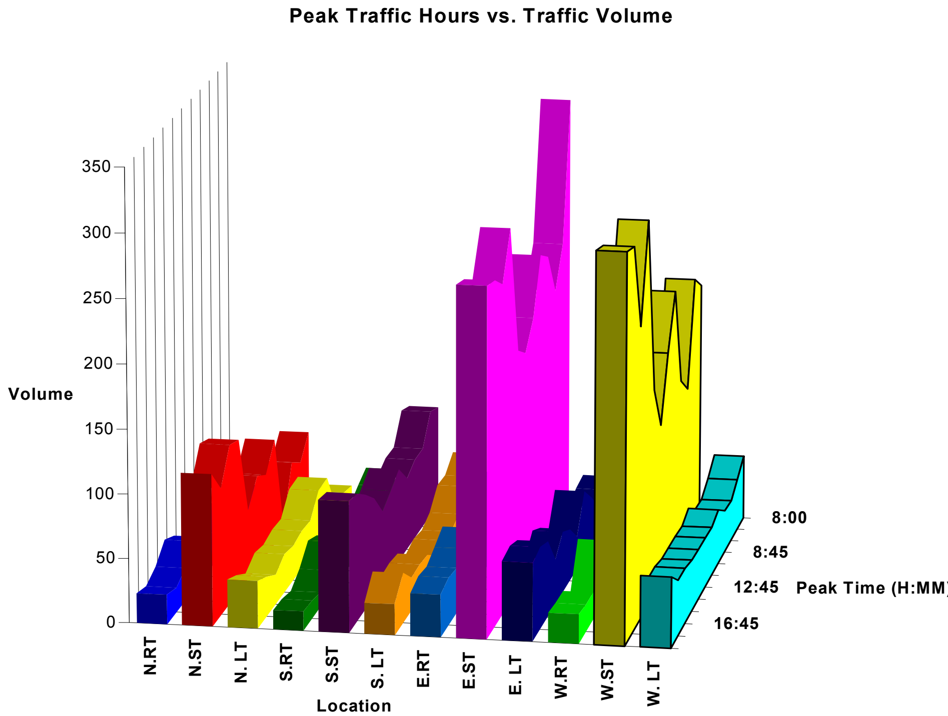

FIGURE 2: Peak Traffic Hours vs. Traffic Volume

Data Source: *Greensboro, NC DOT*

Traffic Volume

Traffic Volume data was furnished by the Greensboro DOT for this beta site. Figure 2 is a 3D graph of the traffic volume vs. location and peak time. The Graph indicates, on the average, east and westbound traffic volume is approximately 3 times more than north and south bound traffic. The left and right turning volumes were not one-to-one, but the delta range is within 67% between East/West and North/South bound traffic.

TABLE 2: Peak Traffic Data for Market and Dudley

From East on Market St.			
	Right	Straight	Left
8:00	23	330	29
8:15	40	223	27
8:30	24	194	22
8:45	24	229	39
12:15	65	238	53
12:30	26	197	26
12:45	22	178	41
13:00	48	188	48
16:30	41	291	36
16:45	35	256	59
17:00	37	267	64
17:15	33	271	61

From West on Market St.			
	Right	Straight	Left
8:00	25	186	50
8:15	16	199	40
8:30	23	120	32
8:45	27	135	33
12:15	29	214	43
12:30	7	174	38
12:45	24	126	39
13:00	43	162	39
16:30	9	301	40
16:45	21	228	43
17:00	21	297	43
17:15	23	301	55

From South on Dudley St			
Time	Right	Straight	Left
8:00	27	79	43
8:15	22	84	33
8:30	15	63	34
8:45	22	63	21
12:15	18	57	16
12:30	16	73	18
12:45	27	65	15
13:00	32	58	25
16:30	19	77	27
16:45	11	89	26
17:00	14	99	39
17:15	15	102	24

From North on Dudley St			
	Right	Straight	Left
8:00	9	54	12
8:15	10	38	15
8:30	12	47	14
8:45	2	6	45
12:15	30	83	44
12:30	16	64	29
12:45	15	72	30
13:00	31	54	27
16:30	37	118	30
16:45	25	123	23
17:00	19	98	31
17:15	23	118	37

Data Source: *Greensboro DOT*

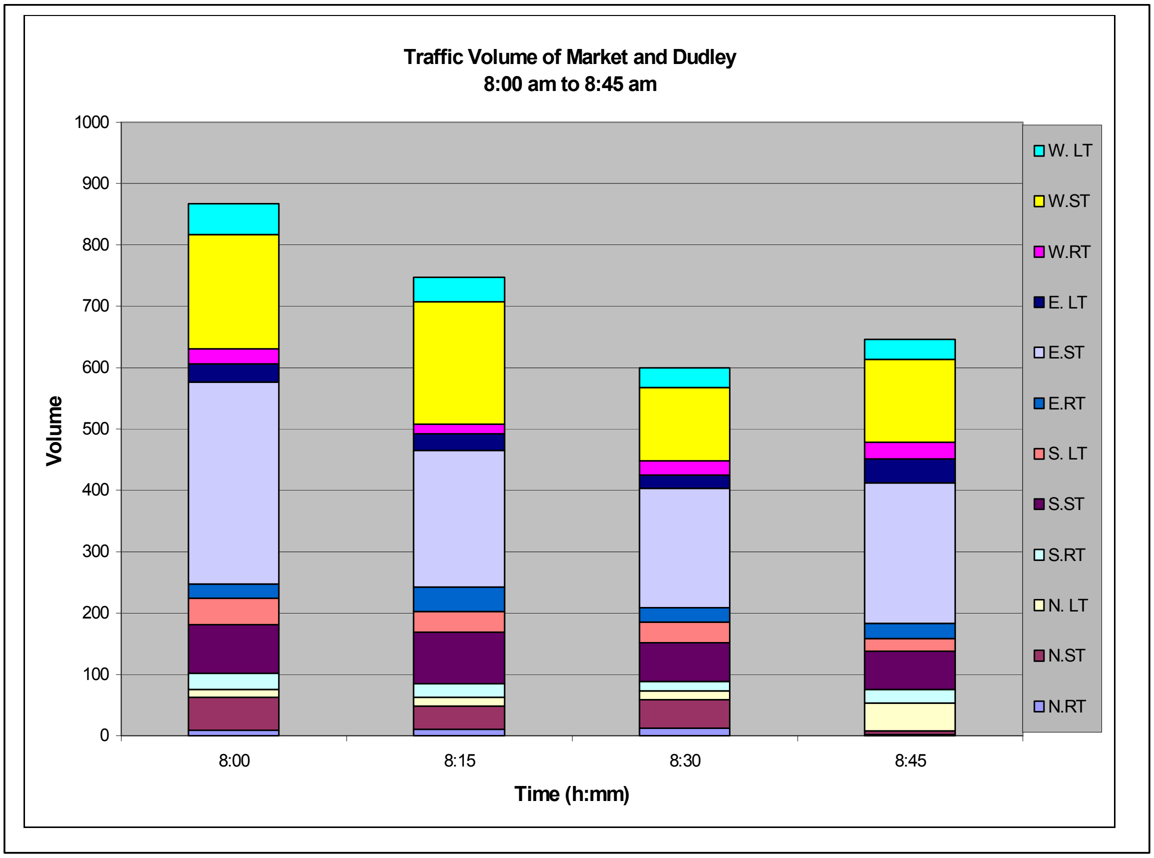

FIGURE 3: Volume Contribution at Intersection
Data Source: *Greensboro DOT*

Public Transit Data for Claim-to-Green

The challenge is to demonstrate how rider-ship can be improved not only for passengers but also for all travelers. The bus travel impacts all in its path, for example the multiple stops along a desired bus rout has an impact on the vehicles traveling behind it. The effects on normal traffic flow from high KE-vehicles are delays in free flow travel time and queuing. The vehicles' slower take off at intersections and scheduled bus stops, compare to a smaller vehicle travel, and reduces the traffic flow from point "A" to "B". When a bus is behind schedule the passenger is potentially behind schedule. All these examples demonstrate the reasoning for improving bus rider-ship.

The model that is presented is derived using 10-city blocks as a beta test site. This site was chosen because it is a major transition (transfer) point for majority of the bus routs in the city of Greensboro. The major through street is Market Street, which is the primary street that buses leave from the transfer terminal-located on Davie Street. From Market Street these 10-cross streets are a prime means for bus transition to their designed routs. According to the Greensboro Transit Authority the minimum average daily traffic volume is greater than 20,000 vehicle on any of the give cross streets. This is a significant and a demanding amount of traffic to manage on a daily basis.

SIMULATION

At this point n the research, field study is performed to gather additional data (volumes, geometry, timing, phasing, etc.) about traffic conditions during peak-and off-peak times. This information will then be input for the simulation. The simulation tool used so far is the demo version of Synchro-4, a package for modeling and optimizing traffic-signal timings. At this time, the software will not allow simulation for ten city blocks, so the full version will be purchased. This tool will allow for modeling actuated signals, skipping and gapping behavior, and applying the information for delay modeling.

CONCLUSION

Two new concepts have been proposed for urban intersection traffic management: the use of vehicle kinetic energy and alternately, momentum, explicitly as a traffic flow parameter, and the traffic management patterned after a traffic cop. The enunciated objective was to relieve urban traffic congestion at urban intersections, thus rendering traffic more environmentally responsible by reducing fuel consumption due to idling, especially by heavier vehicles, and promoting public transit ridership. We believe the proposed scheme should work on paper. By resorting to physics of vehicle motion or kinematics, such as momentum MV or kinetic energy KE in the CCMV model, we have incorporated physics into the geometry of traffic flow. It is almost a scientific axiom that the more elaborate the formulations of a solution scheme to a problem, the simpler the implementation. Therefore, we anticipate a simpler implementation. Since momentum is the essential parameter, vehicle weight measurement must be performed. Present –day above ground detection systems are unable to measure vehicle weight, and hence, rely on studying the geometry for traffic flow. The induction loops, although installed in-ground, detect only inductance change that is unrelated to vehicle weight. Our smart “traffic cop” concept originally proposed required vision of the traffic scene, and naturally pointed to an aboveground system. As the old saying goes: “to see far, you must climb high,” thus aboveground. Our system would have the same limitations as all aboveground systems. In this case, the only means to measure vehicle momentum or the product of weight and speed was to infer weight from vehicle length or size. The speed could be found geometrically by measuring distance covered within a certain period. Our success would have been limited since vehicle weight is not necessarily reflected by its length or size. Even if we did make that correlation, vehicle length measured using overhead cameras would be highly dependent on the aspect or the relative orientation or angle between the vehicle and the camera. Light conditions could potentially affect the accuracy of such measurements as well. We have thus switched to an in-ground fiber optic load sensor, which we found to be much less costly than an Autoscope MVP system. The cost of the latter was beyond our originally proposed budget. However, the downside is that this switch has cost us valuable time. A significant part of the delay was caused by the campus integrated software support policy. Thus, if the campus Center of Information Technology could not support the purchased software, then such software purchase would be allowed. Our TCC 450 software for traffic counter/classifier was deemed to be in that category. After lengthy negotiations, we were able to purchase it. We have however, passed the extended deadline of the project. Fortunately, a follow-up proposal using CCMV model has been founded. So, the implementation phase of the proposed work will continue.

We have also downsized kinetic energy (MV2) to momentum (MV), so as to treat vehicle weight and speed at par. This also renders velocity or speed measurement linear, thus simplifying data processing. We plan to emphasize the Claim-to-Green algorithm for public transit and heavier vehicles. This is automatically incorporated in the CCMV model. Thus, the main street, in which most buses and trucks ply, will automatically command the Green. The impact study to work on the Green extension and its impact on cross streets is in progress.

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Integrating Preventive Maintenance and Pavement Management Practices

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ABSTRACT

Managing a pavement network has become more complex as the competition for pavement preservation funds has grown and the need to justify decisions has increased. As a result, tools that can help an agency collect, analyze, and summarize data have become increasingly important. As a result, asset management tools, such as pavement management systems, have become a necessary component of today's transportation agencies. Coupled with a change in agency focus from expansion to preservation, pavement management systems are now being used to support the use of cost-effective preservation strategies such as preventive maintenance.

However, the use of pavement management tools to support preventive maintenance programs is impacted by the degree to which preventive maintenance treatments are integrated into a pavement management system. Agencies that have successfully integrated their preventive maintenance treatments into their pavement management system have been able to demonstrate the benefits of their preventive maintenance programs to upper management. For the most part, these analyses have focused on the comparison of worst-first strategies to strategies that incorporate preventive maintenance treatments or illustrations of the cost-effectiveness of preventive maintenance treatments.

To fully support a preventive maintenance program, the preventive maintenance treatments must be fully integrated into the pavement management models. This requires a concentrated effort on the part of the transportation agency to re-evaluate its pavement management analysis models to ensure that preventive maintenance treatments and timings are incorporated into the pavement management analysis.

The technical issues that must be addressed to successfully integrate preventive maintenance treatments into an agency's pavement management activities are discussed further in this paper. Examples from transportation agencies that have addressed the integration issues are provided to illustrate possible approaches that may be used.

Key words: pavement management—pavement preservation—preventive maintenance

INTRODUCTION

As the management of transportation networks has become increasingly complex, and the competition for pavement preservation funds has grown, tools that can help an agency collect, analyze, and summarize data have become increasingly important. Nationally, the emphasis on the use of these tools has resulted in the development of a Strategic Plan for the Task Force on Transportation Asset Management that emphasizes the importance of promoting the development of asset management tools and analysis methods, and communicates to states how to better utilize and implement these concepts (*1*). As a result, asset management tools, including the use of pavement management systems, have become a necessary component of today's transportation agencies. Coupled with a change in agency focus from expansion to preservation, pavement management systems are now being used to support the application of cost-effective preservation strategies such as preventive maintenance.

One of the factors that has driven the need for asset management tools is the change in focus in transportation agencies from new construction to preservation. Coupled with changes in legislation that have simplified the use of federal funds for maintenance activities (such as TEA-21), new preservation strategies are being introduced in transportation agencies. Preventive maintenance programs are being developed and implemented as cost-effective strategies for accomplishing an agency's preservation goals.

Nationwide, the experience with pavement preservation programs in general, and preventive maintenance programs specifically, has been limited. Therefore, experience combining pavement preservation and pavement management is even more limited. Several agencies have successfully used their pavement management systems to demonstrate the benefits of preventive maintenance as they began to implement preventive maintenance programs. Michigan, New York, and California are all examples of states that have been able to use pavement management tools to jump-start their preventive maintenance programs and to bolster the amount of dedicated funding allocated to it. For the most part, these analyses have focused on the comparison of worst-first strategies to strategies that incorporate preventive maintenance treatments or illustrations of the cost-effectiveness of preventive maintenance treatments.

Although these communication efforts have proven successful in helping to promote preventive maintenance programs, the use of pavement management to illustrate the potential benefits to the agency is only one aspect of how pavement management can be used to support a preventive maintenance program. Another important component is fully integrating the preventive maintenance activities into the pavement management system models so that preventive maintenance treatments are incorporated into the pavement management recommendations provided from the optimization analysis. This second function is much more difficult than the first, as it often relies on "simulating" conditions. The integration of preventive maintenance into pavement management requires a concentrated effort on the part of the transportation agency to re-evaluate its data collection activities, performance modeling approach, and program development activities to ensure that preventive maintenance treatments, and their timings, can be identified by the pavement management system, and that the benefits realized from the application of the treatments can be accounted for in the optimization analysis. This paper introduces the preventive maintenance concept and outlines the considerations that must be made to integrate preventive maintenance into a pavement management system.

THE PREVENTIVE MAINTENANCE CONCEPT

As defined by the Federal Highway Administration (FHWA), pavement preservation involves a systematic approach to preserving the investment in existing roadways by improving pavement performance and extending pavement life in a cost-effective manner (*2*). It includes a variety of activities that are undertaken to provide and maintain serviceable roadways, including corrective and preventive maintenance as well as minor rehabilitation activities.

An important part of a preservation program is the use of pavement preventive maintenance treatments to improve the functional condition of the network and retard the overall rate of deterioration. Since preventive maintenance treatments are relatively inexpensive in comparison to resurfacing or reconstruction projects, preventive maintenance programs have been found to be a cost-effective means of meeting pavement performance goals. The use of preventive maintenance treatments also slows the rate of pavement deterioration, thereby delaying the need for major rehabilitation by several years, as shown in Figure 1. The delay in rehabilitation needs is more than offset by the fairly low cost of preventive maintenance treatments, which results in a fairly dramatic cost savings in the total costs associated with preserving the pavement network.

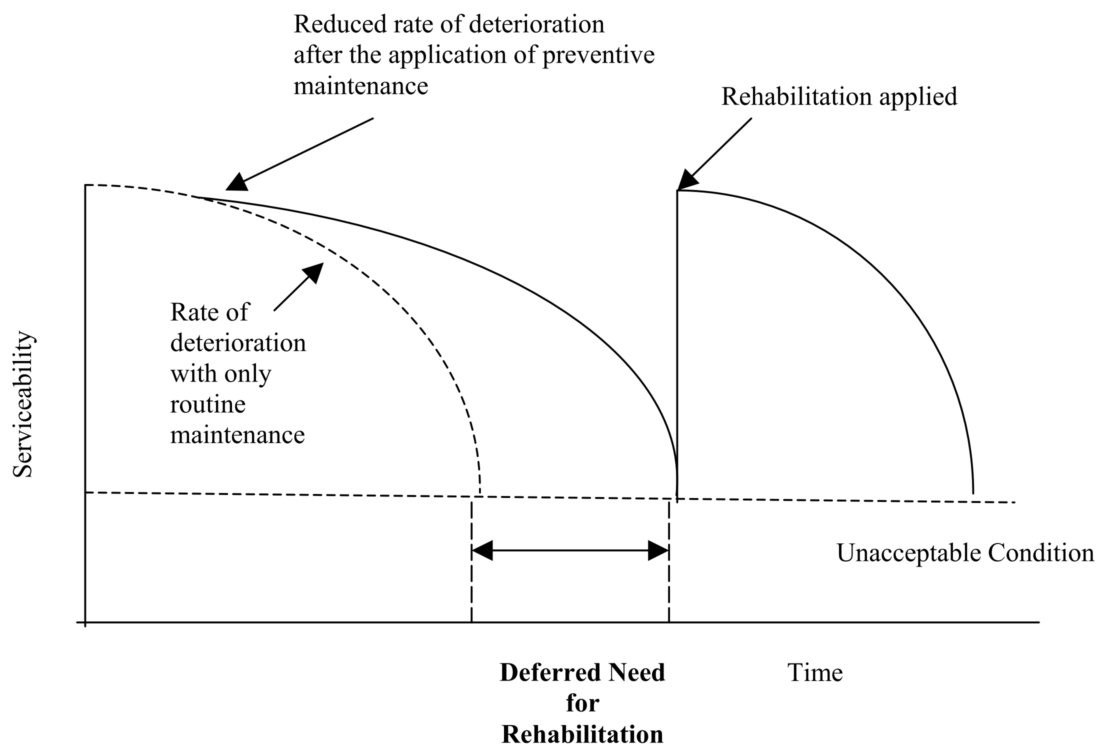

FIGURE 1. Deferred Need for Rehabilitation with the Use of Preventive Maintenance Treatments

There area other benefits that can be realized through the use of a pavement preventive maintenance program. Some of the benefits documented in the literature are listed below (*3*):

- Higher customer satisfaction with the road network.
- The ability to make better, more informed decisions on an objective basis.
- The more appropriate use of maintenance techniques.
- Improved pavement conditions over time.
- Increased safety.
- Reduced overall costs for maintaining the road network.

Despite the benefits that may be realized, many agencies have found it difficult to implement a preventive maintenance program because of the agency's resistance to apply preventive maintenance treatments to roads in good condition when a large part of the network is in poor condition. This is especially difficult when pressures from the public or politicians tend to support a "worst-first" strategy. To overcome this challenge, agencies must utilize their pavement management systems to demonstrate the benefits of a preventive maintenance strategy and support the agency's change in philosophy. However, since pavement management systems have primarily been relied on to serve as programs for identifying and prioritizing rehabilitation needs, integrating preventive maintenance and pavement management requires some modifications. Some of the approaches that are used to integrate preventive maintenance into pavement management and the specific technical areas that need to be addressed to successfully integrate these programs are discussed in the following sections of this paper.

APPROACHES TO INTEGRATION

There are three primary approaches that can be used to integrate preventive maintenance treatments into a pavement management system. The approach used by an agency is dependent on the availability of information in the pavement management system to support the development of the required models, the overall objectives for the analysis of the preventive maintenance treatments, and the sophistication of the pavement management system.

The simplest approach provides recommendations for preventive maintenance candidate sections as a default to the analysis of pavement rehabilitation and reconstruction needs. Using this type of approach, the pavement management system is used to analyze the rehabilitation and reconstruction needs of a network, and any pavement sections that are not candidates for these types of treatments are automatically considered to be candidates for preventive maintenance. Although this approach is easy to implement because it requires no changes to the pavement management models, it provides limited support for analyzing the impacts associated with the use of preventive maintenance treatments.

A slightly more sophisticated approach is to incorporate a single treatment into the pavement management analysis models that represents a variety of preventive maintenance treatments. For example, in addition to evaluating thin overlays, mill and fills, and structural overlays as treatment options, the pavement management analysis will also consider pavement sections for a treatment called preventive maintenance, which is generally applied to pavements in fairly good condition. Pavement sections that are recommended for preventive maintenance are then investigated in more detail to determine the specific type of preventive maintenance treatment that should be applied. This approach is relatively simple to implement and does not require very sophisticated analysis tools to use. However, the performance rules and cost models that are used to analyze the preventive maintenance treatment tend to be averages that represent a broad range of possible treatments. This approach is better than the first approach for analyzing the impacts of preventive maintenance, but does not provide a specific recommendation for the type of treatment that should be used.

The Metropolitan Transportation Commission (MTC) in the San Francisco Bay area uses this type of approach for its analysis of preventive maintenance treatments. Based upon a standard deterioration curve, treatment trigger levels are set to establish the condition level at which various treatments are considered. Figure 2 illustrates the standard deterioration of the Pavement Condition Index (PCI) over time and the progression of treatments that the MTC system utilizes to appropriate identify maintenance and rehabilitation activities. The trigger value between preventive maintenance and light to moderate rehabilitation occurs at a PCI of 70, the trigger value between light to moderate rehabilitation and heavy rehabilitation occurs at a PCI of 50, and the trigger value between heavy rehabilitation and reconstruction occurs at a PCI of 25 (*4*). These values can be modified in the program for various pavement categories, or families. In addition to a treatment trigger, the MTC system has supplemental requirements that influence treatment selection. For example, in order for preventive maintenance to be selected, the pavement section must have a condition higher than 70 and the condition must be projected to remain above the trigger value for at least three years.

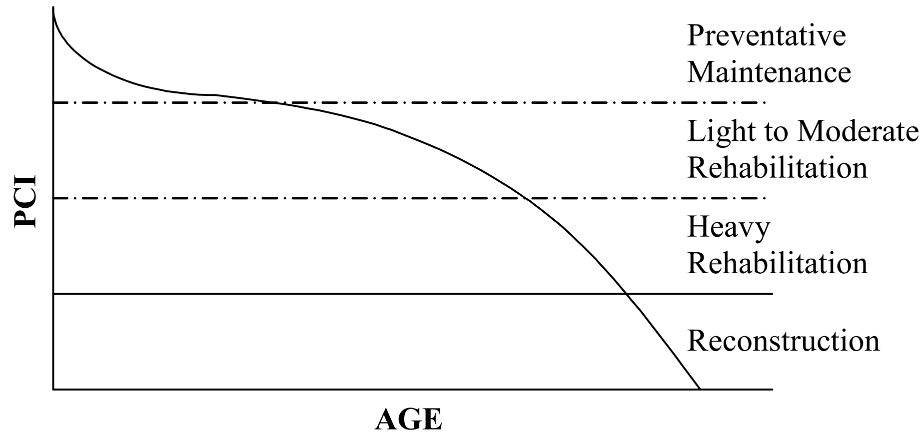

FIGURE 2. PCI Deterioration Curve and Default MTC Treatment Levels

The most sophisticated approach is to define specific preventive maintenance treatments into the pavement management system and to develop performance models, treatment rules, cost functions, and impact models for each treatment that is defined. This approach requires the most effort to establish, but provides the greatest level of support for the preventive maintenance program. An agency using this approach is best prepared to use its pavement management system to support the analysis of the benefits associated with a preventive maintenance program as well as to assist in the identification and prioritization of appropriate preventive maintenance treatments. The requirements to achieve this level of integration are further discussed in the next section of the paper.

INTEGRATING PREVENTIVE MAINTENANCE INTO PAVEMENT MANAGEMENT MODELS

To fully integrate preventive maintenance treatments into a pavement management system, all components of the pavement management system must be examined to determine whether changes are necessary to accommodate the consideration of maintenance treatments, as discussed in the following subsections.

Pavement Condition Assessment and Condition Indexes

Agencies that are taking steps to incorporate preventive maintenance treatments into a pavement management system should investigate several aspects of the pavement condition survey procedures. First, the agency should evaluate the types of distress that are collected during the survey to determine whether the factors that trigger the selection of preventive maintenance treatments are included. For example, some preventive maintenance treatments are triggered to address friction problems caused by excessive bleeding or raveling of the pavement surface. If the pavement condition survey procedures do not report the presence of bleeding or raveling, the system will not be able to recommend appropriate treatments.

Secondly, the agency must evaluate the procedures that are used to convert pavement distress information into pavement condition indexes. If all the distress information is compiled into one, single composite index, then the pavement management recommendations may not be specific enough to identify feasible treatment options. Alternatively, an agency that uses individual indexes for triggering treatments may be better prepared to identify treatments that address the primary cause of the deterioration. For example, if a friction index is available, it could be used to trigger preventive maintenance treatments designed to improve the safety of the facilities. A pavement management system that only uses an overall condition index would not provide enough of an indication of the type of deterioration present to effectively identify an appropriate action.

The agency should also review its survey frequency to determine whether it is sufficient for identifying the window of opportunity for preventive maintenance treatments. In many instances, there is a short window during which preventive maintenance treatments are appropriate. If pavement condition surveys are conducted outside of that window, the opportunity for applying effective preventive maintenance treatments may be lost.

Pavement Performance Models

Pavement performance models are used in a pavement management system to predict future conditions. The results are then used to determine future maintenance and rehabilitation needs and to report the future condition of the pavement network under various scenarios. Many transportation agencies use a "family" modeling approach that groups pavement sections with similar characteristics and uses regression techniques to determine the deterioration pattern that is reflective of the family performance data. In order to properly incorporate preventive maintenance treatments into the pavement performance models an agency must address the development of performance models for each of the preventive maintenance treatments included in the analysis and for each condition index that is used to trigger treatments. The development of performance models that show the change in pavement conditions with and without the application of the treatment provide the information necessary for the pavement management system to evaluate the additional performance that can be realized by the application of the treatment.

The pavement management database must be able to provide the information necessary to support the changes to the pavement performance models (*5*). For instance, if a chip seal curve is developed, chip seal treatments must be identified in the pavement management database. In many agencies, that means that maintenance treatment information has to find its way to pavement management and that the information reported is provided in a manner that is useful to the pavement management section. This requires that data are collected using a common reference system (or a compatible referencing system) to ensure compatibility.

Pavement Treatment Rules

In addition to defining pavement performance models for each treatment considered in the analysis, an agency must also develop treatment rules that indicate the conditions under which the treatment is considered feasible and the reset rules that define the conditions that exist after the treatment has been applied (*5*). In general, setting up treatment rules is not difficult. Many agencies use decision trees, such as the one illustrated in Figure 3, to define the set of conditions under which a preventive maintenance treatment is considered feasible. As with the development of performance models, the pavement management database must contain the information used in establishing the treatment rules.

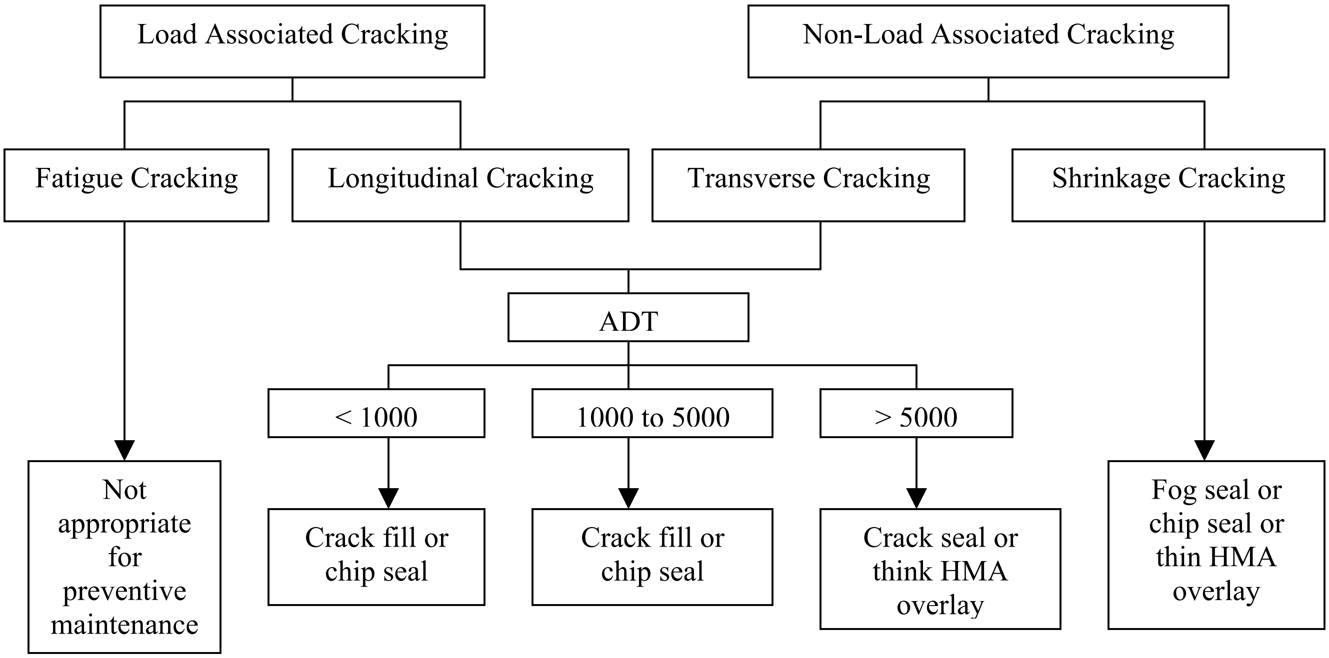

FIGURE 3. Sample Decision Tree for Cracking (*6*)

Treatment Impact Rules

More difficult than establishing the treatment rules for identifying feasible preventive maintenance treatments is defining the reset rules that the pavement management system uses to analyze the conditions that apply immediately after a preventive maintenance treatment is selected for a pavement section. This is more complicated for preventive maintenance treatments than it is for a rehabilitation or reconstruction treatment. For rehabilitation and reconstruction treatments, the condition indexes generally return to a perfect (or near perfect score) and existing pavement performance curves are used to reflect the new rate of deterioration. However, with preventive maintenance treatments this activity is somewhat more complex because the treatments do not necessarily return the pavements back to the highest rating (*5*). Instead, an incremental increase represented by a percentage improvement in condition or some other mathematical expression may be more appropriate. For example, an agency may set a rule that crack sealing provides a 10 percent improvement in pavement condition after its application and returns to the original performance curve within a 3-year window. Alternatively, the preventive maintenance treatment might provide an immediate improvement in pavement condition and follow a slower rate of deterioration after the application of the treatment. The agency’s challenge is to define these rules for each treatment considered in the analysis. The issue is further complicated by the fact that not all pavement management systems allow for the use of these more complex reset rules and the performance of maintenance treatments is not well documented in most agencies.

USE OF PAVEMENT MANAGEMENT TO DEMONSTRATE COST-EFFECTIVENESS

The benefits associated with the use of preventive maintenance treatments as part of a pavement preservation program are numerous. When properly integrated into a pavement management system, the pavement management system can be used to help demonstrate the benefits associated with the use of preventive maintenance treatments, including improved overall conditions or reduced preservation costs.

This concept is illustrated in Figure 4, which shows the dramatic difference that can occur in overall pavement conditions (as defined using a Pavement Condition Rating, or PCR) after several years of incorporating preventive maintenance treatments into a pavement preservation strategy. In this example, the agency compared the resulting conditions from two preservation strategies using the same level of funding for each. In the worst-first strategy, the pavements that were in the worst condition were addressed before any other pavements in the network. In the pavement preservation strategy, a preventive maintenance program was implemented in conjunction with traditional rehabilitation and reconstruction activities.

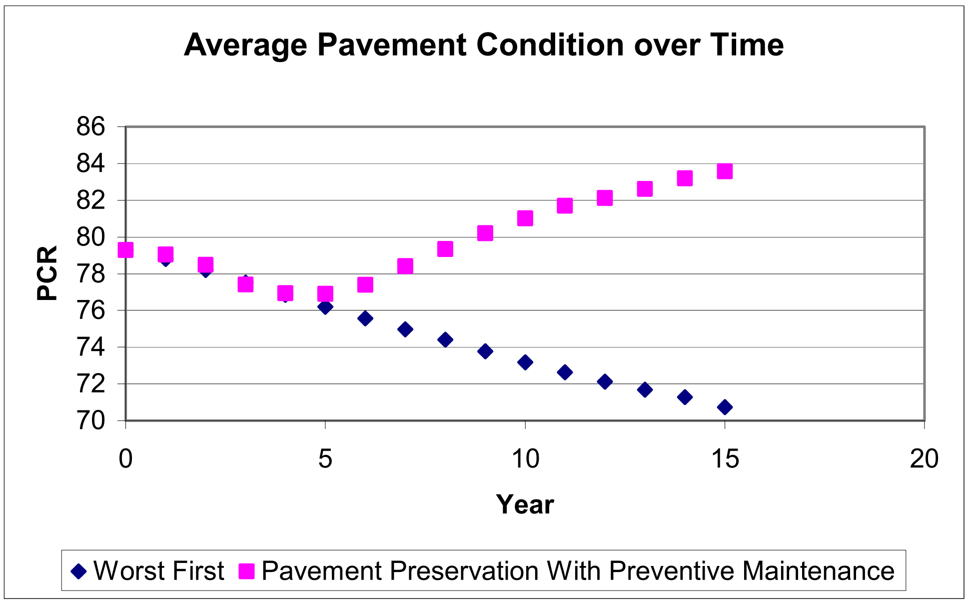

FIGURE 4. Benefits Associated with the Use of Preventive Maintenance Treatments as Part of a Pavement Preservation Strategy (North Carolina DOT, unpublished data)

CONCLUSIONS

A pavement management system can be used effectively to support the use of preventive maintenance treatments as part of a pavement preservation program. However, to successfully use the pavement management system in this manner, the preventive maintenance treatments must be integrated into the pavement management analysis models. This paper introduces three approaches that can be used to integrate preventive maintenance treatments into a pavement management system and the considerations that must be made to achieve the highest level of integration. The benefits that can be realized by integrating preventive maintenance treatments into a pavement management system include the ability to produce more coordinate work plans that demonstrate the cost-effectiveness associated with the use of sound pavement preservation principles.

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