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Crossroads 2000 Proceedings

Contents

PREFACE

Crossroads 2000 was the second biennial transportation research conference cosponsored by the Center for Transportation Research and Education (CTRE) at Iowa State University and the Iowa Department of Transportation. The conference and proceedings were developed under a collaborative arrangement between the two organizations formalized in a Memorandum of Agreement in 1992 and expanded in 1996 to include the University of Iowa and the University of Northern Iowa. The day-and-a-half conference provided an opportunity for Iowans and other Midwesterners to focus on regional and national transportation research and technology transfer issues through presentations of the caliber generally found at national events. The conference allowed transportation professionals from the region to attend a content-rich event without having to travel outside the region.

This proceedings is the set of papers presented at the conference. Twenty-five categories of papers were presented in five concurrent sessions. Reflecting the increasingly critical role of intelligent transportation systems (ITS) in maintaining and enhancing transportation safety and efficiency, one category in each concurrent session addressed an area of ITS. However, papers were included from all areas of interest, ranging from transportation infrastructure design to transportation policy. Together the 76 papers presented at the conference showcase the variety of research that is shaping the next era of transportation.

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The following presentations were not available for inclusion in the proceedings:

“The Kansas City International Trade Processing Center” Ronald Achelpohl, Mid-America Regional Council

“The Role of Anti-Icing and Prewetting at the Iowa DOT (Moving away from Theory)” Dennis Burkheimer, Iowa DOT

“Advanced Public Transit Systems” Craig Cole, Rockwell

“Travel Model Improvement Program Current Status” Michael Culp, FHWA

“Using a Desktop GIS and Travel Demand Software to Develop Traffic Models for Small Cities” Jennifer Kragt, Iowa Northland Regional Council of Governments

“Public Perceptions of the Midwest’s Pavements” David Kuemmel, Marquette University

“Statewide Freight Modeling Using the 1993 Commodity Flow Survey” Mike Lipsman, Iowa DOT

“What Comes after the ISTEA Management Systems? Asset Management” Tom Maze, CTRE, Iowa State University

“The Montana Experience in Statewide ITS Planning” Pat McGowen, Western Transportation Institute

“Electronic Commerce in Transportation: A Carrier's Response to Global Shipper Needs” Gary Nichols, Contract Freighters, Inc.

“Rural Minnesota: Recognizing the Benefits of a Statewide Strategic Plan” Marthand Nookala, Minnesota DOT

“Intelligent Transportation: Setting Statewide Strategic Direction” Vincent Pearce, Booz-Allen & Hamilton

“Encouraging Public Access to GIS Data” Ann Peton, Iowa Office of Information Technologies Services

“Improving Employment Data for Transportation Planning” David Plazak, CTRE, Iowa State University

“Multimodal Statewide Freight Transportation Planning” David Preissig, Iowa State University

“Traffic Flow and Safety Implications of Access Management on Arterial Roadways” Howard Preston, BRW, Inc.

“Traffic Safety and Operational Effects of Protected-Permitted Left Turn Traffic Signal Phasing” Howard Preston, BRW, Inc.

“Roadsurfing the Web in ITS” Matthew Tondl, HDR Engineering

“Selecting Roadside Safety Hardware” Bill Wendling, HCR

The following acronyms may be used without definition in this proceedings:

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society of Testing and Materials
FHWA	Federal Highway Administration
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
SHRP	Strategic Highway Research Program
TRB	Transportation Research Board
USDOT	United States Department of Transportation

Traffic Flow Simulation for an Urban Freeway Corridor

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The objective of this paper is to develop a realistic and operational macroscopic traffic flow simulation model which requires relatively less data collection efforts. Such a model should be capable of delineating the dynamics of traffic flow created by the merging and diverging activities and complex geometric conditions. In addition, it should have the capability of describing shock wave phenomena and movements along the freeway corridor. A modification of the existing equilibrium speed-density function was made. The modified equilibrium speed-density model provided a greater degree of accuracy in describing the nonlinear speed-density relationship. A modified macroscopic traffic flow simulation model was developed in conjunction with the modified speed-density model. The resulting simulation model showed considerable success in simulating actual freeway conditions, and more importantly, provided accurate solutions of ramp metering requirements for the study section. Key words: traffic flow simulation, speed-density relationship, ITS/ATMS application.

INTRODUCTION

The approaches to traffic flow theory may be microscopic or macroscopic. The microscopic approach has resulted in car-following theories which study the behavior of one vehicle following another. The macroscopic approach is analogous to theories of fluid dynamics or continuum theories. Macroscopic traffic flow models are characterized by representations of traffic flow in terms of aggregate measures such as volume, space mean speed, and density. Unlike microscopic models which represent individual vehicle movements, macroscopic models sacrifice a great deal of detail but gain by way of efficiency an ability to deal with problems of much larger scope. An important feature of these latter theories is the conservation of vehicles. It is particularly useful in describing the generation of waves in a traffic stream, their speed, and the behavior of vehicles passing through the waves.

Many older urban freeway corridors are characterized with heavy traffic flow, numerous on- and off-ramps and multiple weaving areas, and poor geometric conditions. Existing macroscopic models generally failed to simulate the traffic operations of such freeway corridors due to their unique characteristics. Therefore, modifications of existing macroscopic traffic flow simulation techniques are necessary to adequate study such fully saturated freeway corridors.

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In addition, the data requirements for existing simulation packages are rather sophisticated. Data usually are collected in an interval between 5 to 30 seconds. Recently developed models usually require information gathered in a five-second interval. Certainly, applying more detailed data may result in relatively superior models. Although today's technology supports such data collection needs, buying such expensive sophisticated data collection equipment is still a major budget concern for many cities and states. The development of a less expensive simulation model in terms of data collection is necessary for cities and states that have budget constraints. Such model development certainly will provide a better opportunity to implement advanced traffic management techniques in these organizations.

The objective of this paper is to develop a realistic and operational macroscopic traffic flow simulation model which requires relatively less data collection efforts. Such a model should be capable of delineating the dynamics of traffic flow created by the merging and diverging activities and complex geometric conditions. In addition, it should have the capability of describing shock wave phenomena and movements along the freeway corridor.

BACKGROUND

A variety of macroscopic models of traffic flow on freeways have been developed during the past two decades. Among those models, Payne's FREFLO is the most well known freeway simulation package. Payne (*1,2*) formulated a variant of the equilibrium speed-density hypothesis that overcomes the spontaneous lockup problem by adding a "look-ahead" term to the speed equation. In Payne's formulation, the equilibrium speed is the speed appropriate for the local density plus a term appropriate for the next downstream section. Although the anticipation term eliminates the spontaneous lockup problem, several studies pointed out that there are other problems with equilibrium speed-density formulations that make them untenable, especially under congested flow conditions (*3,4,5*).

A major extension of Payne's is due to Papageorgiou et al. (*6,7,8,9*). Applying a similar space-time descritization of the conservation equation, Papageorgiou further assumed that traffic volume between two freeway sections might be expressed as a weighted sum of the traffic volumes corresponding to the densities of the sections. Papageorgiou's model is well validated and is capable of describing complicated traffic phenomena with considerable accuracy. However, it consists of a number of nonlinear equations, and required computation time and cost is considerably higher.

Michalopoulos et al. (*10,11*) proposed a simulation model which used the simple continuum modeling based on the conservation equation and an equilibrium speed-density relationship. He argued that the hypothesis of an equilibrium speed-density relationship proposed by Payne (*1*) may not hold, especially at congested and interrupted flows. As such, he developed a model for simulating congested and interrupted flows which does not contain an equilibrium speed-density relationship.

The most noticeable inaccuracy in the above models was the instability in simulating severe congestion with extremely high density. The instability occurred when the traffic entering the freeway from an on-ramp was relatively higher than its normal load, and the densities in the study section as well as adjacent sections tended to be unstable. The potential "lockup" of density was also a major problem of such instability. As such, modifications of the flow conservation equation as well as other components are necessary (*12*,*13*,*14*,*15*).

SIMULATION MODEL FORMULATION

Finite difference methods are the most common approaches to develop macroscopic traffic simulation models. The time and space continuum of traffic flow is divided into discrete intervals to form a difference mesh, and the continuous equation is numerically approximated at the lattice points of the mesh. A criterion $\Delta x/\Delta t > u_f$ has to be met in the discretization process, where Δx is the distance increment of the mesh, Δt is the time increment, and u_f is the free-flow speed. This criterion was included to ensure that the law of traffic flow conservation was not violated, and further, to improve model convergence and numerical stability (*16*).

Four simulation models were developed by employing finite difference methods with the above criterion. The structures of these models are presented below.

Model A

Applying the forward difference method, a discretized conservation equation identical to the FREFLO formula was obtained.
Equation (1):

$$k_j(n+1)=k_j(n)+\frac{\Delta t}{l_j \Delta x_j}[q_{j-1}(n)-q_j(n)+q_j^{on}(n)-q_j^{off}(n)]$$

where $k_j(n)$, $u_j(n)$ = traffic density and speed, respectively in section j at time n;

$q_j(n)$ = flow rate at the downstream boundary of section j at time n;

$q_j^{on}(n)$= number of vehicles entering freeway via on-ramps in section j at time n;

$q_j^{off}(n)$= number of vehicles leaving freeway via off-ramps in section j at time n,

$q_j^{off}(n)=\beta_{1j}\times k_j(n)\times u_j(n)$, and β_{1j} is the proportion of mainline traffic leaving the freeway via off-ramps in section j.

Δt = time interval, 1 minute;

Δx_j = section length (miles)

l_j = number of lanes;

A necessary boundary condition for Equation (1) is $k_{j+1}(n)=k_j(n)$ if there is no downstream section.

Equation (2) represents the dynamic speed-density relationship:

$$u_j(n+1)=u_e[k_j(n+1)]+K_T\left[\frac{u_j(n)-u_e[k_j(n)]}{\Delta x_j}\right]+K_v\left[\frac{k_{j+1}(n)-k_j(n)}{\Delta x_j}\right]$$

where $u_e[k_j(n)]$ = equilibrium speed-density relationship; K_T, K_v = constants.

The first term of the right hand side of Equation (2) is the equilibrium speed function. The second term of the right hand side is an adjustment to speed by relating to the equilibrium speed. The third term provides an adjustment to speed by relating to the changing density in the downstream section. This third term takes into account the effect of drivers' reaction to the changes of traffic condition ahead, as they will either increase or reduce the speed according to the changing densities in the adjacent downstream section.

Equation (3) is the relation to flow-speed-density. The quantity $q_j(n+1)$ in Equation (3) is defined as the flow rate passing through the downstream boundary of section j during time n+1.

$$q_j(n+1)=(1-\beta_{2j})\cdot l_j\cdot k_j(n+1)\cdot u_j(n+1)$$

where β_{2j} is the proportion of mainline traffic leaving the freeway via the off-ramps downstream to the critical location of section j. Both the values of β_{1j} and β_{2j} were obtained from the collected data. It was found that the ratios of off-ramp volumes to mainline volumes were nearly constant.

Model B

The first Equation (4) of Model B is a modified conservation equation which was obtained also by applying the forward difference method. The first term of the right hand side in Equation (1), $k_j(n)$, was replaced by a linear combination of $k_j(n)$ and $k_{j+1}(n)$. The first two terms on the right hand side of Equation (4), α_j x $k_j(n)$ and $(1-\alpha_j)$ x $k_{j+1}(n)$, represent adjustments in changing density with respect to the density measures in the subject and downstream sections, respectively. That is, Equation (4) describes the change in section density by taking into account a "look-ahead" factor—the impact of density in the downstream section. The least squares method was used to calibrate the constant α_j. The speed-density Equation (5) was simplified by using the steady-state equilibrium function. The complete set of Model B is presented as the following:
Equation (4):

$$k_j(n+1)=\alpha_j\cdot k_j(n)+(1-\alpha_j)k_{j+1}(n)+\frac{\Delta t}{l_j\Delta x_j}\left[q_{j-1}(n)-q_j(n)+q_j^{on}(n)-q_j^{off}(n)\right]$$

Equation (5):

$$u_j(n+1)=u_e[k_j(n+1)]$$

Equation (6):

$$q_j(n+1)=(1-\beta_{2j})\cdot l_j\cdot k_j(n+1)\cdot u_j(n+1)$$

where $q_j^{off}(n)=\beta_{1j}\times k_j(n)\times u_j(n)$

α_j = constant

The objective of this modification was to examine and evaluate the choice of α_j in conjunction with the simplified speed–density equation to eliminate the tendency of density "lockup" and instability.

Model C

The flow conservation equations in both Model A and Model B were discretized using the forward difference method. The central difference method was employed to develop the third model. Using the central difference method with a minor modification, the conservation equation can be discretized in terms of space and time in the following form:

Equation (7):

$$k_j(n+1) = \frac{k_{j-1}(n) + k_{j+1}(n)}{2} + \frac{\Delta t}{l_j \Delta x_j}\left[\frac{q_{j-1}(n) - q_{j+1}(n)}{2} + q_j^{on}(n) - q_j^{off}(n)\right]$$

where $q_j^{off}(n) = \beta_{1j} \times q_j(n)$

Equation (7) describes traffic density in progression quite realistically, as a future example will illustrate. However, a minor adjustment has to be made to increase the accuracy of the model. A boundary condition of Equation (7) is same as the one in the previous models, that is, $k_{j+1}(n) = k_j(n)$ if there is no downstream section. In addition, a necessary boundary condition to this model is that, $k_{j-1}(n) = k_j(n)$ if there is no upstream section. The model does not work well at the first and the last sections due to the above boundary condition. Thus, Equation (7) was modified as the following:

Equation (8):

$$k_j(n+1) = \alpha_j \cdot k_{j-1}(n) + (1-\alpha_j)k_{j+1}(n) + \frac{\Delta t}{l_j \Delta x_j}\left[\frac{q_{j-1}(n) - q_{j+1}(n)}{2} + q_j^{on}(n) - q_j^{off}(n)\right]$$

The equilibrium speed-density equation is applied directly to obtain the simulated section mean speed. That is,

Equation (9):

$$u_j(n+1) = u_e[k_j(n+1)]$$

The flow-speed-density equation is Equation (10):

$$q_j(n+1) = l_j \cdot k_j(n+1) \cdot u_j(n+1)$$

Model D

Model D is the most complicated model developed in this paper. The conservation equation obtained from the central difference method, Equation (7), was further modified by adding another term $\theta_j \times q_j^w(n)$.

Equation (11):

$$k_j(n+1) = \alpha_j \cdot k_{j-1}(n) + (1-\alpha_j)k_{j+1}(n) + \frac{\Delta t}{l_j \Delta x_j}\left[\frac{q_{j-1}(n) - q_{j+1}(n)}{2} + q_j^{on}(n) - q_j^{off}(n) + \theta_j \times q_j^w(n)\right]$$

where $q_j^w(n)$ = effective weaving volume in section *j* at time *n*

θ_j = weaving constant of section *j*

$$\alpha_j = \begin{cases} 0.67 & \text{when } j = 1 \\ 0.33 & \text{when } j = J \\ 0.50 & \text{otherwise} \end{cases} \qquad j = 1, 2, \ldots, J$$

$q_j^w(n)$ is computed by summing the volumes of on- and off-ramps within the weaving area. The addition of this new term takes into account the weaving conflict in traffic flow simulation. The speed-density equation of Model A, Equation (2), was applied to this model. The flow-speed-density equation of this model is identical to the one in Model C, that is, Equation (10).

SPEED-DENSITY RELATIONSHIP

Another major modification is the interpretation and construction of the speed-density relationship. The first steady-state speed-density model is introduced by Greenshields (*17*), who proposed a linear relationship between speed and density. Various models were developed following Greenshields' direction, including a logarithmic model (*18*), generalized single-regimes models (*19*,*20*,*21*), and multiregime models (*22*). These models in fact can be summarized in a fairly general form as Equation (12):

$$u_e = u_f\left[1 - \left(\frac{k}{k_{jam}}\right)^l\right]^m$$

where u_f = free flow speed

k_{jam} = jam density

The above formula can be transformed into Equation (13):

$$u_e = u_f \cdot \exp\left[a\left(\frac{k_j}{k_{cr}}\right)^b\right]$$

where k_{cr} = critical density

a, *b* = constants

From speed-density curves as shown in Figure 1, it was found that the speed-density relationships can be viewed as two different curves separated by the corresponding critical density. Therefore, Equation (13) can be rewritten as Equation (14):

$$u_e = u_f \times \exp\left[a_j \times \left(\frac{k_j}{k_{cr}}\right)^{b_j}\right] \qquad \begin{cases} b_j = b_{1j}, & if\ k_j \le k_{cr} \\ b_j = b_{2j}, & if\ k_j > k_{cr} \end{cases}$$

The method of least squares was applied to the curve fitting process. As shown in Figure 1, the resulting model generally describes the relationship between speed and density with greater accuracy.

MODEL APPLICATION AND RESULT EVALUATION

The above models were calibrated and applied to simulate a 5.4-mile section of the I-64-40 corridor in the St. Louis metropolitan area. The study area is comprised of the eastbound and westbound sections between Kingshighway Boulevard on the east and McKnight Road on the west, as illustrated in Figure 2.

The simulation results showed that Model A, shared a similar structure to FREFLO, had a high tendency of "lockup" phenomenon. That is, the density tended to increase very fast and exceeded

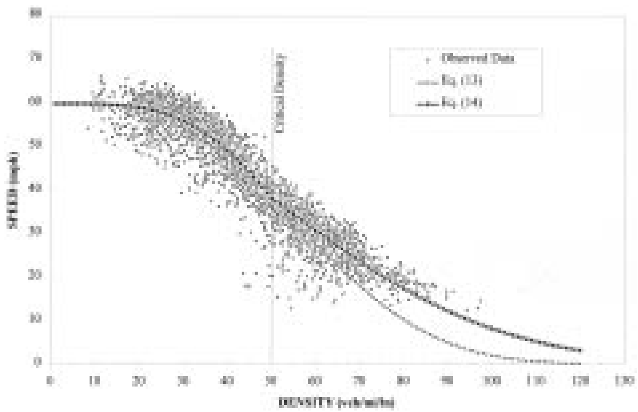

FIGURE 1 Equilibrium speed-density relationship.

the reasonable values. Model B, on the other hand, performed much better than Model A. However, Model B did not simulate well in sections 3 and 4, although it did capture the dramatic increase in density in section 3. The simulation for section 4 generally underestimated the density, and the instability was again a problem of this model.

The results of Model C showed the improvement of the simulation in section 4 as well as the overall model stability. However, it underestimated the impact of the weaving traffic in sections 3 and 4 and failed to capture the dynamics accurately.

The results of Model D showed great improvement in simulation accuracy and stability, especially in sections 2, 3, and 4. The addition of the weaving term in Model D captures the traffic dynamics amplified by the weaving operation in sections 3 and 4. The results of Model D are illustrated in Figures 3 to 5. These graphics vividly show the propagation of congestion along the freeway corridor and the description of the shock wave phenomenon with respect to time and space.

CONCLUSIONS

Modification of existing macroscopic traffic flow simulation techniques is necessary to adequate study fully saturated freeways with multiple ramps in short distance, multiple weaving areas, poor geometric design, and short sight distances, such as the I-64-40 corridor. A successful modification of the modeling approaches has been successfully developed. The modified simulation model takes into account the impacts of merging and diverging activities and weaving operations, resulting in significant improvement in accuracy in simulating the dynamics of traffic operations.

In addition, the data requirements of the developed traffic flow simulation model are less complicated. The simulation model results in high accuracy in simulating the traffic dynamics in real time, and it is sufficient for the development of advanced traffic management programs. Thus, it provides significant cost savings in data collection and model implementation.

The simulation model presented herein is useful to simulate the changes of traffic conditions with the employment of control strategies. Various Advanced Traffic Management Systems (ATMS) control strategies can be developed in conjunction with the simulation model. The freeway simulation model provides an essential requirement for the successful development of a comprehensive

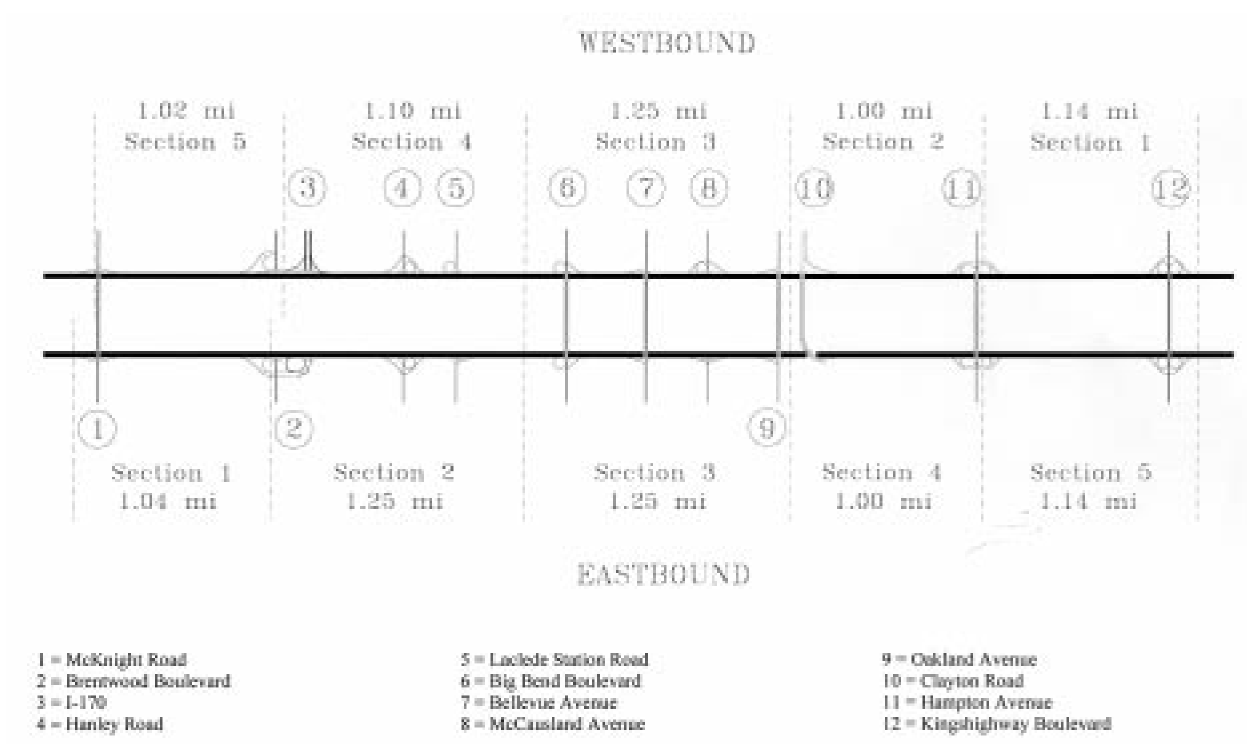

FIGURE 2 Schematic diagram of study site.

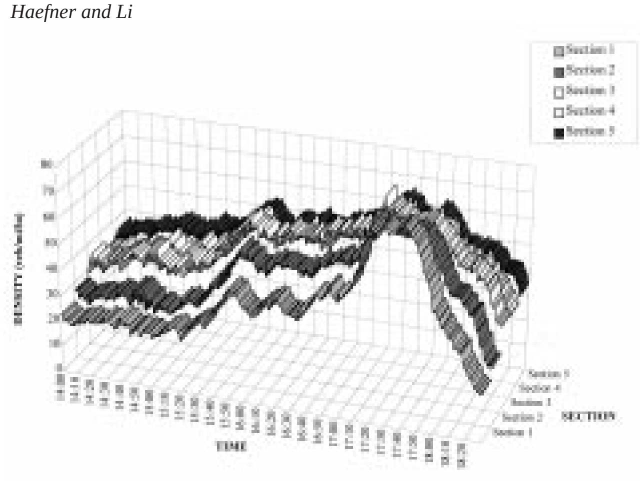

FIGURE 3 Simulation result, Model D, density.

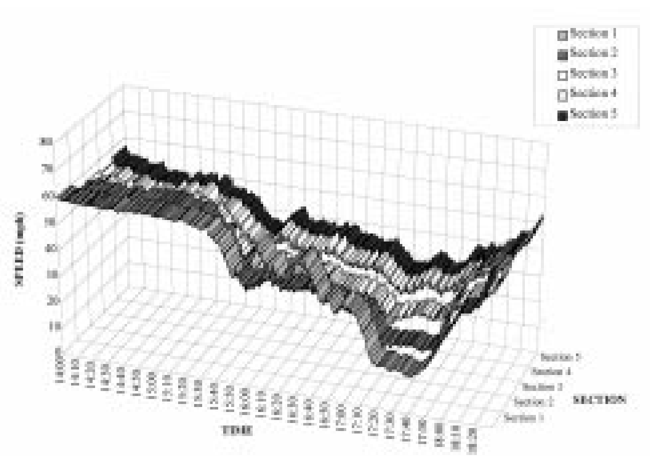

FIGURE 4 Simulation result, Model D, speed.

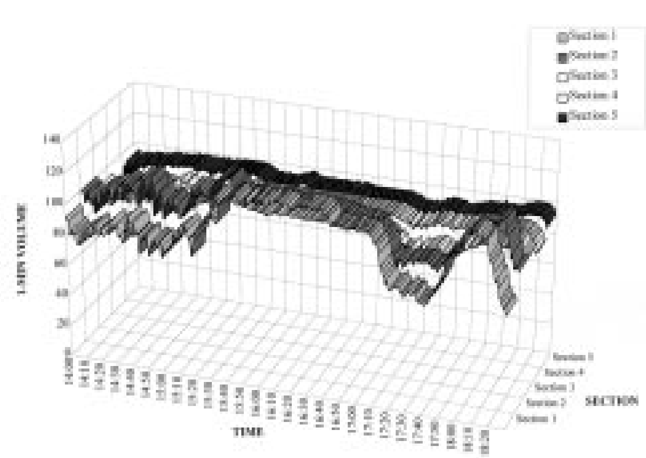

FIGURE 5 Simulation result, Model D, volume.

Intelligent Transportation System (ITS) program. Continued research and implementation of the model refinement will result in improved freeway efficiency and quality of life for many metropolitan areas.

ACKNOWLEDGMENT

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Simulating Traffic for Incident Management and ITS Investment Decisions

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UTPS-type models were designed to adequately support planning activities typical of the 1960's and 1970's. However, these packages were not designed to model intelligent transportation systems (ITS) and support incident management planning. To overcome these limitations, improved algorithms have been proposed and tested in some markets. Unfortunately, these improvements generally have not been included in the commercially available packages and agencies continue to use UTPS-type packages. Therefore, our effort is intended to supplement existing planning model capabilities by exploiting the capabilities of available micro-simulation models. Micro-simulation models can assess, for a localized area or corridor, the effect of ITS implementation and incident response. We introduce a modeling methodology where a sketch-level model represents the regional network and we retain a portion of the available links to develop roadway detail in the area proximate the proposed ITS technology deployment or incident. The combined system is designed to provide real-time travel information for agencies to improve traffic flow through changeable message signs or advanced traffic signals and for travelers to alter route or destination choices through pre-trip information systems such as cable television or web sites. Within this paper, a methodology is demonstrated that uses a GIS interface between Tranplan and Corsim for a test case in Des Moines, Iowa, a medium sized urban area. Key words: travel forecasting, simulation, GIS and ITS.

INTRODUCTION

Conventional transportation planning developed in the early sixties when cities with populations exceeding 50,000 were required to develop "continuous, comprehensive, and cooperative" plans to support the development of the Interstate Defense Highway System through region-wide, systems-oreiented studies with long horizon times (*1*). In the 1970's, transportation planning shifted towards shorter horizon years focusing on corridor analysis. The eighties continued the focus on shorter horizon transportation planning and saw continued improvements in computer tools and computing capabilities. The 1990's have seen a division in planning horizons with shorter horizons focusing on transportation system management, and longer planning horizons focusing on sustainablility. Funding provided by in the Intermodal Surface Transportation Efficiency Act of 1991 promoted the development of intelligent transportation systems (ITS) to improve traffic flow and incident management response (*2*). However, planning and engineering agencies continue to forecast travel using the sequential modeling methodology developed in the 1960's, which was not developed to and have limited ability to address ITS planning issues.

Although work is being performed by the Travel Model Improvement Program (TMIP) to improve traffic models for incident management and ITS investment decisions, many planning agencies still operate computer modeling packages written in the 1970s and 1980s to develop traffic forecasts. While these UTPS-type models were sufficient for regional and corridor level analysis, they are insensitive with respect to the real-time modeling needs of ITS technologies and supporting incident management schemes. To overcome these limitations, alternative algorithms based on dynamic modeling have been proposed and tested in some markets. Unfortunately, these improvements generally have not been implemented into the commercially available packages and agencies continue to use UTPS-type packages. The TRANSIMS model represents a major research effort that is attempting to develop a region-wide micro-simulation tool for traffic modeling (*3*). Although the new model techniques are promising, most agencies have invested staff time and other resources in the conventional travel models and many are looking for a lower investment cost alternative to a completely new system.

Our effort is intended to supplement existing planning model capabilities by integrating them with a micro-simulation model. Micro-simulation models can assess, for a localized area or corridor, the effect of ITS implementation and incident response. The selected model, Corsim, however, is limited in size to approximately 500 surface and 600 freeway links for standard PC implementation. To overcome this limitation we introduce a modeling methodology where a sketch-level model represents the regional network and we retain a portion of the available links to develop roadway detail in the area proximate the proposed ITS technology deployment or incident. The methodology is demonstrated using a GIS interface between Tranplan and Corsim for a test case in Des Moines, Iowa, a medium-sized urban area.

BACKGROUND

Most existing travel models are not designed to represent time dependent delay and are therefore insensitive to evaluating different

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ITS alternatives and responding to traffic incidents (*4*). Travel Forecasting Guidelines indicate that "research is needed to more closely link planning and simulation models to provide more sensitivity to traffic management options while maintaining reasonable resource requirements" (*4*).

Ricci and Gazda identify improvements for existing model limitations including modeling smaller time intervals and integrating planning and traffic simulation models (*5*). With a focus on ITS, a paper by Ben-Akiva et al. defines the heart of successful ITS implementation and operation to be a traffic model that operates in and provides real time information on network traffic conditions (*6*). They indicate that the model should be constructed using a competent simulation model. They further indicate a need for a traffic micro-simulation model to support transportation planning activities in the future.

METHODOLOGY

Current sequential modeling packages are not designed to support Intelligent Transportation Systems (ITS) strategies and incident management responses. Therefore, our approach defines a new framework that integrates an existing travel demand model and micro-simulation package to support modeling ITS alternatives and incident response while minimizing data collection efforts. This effort uses a sketch-level planning model derived from an existing model to determine driver route choice and a traffic micro-simulation package to provide real-time network information which can be fed back into the planning model or visualized using an animation program.

The methodology for developing the system is comprised of converting the existing regional model into a sketch-level model and transferring the new network model into a format for incorporation into the simulation package. These two steps are described in this section.

Development of a Sketch-Level Planning Model

As most existing models are too large for incorporation into the Corsim simulation package (because of program limitations), only a select number of links will be taken form the model to comprise the sketch-level planning model. To determine the streets that will be incorporated into the sketch-level planning model, local knowledge about the area and the existing traffic conditions are required. Examining traffic volumes and identifying congestion locations suggest the basic streets for the network. Other network street should be selected to provide connectivity and to develop sufficient detail without using all the streets.

All attribute information for the sketch-level model is stored in the GIS with the required Tranplan information. As with the existing regional Tranplan model, the sketch-level Tranplan roadway data includes beginning and ending node numbers, distance, operating speed, and available capacity. The sketch-level network does not need explicit socio-economic data or productions and attraction data attributed to the zone centroids as the traffic assignment step will be performed using trip tables derived from the regional travel demand network. After defining the sketch-level planning network, the next step is to modify the regional origin-destination table to include only those trips which use roadways in the sketch-level model. This step involves identifying common roadway elements from both the original and sketch-level models to assign trips to only the selected streets. This selected assignment is then reported as a new trip table.

A series of Fortran programs were written which aggregate the origin-destination information to the new zone numbers in the sketch-level model. The first program develops a reference table matching the structures from the two networks by associating regional centroids with the nearest centroid in the sketch-level network. The second program examines the original origin-destination pair from the Tranplan program, assigns new zone numbers corresponding the sketch-level model, and aggregates the number of trips between the zones.

After aggregating the trip table to represent the new zone numbers, the trip table is assigned to the sketch-level planning model network using Tranplan to produce forecast volumes for the selected links that is representative of the original model forecast. Figure 1 shows a flowchart of the entire system.

Incorporation to the Simulation Package

The simulation package used in this work, Corsim, operates using a space-delimited control file that defines the network through link geometry, turning movement percentages, intersection control, zonal production rates and coordinate data. In addition, the control file contains all the run specific information such as run duration. All the data elements are extracted from the sketch-level Tranplan network and formatted for Corsim through a MapBasic and Fortran program.

The information entered into the control file for the network geometry and operating conditions is provided by the GIS network files: intersections, roadways and turning movements. Each roadway is identified through a beginning and ending node number and attributed with distance, both of which are easily incorporated into the simulation control file. The number of lanes entered for the roadway segments is based on a function of Tranplan capacity and lane channelization is entered as all possible movements allowed based on the existing geometry of the network and number of lanes. The turning movement percentages entered into the simulation are based on the output of the Tranplan assignment and the intersection control is entered with a default value of "green" for all approaches. This replicates the situation encountered within the Tranplan network. However, since there is no means to identify traffic congestion related to intersection queuing and signalization, manual effort is required to incorporate traffic signalization before performing the micro-simulation. The final item that is required for the simulation control file is the zonal productions and coordinate data contained in the intersection table.

After developing the simulation control file, a run of the micro-simulation program is performed. The Corsim program begins with an initial warm-up period for the simulation to reach an equilibrium state, then the software will simulate the network for the duration entered in the control file. After running the simulation, the network will have an output file contain various statistics including intersection delay which can be fed back into the planning model to improve the assignment and an animation file which can be brought into the TRAFVU program to visualize the network operation.

Key outputs from the simulation program are the intersection queue length, average number of vehicles in the queue and average delay times for vehicles. This information is contained in the out-

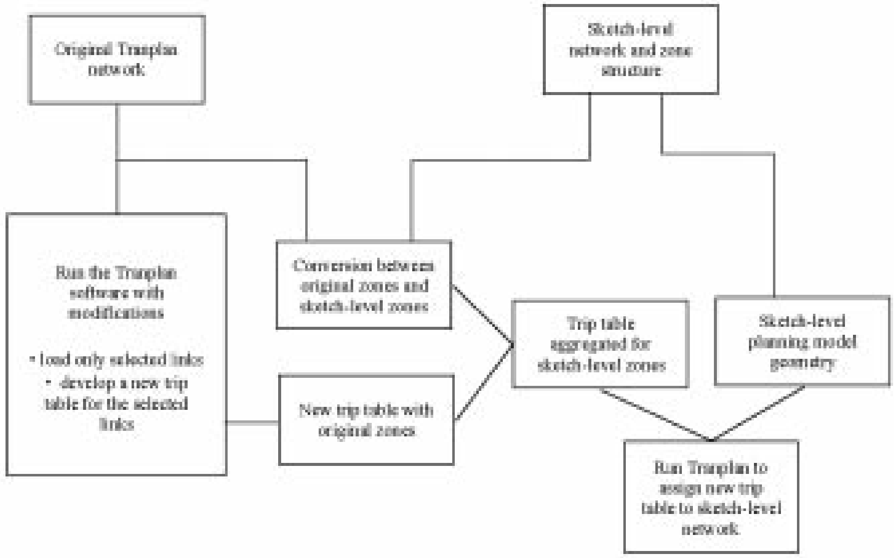

FIGURE 1 Process for developing the sketch-level planning model.

FIGURE 2 The existing Des Moines Tranplan network.

TABLE 1 Peak Hour Factors by Trip Purpose (Morning Peak)

Trip Purpose	Percentage of Trips
Home-Based Work	20%
Home-Based Other	3%
Non-Home Based	3%
Commercial Truck	3%
External-Internal, External-External	3%

put file and can be an important tool for improving the flow of vehicles through the network through feedback loops. This information can also be used through advanced traffic control strategies to change existing signal timings to improve traffic flow.

CASE STUDY: DES MOINES

The case study of the interface between Tranplan and Corsim will focus on the Des Moines metropolitan area. Des Moines, Iowa's capital, has a resident population of nearly 200,000 with almost 400,000 people in the area. The first item addressed in the case study is the development and operation of the sketch-level planning model. Before any work is implemented on the new model, the existing network model needed to be altered to represent a peak period model versus the 24-hour configuration. This was performed through a model alteration with the following factors developed in the NCHRP Report 187 document (Table 28) (*7*). This document outlines the percentage of trip, by purpose, which occur in different hours of the day. For the morning peak hour model developed, the factors are shown in Table 1. The factors were applied to the original trip table used in the network model and all trip purposes, with the exception of the home-based work, were balanced to remove directionality. The development of the peak period model was performed through a series of trip table manipulations. For orientation, Figure 2 shows the existing travel demand network model for Des Moines.

The links included in the sketch-level planning were defined from the Early Deployment Study for Intelligent Transportation Systems performed for the Des Moines area (*8*). As mentioned, this document provides a figure showing all the location for technology deployment within the metropolitan area and this figure was used to select the appropriate roadways. The sketch level planning model developed for the area is comprised of 71 zones, 125 inter-

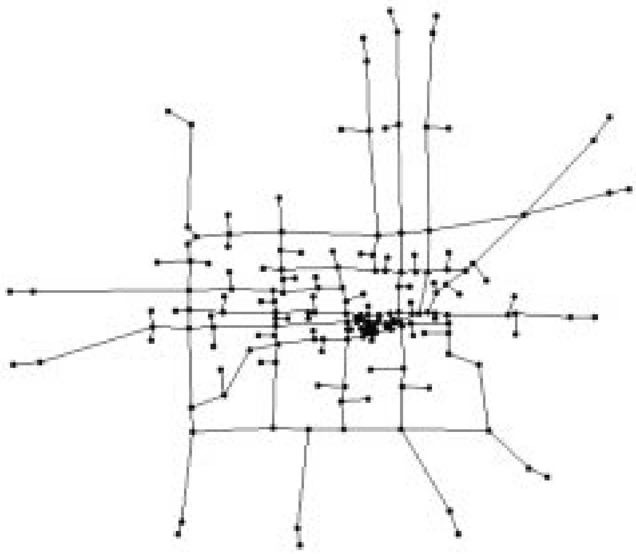

FIGURE 3 The sketch-level network for Des Moines.

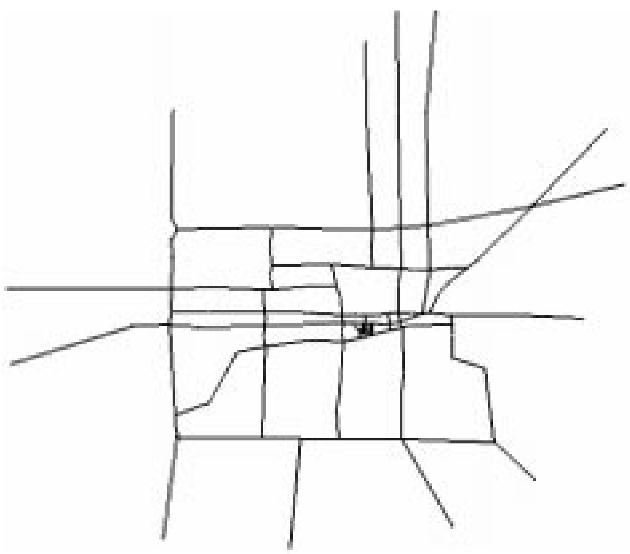

FIGURE 4 The entire network.

nal intersections and 232 roadway links representing arterials, freeway segments and source links. The network is defined representative to a typical Tranplan network within Iowa, therefore, no interchange ramps and all links are assigned a capacity to represent the number of lanes. The sketch-level network for the Des Moines area is shown in Figure 3.

Following the methodology to develop the origin-destination table for the sketch-level network, certain links from the existing were selected and a special assignment of the original network was made to assign only the traffic that used the selected links. After developing a trip table for the selected links, Fortran programs were used to develop a correspondence table between the original zone structure and the sketch-level zones and a new trip table based on the sketch-level models zonal structure was created. It is this new trip table that is assigned to the sketch-level model to perform a run of the Tranplan software. After assigning the new trip table to the sketch-level network, the limited roadway network has traffic volumes representative of the peak hour.

It is this sketch-level network, with assigned traffic data, that is used to develop the simulation network. After running the simulation for the network, the output can be viewed in the animation software, TRAF-VU. The Des Moines sketch-level model, as viewed in TRAF-VU is provided at two scales (Figure 4 shows the entire network and Figure 5 shows an area of downtown).

The combined travel demand and micro-simulation model provides the ability to forecast real-time network operations to support the advanced traveler information and traffic management strategies of ITS. Traveler information provides pre-trip network conditions through various media (cable television, web sites) and allows user to alter trip decisions, such as destination, route and mode. The traffic management strategy supports incident mitigation by identifying demand for alternate routes and provides necessary information for changeable message signs. For ITS, the traffic management strategy supports the identification of diversion routes which will help mitigate travel problem arising from activities such as the reconstruction of Interstate 235 through Des Moines.

CONCLUSIONS AND RECOMMENDATIONS

Included in this paper is a framework and methodology presented and tested which allows users to receive the benefits of real-time micro-simulation from an existing travel demand model. The steps comprising the methodology are:

- start with regional model
- convert to GIS using developed software, if not already in GIS format
- in the GIS, sketch it down to less than 500 links (leaving room to add detail)
- add new links to increase level of detail and sensitivity in study corridor (if needed)
- use developed programs to convert regional model trip tables to sketch model level
- develop network or demand alternative scenarios (if needed)
- use translation program to convert scenario(s) to Corsim
- add traffic control parameters in Corsim to represent various scenarios (if needed)
- run Corsim to assess implications of demand, network or control strategies.

The Des Moines case study demonstrated that the existing model can be aggregated to a sketch-level model and the traffic can be simulated providing real-time traffic information, such as intersection queues and travel delays which can be visualized in the animation software.

A future recommendation being addressed is the ability to perform a focused (or sub-area) modeling if sufficient detail cannot be provided through the sketch-level design. As mentioned in the pa-

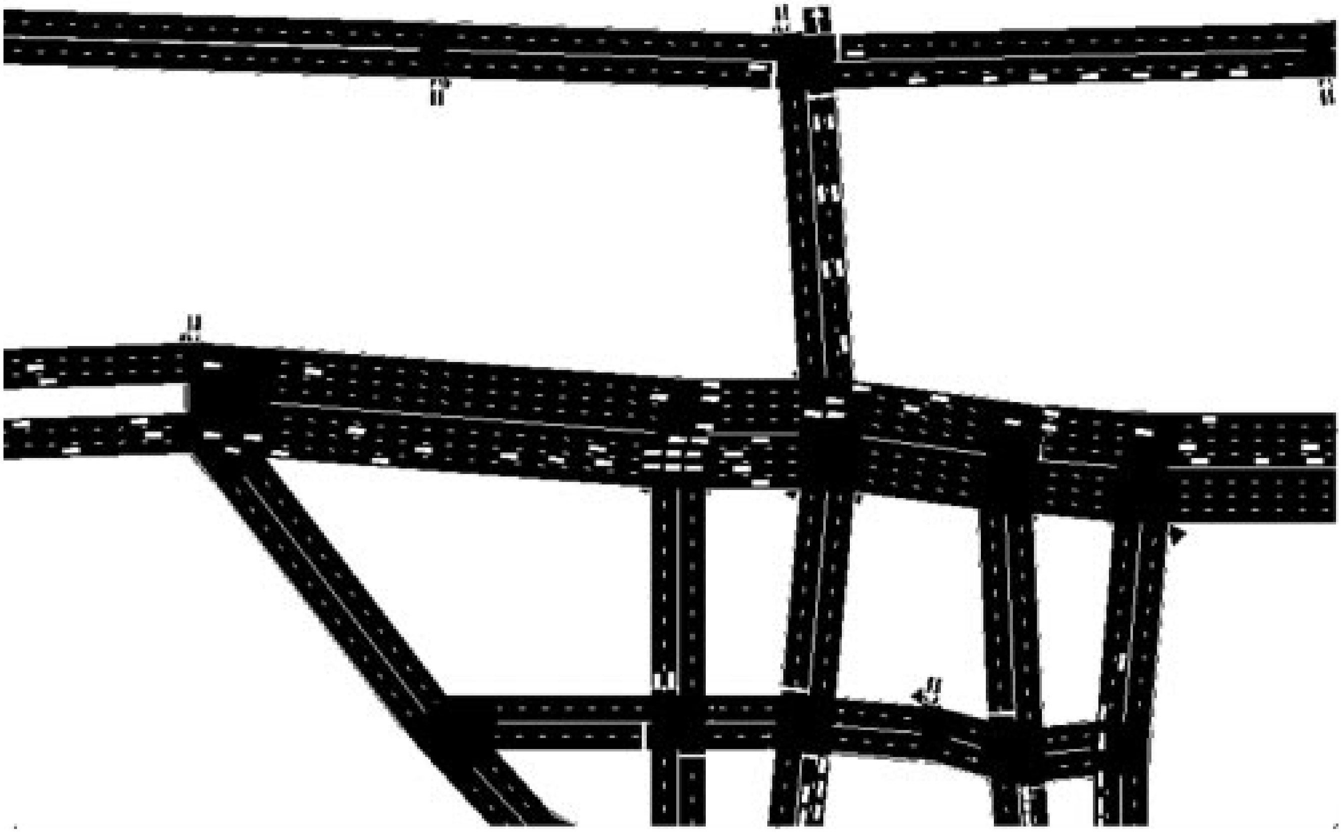

FIGURE 5 Zoomed area of downtown.

per, programs are being written which will provide the ability to "feedback" travel time and delay information into the travel demand package to improve the traffic assignment. The case study performed in this paper examined the use of the methodology for Des Moines, however, there was no effort made to calibrate or validate the output from the simulation. If investment decisions were to be based on the output from a model following this methodology, it is recommended that extensive calibration and validation efforts are performed. The final recommendation is to incorporate dynamic trip table development strategies to improve the analysis of specific time intervals.

ACKNOWLEDGMENT

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Examination of Fundamental Traffic Characteristics and Implications to ITS

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The purpose of this paper is to study and analyze the essential traffic characteristics of a 5.4-mile stretch of I-64-40 located within the St. Louis metropolitan area. The freeway experiences heavy congestion during peak periods attributed to factors such as exceedingly high traffic volume, conflicting merging and diverging movements, multiple weaving areas, and overall poor geometric design. The freeway volume, speed, and density data were collected on various locations within the study area and the data was translated into a series of graphics including, a speed-volume-time of day chart, a speed-volume chart, a volume-density chart, and a speed-density chart. The numerical result indicated that the capacity, free flow speeds, and critical speeds varied for different lanes. However, it was observed that the critical densities were relatively similar for all lanes at the same location. This indicates that density may be a rather important and more reliable measure than volume and speed in the determination of freeway capacity. This study provides a better understanding of the freeway dynamics. In addition, it forms a comprehensive database for the development and implementation of congestion management techniques utilizing Intelligent Transportation System (ITS). Key words: freeway dynamics, u-q curve, q-k curve, u-k curve, critical density, ITS application.

INTRODUCTION

The purpose of this paper is to study and analyze the essential traffic characteristics of a 5.4-mile stretch of I-64-40 located within the St. Louis metropolitan area. The freeway experiences heavy congestion during peak periods. The congestion is mainly attributed to factors such as exceedingly high traffic volume, conflicting merging and diverging movements, multiple weaving areas, and overall poor geometric design. This study identified and examined the important characteristics of traffic operation, and further, it provided an opportunity for effective exploration of ITS programs.

TRAFFIC VOLUME-SPEED-DENSITY RELATIONSHIPS

The freeway volume, speed, and density data were collected in two different intervals: five-minute intervals when using tube counters and one-minute intervals when using Nu-Metrics Hi-Star counters. However, one-minute counts were derived from the five-minute counts using Poisson distribution random number generation. This meant that all counts could be treated as one-minute counts for the purpose of analysis (*1*).

The portion of I-64-40 being researched for this study was divided into five sections as shown in Figure 1 (*2*). Data was collected at strategic locations within each of these sections for both directions and for the A.M. and P.M. time periods. The collected data was translated into a series of graphics including: a speed-volume-time of day chart, a speed-volume chart, a volume-density chart, and a speed-density chart (*1*).

Data was collected multiple times for a given location and the aforementioned series of graphics were created for each data collection period. However, in order to gain a better picture of the freeway conditions and relationships, speed-volume (u-q), volume-density (q-k), and speed-density (u-k) charts were created that are composed of all data obtained for a given location within a freeway section. Figures 2 through 4 illustrate these graphics for westbound I-64-40 at the Hanley Road location. From these figures, the capacity and critical density of the freeway section may be determined. It is not necessary to include all of the charts that were generated for each of these locations. Instead, summaries of the results for each section of the study area are shown in Tables 1 and 2 (*1*).

Observing Table 1, we see that the capacity and density of the eastbound sections remain essentially the same at 2300 veh/hr/ln and 50 veh/mi/ln respectively until the Clayton-Hampton section. In this section, the capacity drops to 1950 veh/hr/ln and the density decreases to 45 veh/mi/ln. Unfortunately, counter malfunction did not permit data collection in the Hampton to Kingshighway study area, thus explaining why data is not shown in Table 1 for this section.

Table 1 Summary of Eastbound Section Information

	Section: McKnight to Brentwood	I-170 to Hanley	Big Bend to Bellevue	Clayton to Hampton	Hampton to Kings-highway
Capacity (veh/hr/ln)	2300	2300	2350	1950	N/A
Critical Density (veh/mi/ln)	50	50	50	45	N/A

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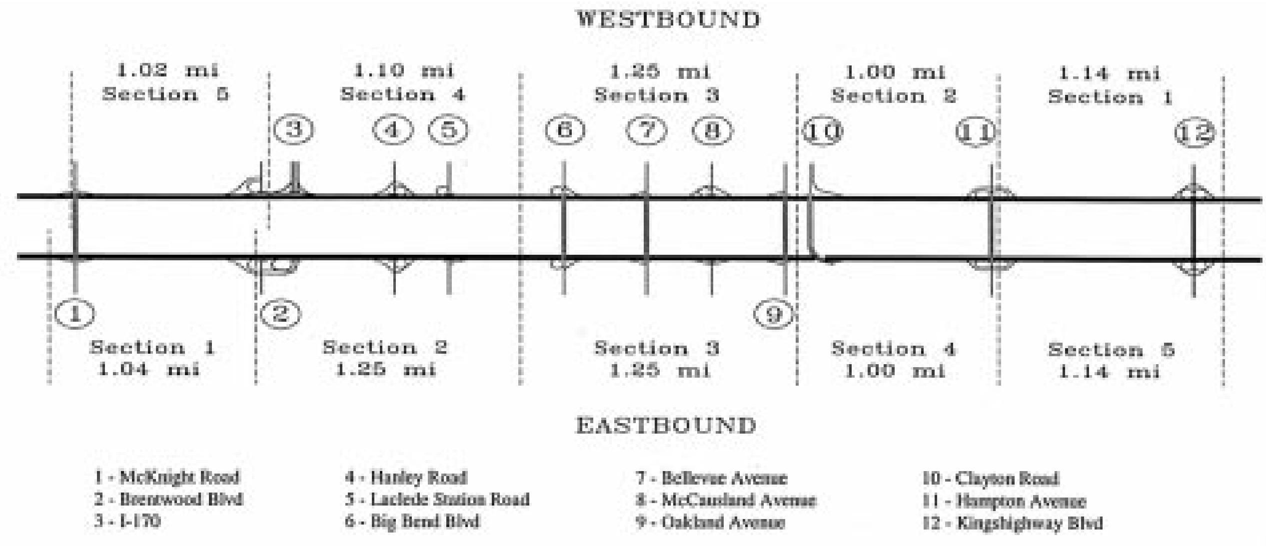

FIGURE 1 Diagram of study section.

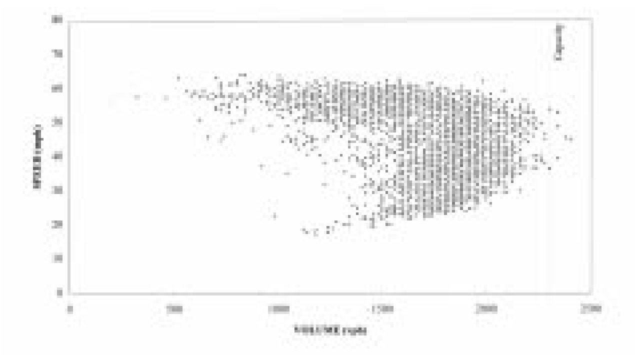

FIGURE 2 Speed versus volume, I-64 WB at Hanley, 3 lanes.

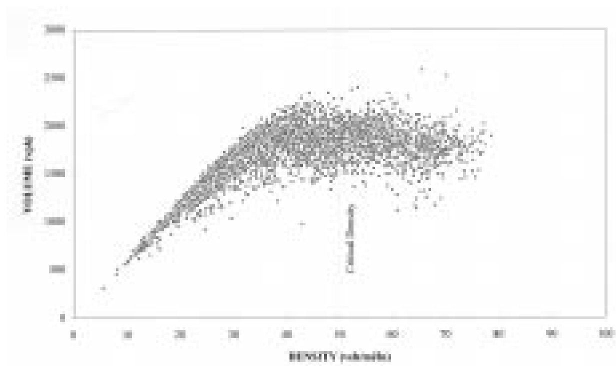

FIGURE 3 Volume versus density, I-64 WB at Hanley, 3 lanes.

Table 2 Summary of Westbound Section Information

		Section				
	Hampton to Clayton	Clayton to Oakland	Bellevue to Big Bend	Big Bend to Laclede	Laclede to Hanley	Brentwood to McKnight
Capacity (veh/hr/ln)	N/A	2150	2350	2375	2300	2250
Critical Density (veh/mi/ln)	N/A	50	50	48	50	50

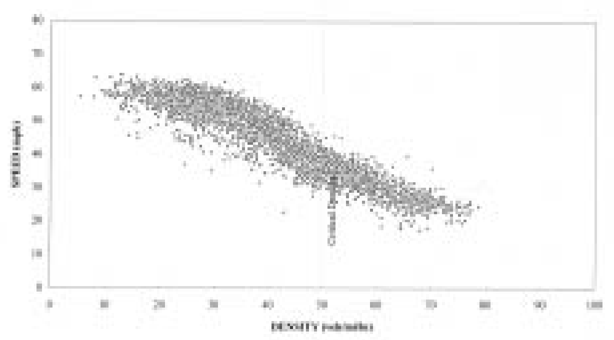

FIGURE 4 Speed versus density, I-64 WB at Hanley, 3 lanes.

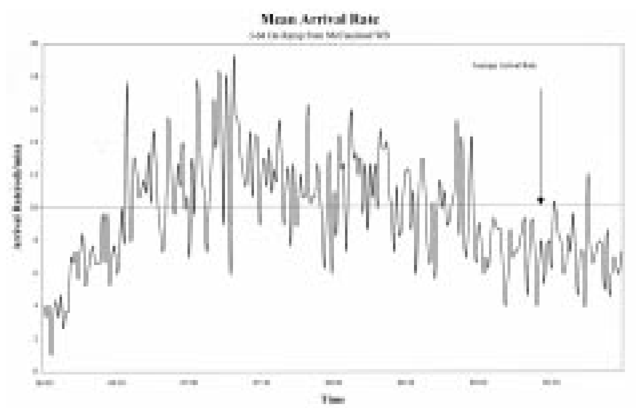

FIGURE 5 Mean arrival rate.

The characteristics of the westbound sections are more dynamic, as can be seen from Table 2. The capacity increases significantly from the Clayton-Oakland to the Bellevue-Big Bend section. It increases then slightly more, but begins a downward trend from the Big Bend-Laclede to the Laclede-Hanley location. Although the capacity changes for each section, the density appears to remain relatively constant.

To this point, the volume-density relationship has been shown in a relatively macroscopic view. In addition, individual lane traffic characteristics were also observed. Free-flow speeds were analyzed and determined to be significantly different for each individual lane. Free-flow speeds were approximately 55 mph, 58mph, and 63 mph for lane 1, lane 2, and lane 3, respectively. Lane capacities were also identified from u-q curves. The capacities for lane 1, lane 2, and lane 3, were approximately 1700 vph, 1900 vph, and 2100 vph, respectively. The traffic flow in lane 3 tended to move faster than the other lanes and its capacity was the highest. Lane 2 traffic moved slightly faster than lane 1 under uninterrupted traffic flow. The traffic in lane 1, which always has to interact with high truck percentages and ramp merging and diverging activities, showed the lowest free-flow speed and capacity (*1*,*3*).

Analysis of the q-k and the u-k curves indicated that density had strong relationships with volume and speed until a certain density range was reached (35-45 veh/mi/ln). When the density was over 40 veh/mi/ln, the relationships were unstable, as volume and speed began to decrease. This phenomenon was experienced for each individual lane.

Due to the different free-flow speeds, the capacities were different by lanes. In a rather conservative way, the critical densities were identified from the q-k curves. The critical density was defined as the density corresponding to the maximum volume that can be accommodated on the freeway. Theoretically, the v/c ratio should be equal to 1 when the density increases from the steady state to the point of critical density (*4*,*5*). The observed critical densities for lane 1, 2, and 3 seem to be very close to each other, regardless of their observed free-flow speeds. This suggests that density may be a rather important and more reliable measure than volume and speed in the determination of freeway capacity (*2*).

RAMP VOLUME AND ARRIVAL PATTERN

Observing the characteristics of ramp behavior helps to understand freeway traffic flow. Volumes and arrival patterns are two important ramp characteristics that have major impacts on freeway operations. Arrival patterns are patterns of vehicles arriving at the ramp in terms of statistical distributions, mean arrival rates, and standard deviations (*3*). This section examines selected on-ramps with the above traffic characteristics to determine the levels of freeway operations.

In this study, 5- and 1-min volumes were established during certain A.M. and P.M. periods only. The study in this paper analyzes freeway volume, speed and density data collected from detailed 5- and 1-min counts. The periods were determined from critical ranges of previous studies (*2*,*3*). The main objective of using traffic counts at short intervals was to focus on the arrival rates and patterns of selected ramps during critical traffic periods.

The output of the ramps monitored were all summarized in detailed 1-minute intervals to obtain more precise information, particularly with respect to arrival patterns. The more detailed 1-minute data was also used directly to develop a macroscopic traffic flow simulation model (*1*). Constrained by the memory capacity of the counters, data collected were for four hours and fifty minutes. The selected A.M. range was from 5:00 to 10:00 A.M. while the P.M. range was from 2:00 to 7:00 P.M. Select on-ramp locations in the westbound direction were analyzed. The same analysis can be applied to the entirety of the study section. The chosen locations were on-ramps from McCausland and Hanley.

Both the A.M. and P.M. mean arrival rates were analyzed for the McCausland on-ramp. This ramp was important because it was the location of the ramp metering testing site. For the A.M. period there was a marked increase in mean arrival rate from 6:30 to 7:00. A typical graph generated for is shown in Figure 5, which represents the mean arrival rate at this location during the morning 'rush hour'. It should be noted that this phenomenon was similar to historical data (*2*,*3*). Since ramp-metering testing was not performed during A.M. periods, the current traffic conditions were not affected with any changes in ramp traffic. The unstable arrival patterns at this location continues to create dramatic impacts on the freeway flow. Converted data indicated an overall arrival rate of 9.63 veh/min. In contrast, the A.M. Peak Average was 11.42 veh/min. The mean arrival rates during the P.M. monitoring period for the McCausland on-ramp show steady mean arrival rates for the data collection period. After further analysis a normal distribution characteristic for arrival rates was evident. The steady arrival rates may be attributed to the ramp metering installed during the study period. A statistical summary of the arrival rates for the afternoon peak and entire period showed that P.M. peak average was noticeably lower than the A.M. peak average at 7.30 veh/min.

A critical part of the freeway section that is of interest because of its significant traffic congestion is near the Hanley on-ramps. The mean arrival rates for the N.B. Hanley on-ramp were recorded and the expected unstable rates were evident with a marked increase from 7:00 to 9:00 A.M. During this period the highest mean arrival rate reached 14 veh/min. This area poses problems due to the high volume of weaving. At this point, the Hanley on-ramp traffic moving to the mainline traffic begins to conflict with the mainline traffic moving towards the I-170 and Brentwood off-ramp traffic. A histogram of the mean arrival rate indicated a normal distribution. Data observed showed that the P.M. average mean arrival rate at 8.91 veh/min is higher than the overall rate of 7.36 veh/min.

Similar instability in mean arrival rates is observed at the S.B. Hanley on-ramp as that experienced in the N.B traffic. This high volatility is the major cause of inhibition of free traffic flow at this ramp location. In addition, it is worsened due to the percentage of heavy vehicles, 11.73%. Further study of the area concluded that inadequate geometric design at this location added constraint to traffic movement.

Analysis of lane 1 for the entire study section was also made. The density and headway characteristics were observed to determine how the on-ramp and off-ramp vehicle flow affected the mainline traffic. Density for the A.M. peak (7:00 to 9:00) was above average most of the time, as expected. The headways were more stable throughout the morning period, except for the period between 6:00 and 6:30 A.M. where there were large increases in gaps between vehicles. The average density and headway for the morning period were 31.25 vpmpl and 6.26 veh/sec respectively.

The afternoon lane 1 summary of density and headway was also observed. As expected, during the P.M. peak (3:00 to 6:00 P.M.) density was typically above the average. The headways were consistent throughout the afternoon study period. The mean density was 47.11 vpmpl. The stability of headway is also indicated by the small difference in mean and mode at 2.23 and 2.20 veh/sec. These trends are evidence of the impact that vehicles entering and exiting the ramps have on the mainline traffic.

The above information regarding arrival rates and patterns, combined with volume, speed, and density measurements, form the basis for a freeway traffic simulation model. Information from the ramp metering testing site is also used to provide insight for the ramp metering optimization algorithm. The following synopsis of the ramp monitoring analysis provides more detail on the analysis of ramp metering effects.

RAMP MONITORING ANALYSIS FOR THE TESTING SITE

Part of this research included monitoring the effects of a ramp metering test site, located on the westbound on-ramp from McCausland Ave. Ideally, ramp metering is most effective when all critical ramps on the freeway system, as determined by their impact on the mainline, are metered (*3*). Thus, this single ramp meter on the westbound leg of the I-64-40 study corridor is a somewhat less than ideal condition. However, appreciable changes in the ramp arrival rate and the mainline densities were noticeable.

The densities upstream and downstream of the westbound McCausland on-ramp for the 1996 and 1997 data were compared. The 1996 data was collected prior to the installation of the ramp meter at the McCausland ramp and the 1997 data was collected after installation. Thus, Figures 6 and 7 provide a "before and after" contrast of the density behavior at the selected site. In comparing the data for both years, it is apparent that the 1997 densities are more uniform. Furthermore, spurts in traffic density are less sudden than those present in the 1996 data. Although there are many possible reasons for this phenomenon, it does seem appropriate, given that the ramp arrival rate is more uniform.

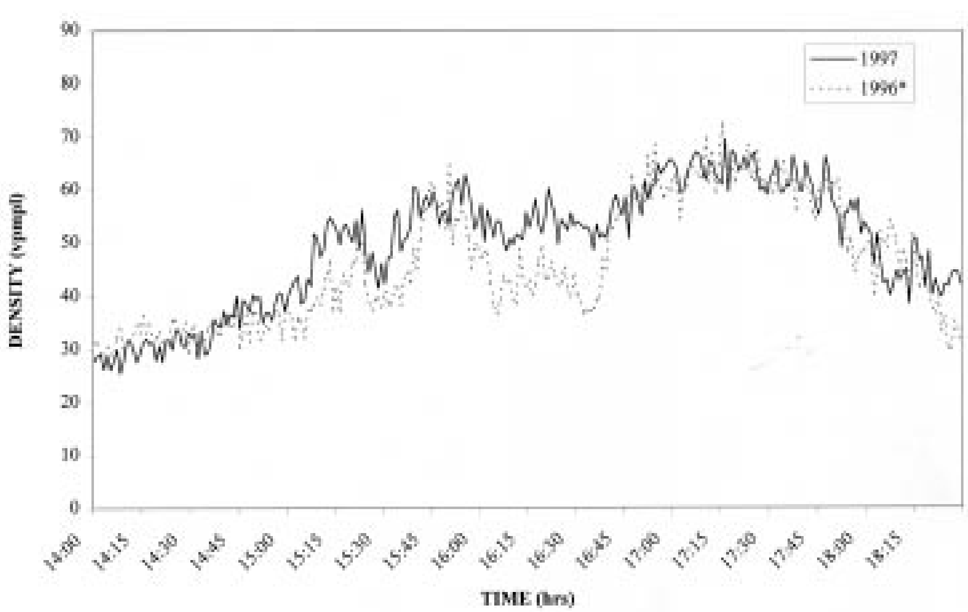

FIGURE 6 I-64 WB upstream of McCausland (all lanes).

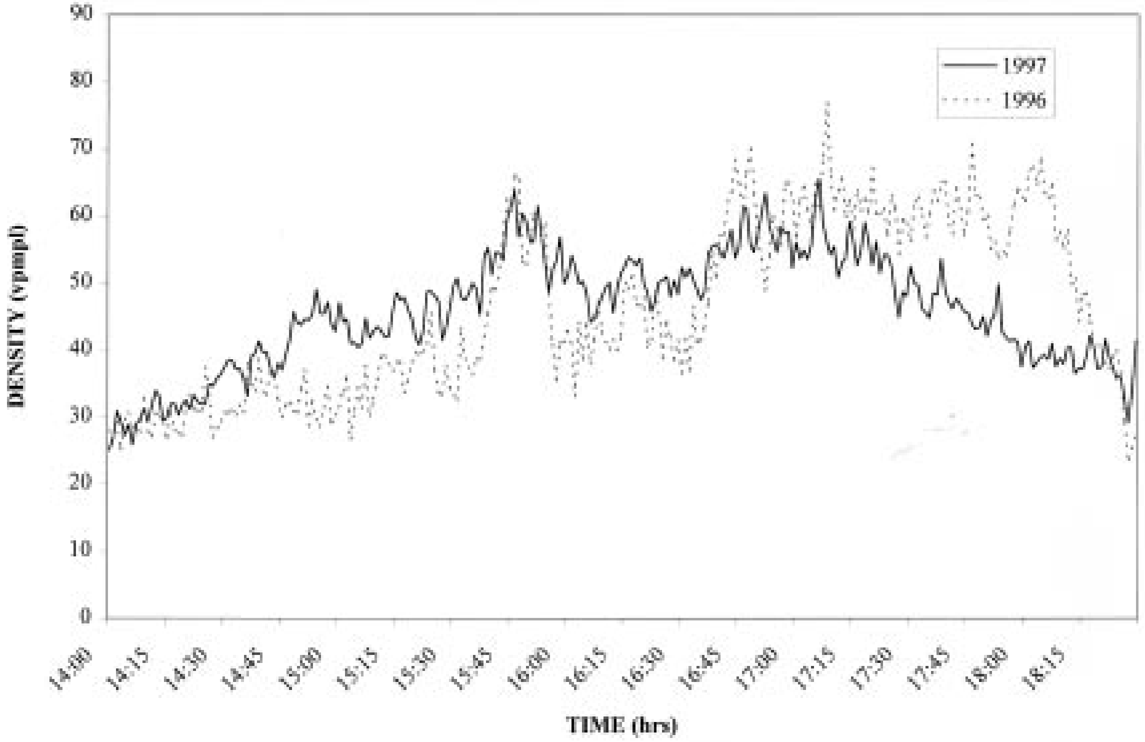

FIGURE 7 I-64 WB downstream of McCausland (all lanes).

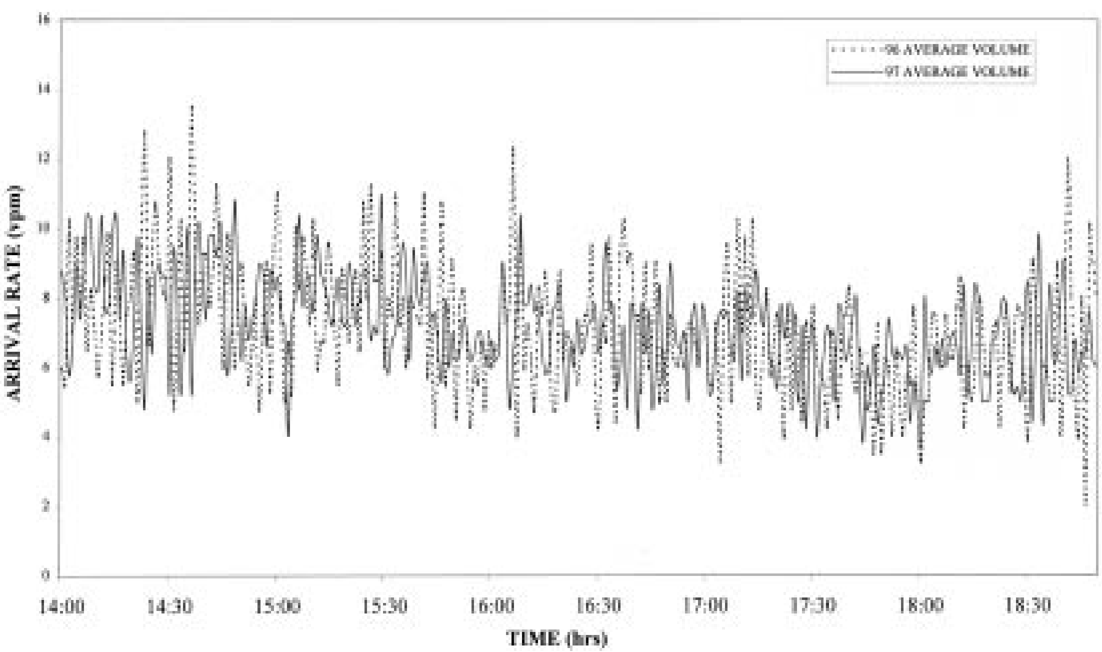

FIGURE 8 Ramp metering effects on mean arrival rates.

Observation of the arrival rate for the McCausland ramp over the P.M. peak period was also performed. A comparison for both years is shown in Figure 8. The 1996 data shows a lack of consistency in the arrival rate. The rate may be large at one point, but significantly lower a short time span later. Now observing the 1997 data, it can be seen that the arrival rate is more uniform due to the presence of the ramp meter. This more uniform arrival rate allows vehicles to enter the mainline traffic stream in a less disruptive manner. Even though the mainline may be operating above critical density, the resulting effect is more consistent densities on the mainline and therefore less probability of shock wave production.

CONCLUSIONS

Traffic engineers have developed many empirical solutions to the problems of operating transportation systems. Subsequent studies in traffic flow theory have verified that approaches developed by experiment and observation may be the best solution. Speed, volume (flow), and density (concentration) are the three primary measures used for the empirical study of traffic operations. The relationships among these three factors are important in providing the basis for the selection of measures of effectiveness and the definition of level-of-service for freeway segments (*6*). In this study, the numerical results indicated that the capacity, free flow speeds, and critical speeds varied for different lanes. However, it was observed that the critical densities were relatively similar for all lanes at the same location. This indicates that density may be a rather important and more reliable measure than volume and speed in the determination of freeway capacity. It also suggests the critical importance of the study of volume-density relationships in order to understand the characteristics of freeway dynamics. Furthermore, the series of plots provide a clear picture of the dynamics and progression of the traffic stream. In addition, such studies as this one form a comprehensive database for the development and implementation of congestion management techniques utilizing ITS.

Studies of the ramp volumes and arrival patterns are important in locating the trouble areas for freeway corridors. Data observed in these areas allow further analysis of critical traffic flow characteristics that is important in concluding congestion mitigation alternatives. The above studies regarding arrival rates and patterns, combined with the volume, speed, and density measurements provide important information for the understanding of the freeway traffic characteristics. The output of these analyses form the basis of the development of the Advanced Traffic Management Systems (ATMS) programs. Such a large traffic database is important in successfully developing a macroscopic traffic flow simulation model. The input of such comprehensive traffic data into the model improves the accuracy of the dynamics of the freeway's operation, and allows meaningful traffic control such as ramp metering to be implemented.

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Strategies for Improving Roadside Safety

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While there has been impressive progress to improve highway safety in the U.S. since the 1950s, there are still significant problems to be addressed. Roadside safety is one such problem. It accounts for about one-third of the fatalities and injuries and it has been estimated to impose an $80 billion dollar annual cost on the society. The NCHRP embarked on a comprehensive effort to address this problem in 1994. A distinguished group of professionals was assembled to review the problem, identify possible solutions, and define impediments to resolving the problem. This group determined that all aspects of the problem need to be considered in the search for means to address the problem. They subsequently sought inputs from other professionals knowledgeable about the roadway, driver, and vehicle aspects of the problem. They structured there possible solutions to the roadside safety problem in the form of a strategic plan. This plan is based upon five missions, each having a series of goals and objectives. Specific actions that should be undertaken are identified under each objective. Needs of information or research were listed where additional actions that are believed important have been identified. The outlines many "strategies" that can be undertaken to start addressing the problem now. It also provides a roadmap for efforts to increase awareness of the problem, establish the information resources needed to monitor roadside safety, measures that can keep the vehicle on the roadway, reduce the likely of a crash if a vehicle inadvertently leaves the roadway, and to minimize harm if a roadside object is struck.

INTRODUCTION

There has been a half-century long commitment in the United States to improve all aspects of highway safety. While efforts to improve roadside safety originated in the early years of the automobile, the major impetus in roadside safety began in 1960 with the Stonex's paper "Roadside Design for Safety" (*1*). Prior to that little attention was given to safety of the roadside and run-off-the-road accidents were attributed to "the nut behind the wheel." His paper identified common roadside hazards such as unprotected bridge abutments, blunt guardrail ends, rigid supports for street lights and signs, tree and utility poles, steep side slopes, and unsafe ditch sections. Solutions to these problems have evolved since then, but the problem still exists.

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THE ROADSIDE SAFETY PROBLEM

Although catastrophic accidents involving airliners, ships and trains receive a great deal of media attention, 94 percent of all transportation fatalities occur on roadways and highways. These traffic deaths, occurring one or two at a time all over the nation on each day of the year, do not receive widespread attention but the cumulative toll is more than 40,000 deaths and more than 3.5 million injuries (*2,3*). The safety of U.S. highways has improved since the 1950's as a result of efforts on many fronts. The number of highway fatalities has declined from a peak of almost 59,000 in 1969 to slightly less than 42,000 in 1995. The decrease is even more impressive when considered in terms of the great increase (doubling) in the vehicle miles of travel that has occurred resulting in a decrease in the fatality rate (i.e., the number of fatalities per 100 million miles traveled) from 5.5 in 1966 to 1.7 in 1995. This makes U.S. roads the safest of any industrialized nation.

Yet, even with this progress, the safety of travel remains an important concern because the number of fatalities and injuries is still too large. Roadside safety is a major component of the highway safety problem in the Unites States. Analysis of available data indicates that:

- On an average day in 1996, over 110 people were killed and over 6,000 persons sustained disabling injuries in crashes on U.S. highways. These grim statistics become even more startling when you realize that more people lose their lives on the highways each week than lose their lives in airline crashes annually.
- Roadside crashes account for one-third of the total U.S. fatalities each year. Each year, more than 14,000 people are killed and almost 1,000,000 people are injured in roadside crashes in the U.S (*4*).
- Roadside crashes cost society $80 billion per year. Traffic crashes impose a tremendous cost to society in medical costs, worker losses, property damages, and emergency services, in addition to pain and suffering. The annual societal cost of roadside crashes is more than three times the annual governmental expenditures on highways in the U.S. (*5*).
- Crashes with trees account for about 10% of the national fatalities. Approximately 3,500 fatalities and 90,000 severe injuries occur annually as a result of crashes into trees. Approximately 19% of the estimated roadside losses are attributed to tree crashes.
- Crashes with utility poles account for about 5% of the national fatalities. Approximately 1,500 fatalities and 110,000 severe injuries occur annually as a result of vehicle impacts into utility poles. About 14% of the estimated roadside losses are attributed to such crashes.
- Rollovers are the most severe type of roadside crashes. Rollovers occur in only 15% of roadside crashes, but are responsible for more than 25% of all roadside fatalities. Nearly three-quarters of rollovers occur on rural 2-lane roads where right-of-way is limited and lower, older design standards have been used. This

problem is exaggerated because today's hottest selling vehicles, pick-ups, sport utility vehicles, and vans, are more likely to roll.

These facts indicate, that despite dedicated efforts over the past three decades, the roadside safety problem remains a major source of injury, death, and economic loss.

ADDRESSING THE ROADSIDE SAFETY PROBLEM

Highway crashes occur when something goes wrong in the driver-vehicle-roadway system. Each element of this dynamic system has its limitations and is subject to failure as noted below:

- The Driver - Physical and mental limitations affect a driver's performance. Age, intoxication, fatigue, or emotion can affect the driver's ability to perceive a hazard, make decisions, and take action and thereby increase the probability of a crash. In addition, benign non-driving activities such as talking, eating, smoking, or adjusting the radio can distract a driver at an inopportune moment. Most drivers have received only minimal training for the driving task, and very few receive training to control a vehicle in incident situations or when it leaves the roadway. Overconfidence in driving abilities or ignorance of safety hazards can lead to risky behaviors, such as speeding, aggressive driving, inattention, and failure to "buckle-up."
- The Vehicle - A driver's control can be affected by vehicle and roadway conditions. Crashes often occur when the laws of physics overcome the driver's ability to control them as provided by the design and condition of the vehicle. Vehicle design can affect the likelihood of a crash and the severity of injuries resulting from a crash. Vehicles need to be compatible with roadside hardware to maximize the level of safety provided. Emerging technologies are expected to improve safety by providing supplemental information and/or automatic controls on the vehicle.
- The Roadway - Roadway geometry, pavement condition, and traffic control devices affect a driver's ability to maintain vehicle control and stay on the roadway. Ideally, clear areas can be provided along the highway to allow an errant vehicle to come to a safe stop or regain control and return to the roadway. Often, it is difficult to provide clear areas, so roadside safety hardware, such as, median barriers and crash cushions, are provided near hazardous roadside objects to protect the motorist in the event of leaving the roadway. There are many potential roadside hazards and treatments can be expensive, posing a major design challenge.

These elements must work in harmony, if the system is to provide mobility at an acceptable level of safety and a reasonable cost.

EXTERNAL INFLUENCES

Addressing the problem will not be easy due to the ever-changing dimensions of the problem. External factors that must be considered include:

- Changing vehicle fleet characteristics and increasing variance in vehicle size and weight
- Growing volumes of traffic across all periods of the day
- Growing demands by non-vehicle users of the highway
- Continued growth of urbanized areas and limits on right-of-way availability
- Deterioration of the infrastructure
- Increased costs associated with construction, repair, and maintenance of highways
- Aging driver population
- Increased competition for government resources.
- Resistance to highway improvements for environmental reasons.

Further, it must be recognized that the roadside safety problem is distributed over the almost 4 million miles of public roads in the U.S.

A VISION FOR IMPROVED ROADSIDE SAFETY

In an effort to address the roadside safety problem, the National Cooperative Highway Research Program (NCHRP) assembled a distinguished group of experts and charged them with the task of identifying ways to improve roadside safety (*6*). In their deliberations, they formulated a vision - A highway system where drivers rarely leave the road; but when they do, the vehicle and roadside work together to protect vehicle occupants and pedestrians from serious harm. To achieve this vision, the experts outlined five basic missions for transportation agencies:

- Mission 1 - Increase the awareness of roadside safety and support for it. Roadside safety cannot be enhanced until the public, decision makers, and other groups see it as a problem. Significant improvements to roadside safety will require a coordinated effort of transportation agencies, manufacturers, departments of motor vehicles, advocacy groups, and others. Additional funding at the federal, state, and local levels is needed to implement critical improvements and upgrade processes for safety management. Coalitions of government, industry, and civic partners should be formed to promote improved roadside safety.
- Mission 2 - Build and maintain information resources and analysis procedures to support continued improvements in roadside safety. A better understanding of the driver-vehicle-roadway relationship in roadside crashes is needed so that cost-effective remedies can be identified. Improved roadside and roadway inventory systems and better crash data are needed to provide highway designers, safety analysts, decision makers, and researchers with much-needed information. State-of-the-art computer analysis techniques can be used to monitor changing conditions and their influence on roadside crashes, provide better information to decisions makers, and/or simulate crash events. Safety audits, safety management systems, and other techniques can assure that efforts to improve roadside safety are most effective.
- Mission 3 - Keep vehicles from leaving the roadway. Roadside crashes occur when vehicles leave the roadway, as the result of driver error, vehicle failure, highway conditions, traffic situations, or environmental factors. Improved highway designs and better control of traffic operations can minimize the occurrence of events that lead to loss of vehicle control and roadside encroachment. Similarly, improved maintenance of highways and vehicles and innovative vehicle-based systems can help keep drivers on the road. Education, altered insurance regulations, and traffic law enforcement can promote appropriate driver behavior important to staying on the road.
- Mission 4 - Keep vehicles from overturning or striking objects on the roadside when they do leave the roadway. The chances for severe injury or death increase greatly when an errant vehicle overturns or hits a fixed object. Utility poles, trees, steep side slopes, drainage facilities, and roadside hardware are potential hazards found along the roadside. The slopes and configurations of ditches on the roadside should be designed to reduce the

chances of rollover. Hazardous fixed objects should be held to a minimum and protected if they must remain. Vehicles should be designed to increase stability in run-off-the-road situations and driver needed to be educated about the proper actions in such situations.

- Mission 5 - Minimize injuries and fatalities when overturns occur or objects are struck in the roadside. When a vehicle rolls or strikes a fixed object, the risk of injuries can be reduced if the occupants are wearing seat belts and the vehicle has airbags and the vehicle has been designed to be crashworthy. Crash severity can also be reduced by better roadside hardware designs that absorb greater amounts of impact energy assuming that these devices are properly selected, installed, and maintained. Better emergency response after highway crashes can also contribute to reductions in the number of fatalities.

Defining the missions was only the first step in developing the roadside safety strategic plan. For each mission, a set of goals have been defined which indicate the desired outcomes. The five fundamental missions and the associated goals are presented in Table 1. These missions and goals have been further subdivided into objectives, actions, and needs so that the roles of various agents can be highlighted, action agendas formulated, and needs for research identified. A considerable degree of detail has been added to the strategic plan based upon inputs from many professionals with diverse backgrounds resulting from an interactive development process.

TRANSLATING THE VISION INTO ACTIONS

The efforts to identify actions in the strategic plan provided the basis for defining specific activities that could improve roadside safety or to reinforce continuance of current activities that are effective. As the strategic plan is being finalized, lists of action items are being developed for planning, design, construction, operations, maintenance, administrative, enforcement, and driver licensing functions of DOTs. In addition, the strategic plan provides a basis for identifying roles for vehicle designer, roadside hardware manufacturers, and other related disciplines. These allow consideration of the trade-offs between agents to determine where it is most effective to address the problem.

The details provided in the strategic plan provide the basis for further analysis. First, decomposition of the strategic plan also reveals areas where synergy can be achieved through the coordination of actions. It is also possible to isolate all actions that can be directed to specific problems for program development purposes. For example, crashes with trees are known to be a serious problem. By tagging all elements of the plan related to that issue, it is possible to recognize all necessary aspects of a program. The strategic plan also permits interaction analysis to provide the foundations for the establishment of coalitions, linking of activities, and finding places where multiple purposes can be achieved.

The NCHRP effort identified many strategies and actions for addressing roadside safety problems. Some of these actions can be

TABLE 1 Missions and Goals of the Strategic Plan

Missions	Goals
1 - Increase the awareness of roadside safety and support for it.	1-A network of partners. 2-Greater public awareness of the importance of roadside safety. 3-Increased emphasis by partners and better communication between them. 4-Sufficient fiscal resources to address critical needs. 5-Programs to disseminate roadside safety information. 6-Integration of roadside safety into SMS. 7-On-going process for updating the plan.
2 - Build and maintain the information resources and analysis procedures.	1-Improved roadway & roadside inventory data systems. 2-Comprehensive roadway safety information resources. 3-Effective tools and methods for safety analyses. 4-On-going programs to monitor roadside safety.
3 - Keep vehicles from leaving the roadway.	1-Improved highway designs and standards. 2-Improved traffic operating environments. 3-Improved vehicle-based systems to keep driver on the road. 4-Improved driver performance & behavior. 5-Sufficient levels of highway & vehicle maintenance.
4 - Keep vehicles from overturning or striking objects on the roadside when they do leave the roadway.	1-Improved roadway design to reduce vehicle overturning. 2-Improved vehicle designs to increase stability. 3-Reduced numbers of hazardous objects on the roadside. 4-Improved driver performance in run-off-the-road situations
5 - Minimize injuries and fatalities when overturns occur or objects are struck in the roadside.	1-Improved roadside safety hardware. 2-Improved vehicle-roadside compatibility & crashworthiness. 3-Proper selection, design, installation, and maintenance of roadside features. 4-Improved emergency team response. 5-Increased seat belt use and effectiveness.

undertaken immediately, at little cost and others will take time and larger amounts of money to implement. Examples of some important actions are:

- Install shoulder rumble strips to alert drivers - Rumble strips located on the highway shoulder are effective in alerting drowsy or inattentive drivers when their vehicle is drifting off the roadway. These have been effectively installed on some interstate highways, but their potential use extends to rural two lane roads as well.
- Strategically remove or shield trees or utility poles close to the roadway - Pole placement policies need to be re-examined and programs need to be developed to reduce the number of poles on the roadside. Strategies for removing trees in particularly hazardous locations on the roadside (e.g., on the outside of tight curves) are needed to enhance the safety of the traveling public while maintaining the aesthetics of our highways.
- Use public service announcements and citizen initiatives to increase awareness - Impressive reductions in DUI have resulted from citizen initiatives such as MADD and SADD. Similar actions are needed to increase awareness of roadside safety, encourage greater use of seat belts, or discourage excessive speed on curves.
- Improve safety management systems - Transportation agencies need better and more accurate data and tools to manage highway safety. These systems can improve the practices for identifying hazardous locations, link highway features and accident data, assistance in making optimal decisions related to limited resources, and allow monitoring of changing traffic conditions that might increase roadside crashes. While most agencies maintain accident records and highway inventories, there is a need to upgrade, expand, and link this information to increase basic capabilities for safety management on a continuing basis.
- Implement proactive highway maintenance programs - Adequate maintenance will assure that the surface provides the skid resistance necessary to negotiate curves, minimize erratic maneuvers to avoid potholes, and eliminate shoulder drop-offs that can lead to loss of control. Attention should also be given to maintenance of adequate sight lines for the driver.
- Improve driver education programs - Drivers need to learn about the hazards of the roadside and rationale of traffic controls to help them to avoid situations where they could leave the roadway. It is even becoming possible to use simulators to train drivers in maneuvering through unusual situations, such as leaving the roadway.
- Increase speed enforcement at locations with known roadside safety problems - Plan police enforcement activities at or near locations where roadside crashes are known to occur. Use automated enforcement methods when they are available to increase enforcement coverage.
- Promote development of innovative technologies to keep vehicles on the road - Utilize ITS technologies to develop systems that will help drivers stay on the road or avoid hazardous objects or automatically prevent collisions.
- Improve the proficiency of persons responsible for roadside safety - Support safety professionals with a legacy of knowledge, interactive networks for information exchange, national databases, and updated training courses with upgraded delivery systems.
- Improve vehicle design to increase compatibility with roadside hardware - Upgrade vehicle designs to assure compatibility (e.g., establish minimums and maximums for bumper height) in both vehicle-to-hardware and vehicle-to-vehicle crashes.
- Improve hardware design - Develop improved methods for the design and testing of safety hardware including simulation methods. Exploit the properties of new materials in roadside safety hardware and review opportunities for new hardware. Assess the functionality of hardware in field applications.

There is a strong consensus that these and other actions can lead to major reductions in the deaths and injuries that are occurring on the roadside.

SUMMARY AND CONCLUSIONS

The efforts under the NCHRP Project are believed to have laid the groundwork for further efforts and a coordinated approach to improving roadside safety. More effort is needed, particularly to assure that all perspectives are considered and that the most cost-effective options are selected. A great deal has been accomplished in improving the effectiveness of roadside safety hardware during the past several decades. The always-changing vehicle fleet and highway environment do not allow the roadside safety community the luxury of complacency. There are significant challenges ahead in improving roadside safety. These challenges can only be met by openly discussing difficult issues as they emerge and focusing the efforts all those with an interest in roadside safety on coordinated action.

ACKNOWLEDGMENTS

The strategies described in this paper were forged through from the contributions of many professionals over the last three years. Over 3 person-years of time was accumulated in 11 meetings, some having more than 100 participants. In these meetings, the vision, missions, goals, objectives, actions, and needs were incrementally identified and continually refined. The NCHRP Project 17-13 Panel participated in all of these meetings and served to assure that the strategies reflected a comprehensive, unbiased look that the roadside safety problem.

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Guidelines for Developing a Guardrail Manual for Low-Volume Roads

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Designing and maintaining a "forgivable roadside" is an important concept to promote highway safety. However, it is not always possible, feasible or "affordable," particularly on Low-Volume Rural Roads (LVR). The results of recent research have provided reasonably good guidelines for roadside safety, barrier rail and end treatments on state highways and other major roads. Generally these sophisticated models and guidelines do not apply to LVR. Local governments have tens of thousands of miles of LVR where rights-of-way are narrow, clear zones and traversable slopes cannot be provided and budgets are inadequate for a multitude of competing problems. The Kansas Department of Transportation (KDOT) contracted with the authors to review the state-of-the-art of roadside safety, interview local roads personnel, study local roadside situations and develop guidelines on low-volume roads, roadside safety and barrier rail use. The computer program ROADSIDE was adapted to LVR conditions and simplified guidelines for its use where developed for LVR conditions. Computer results based on a range of Kansas LVR conditions were compiled and presented in a manual for easy use by Kansas personnel with responsibility for LVR safety. The paper presents information on adapting ROADSIDE for use on LVR and a sample of results from the Kansas Manual. Key words: guardrail, roadside safety, clear zone, low-volume road safety.

INTRODUCTION

Kansas State University (KSU) recently completed a research contract with Kansas Department of Transportation (KDOT) to develop guidelines for the use of guardrail on low-volume roads (LVR) in Kansas according to safety and effectiveness. The computer program ROADSIDE is widely used to assist designers in making informed choices regarding alternate, guardrail design concepts. ROADSIDE follows the Barrier Guide cost-effective methodology. The ROADSIDE program was adapted to Kansas LVR parameters. LVR are generally defined as roads with 400 ADT, although many LVR's have much lower ADT's.

A comprehensive review of the research literature was conducted to explore and gather information on the use of guardrail on LVR. The purpose of this information search was to identify the general elements used to determine the need for guardrail on LVR and to review any specific guidelines already in use by other states. The principal findings from this literature review are presented below.

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Existing Guidelines on LVR

Currently most states are using or developing guidelines for the installation of guardrail on state highways based on the *Roadside Design Guide*. Published by the American Association of State Highway and Transportation Officials. These AASHTO guidelines recommend guardrail if the consequences of hitting a roadside fixed object or running off the road would be more serious than those associated with striking the guardrail *(1)*. The guidelines to warrant guardrail should consider two roadside conditions: embankment cross sections and fixed objects. The AASHTO guidelines do not have embankment warrants specifically for LVR.

Guardrail Guidelines for Roadside Embankments

According to the *Roadside Design Guide*, a guardrail is warranted relative to roadside embankments based on the fill section height and the reciprocal of the fill section slope, without considering the ADT. Arnold mentions that several states use the *Roadside Design Guide* warrants directly, or in a modified form, regardless of ADT *(2)*. Many states use the computer program ROADSIDE. However, this program has to be adapted for LVR. Some states have done this and developed curves and tables for LVR.

North Carolina considers speed and the length of embankment. For example, for an ADT of 400, 55 mph (88.2 kph), and a 2 1/2:1 slope, guardrail would be warranted on a 30 foot (9.1 Meters) embankment if it were over 150 feet (45.7 meters) long, on a 20 foot (6.1 meters) embankment if it were over 1,000 feet (305 meters) long, and on a 17 foot (5.2 meters) embankment if it were over 2,000 feet (610 meters) long.

The Arnold report presented guidelines to assist in evaluating the need for guardrail on secondary roads *(2)* (generally ADT's ≤ 10,000).

Missouri developed guidelines for guardrail on LVR in Missouri which considers the total life cycle cost of guardrail installations, physical characteristics of the hazard, severity or costs of accidents, and expected frequency of accident occurrence *(3)*. For design speeds of 40 and 50 mph, (64 and 80 kph) guardrail installation was found to be not economically justified for any of the conditions used (slopes 2:1, 3:1; lateral offset of the hazard of 6, 8, and 10 feet (1.8, 2.4 and 3.0 meters); and length of the hazard of 100, 500, and 1000 feet (30.5, 152 and 305 meters) regardless of the embankment height, when the ADTs were lower than 400 vehicles. For design speed of 60 mph (96 kph) the guardrail was warranted for ADTs between 350 and 400 vehicles only when the embankment height was of 20 feet (6.1 meters) for designs with cross slope of 2:1 or greater and certain combinations of lateral offset and length of the hazard: 6-100, 6-500, 6-1000, 8-500, and 8-1000 feet-feet (1.8-30.5, 1.8-152, 1.8-305, 2.4-152, and 2.4-305 meters-meters).

Guardrail Guidelines for Roadside Obstacles

Most states have followed the AASHTO guidelines (*Roadside Design Guide*, 1989) for roadside obstacles and clear zone distances. Arnold reported that twenty-seven of thirty-nine states contacted were using the AASHTO guidelines but with a policy that considered exceptions on low-volume, low-speed roads *(2)*. Exceptions to the AASHTO guidelines on LVR generally call for clear zone distances of 7 ft (2.1 m) to 10 ft (3.0 m) for ADT's between 400 and 750. Two states waived clear zone or do not install guardrail with design speed < 40 mph (64 knp) or ADT < 1250.

Pigman and Agent discussed the development of warranting guidelines for clear zones in the state of Kentucky based on Kentucky accident severities (*4*). The computer program ROADSIDE was used to obtain the warranting guidelines.

Type of Guardrail Systems and their Costs

Once the guardrail is warranted the next problem that the local agencies face is to determine the type of guardrail needed for low volume, low speed roads. The AASHTO *Roadside Design Guide* (1989), describes a number of operational and experimental guardrail systems. Three of the operational systems that are currently being used in virtually all of the LVR applications throughout the USA are (Stephens, 1992): the G-1 cable system, the G-2 weak post W-Beam, and the G-4 strong post W-Beam. The G-1 and G-4 systems have variations in the type of post used.

The Roadside Approach for Performing Guardrail Assessments

Although the Roadside Design Guide presented warrants for the need for guardrail based on embankment and roadside obstacle criteria, the recommendation was made that highway agencies develop specific guidelines for their agency based on a cost-effectiveness selection procedure based on the application of the computer program ROADSIDE *(1)*. The procedure to evaluate alternatives should be based on a cost-effectiveness analysis with or without the ROADSIDE computer program. ROADSIDE allows the user to calculate the present worth and annualized cost (including accidents, installation, repair and maintenance) of a specific safety improvement at a specific location. The real value of the program is that it allows a cost comparison of alternative improvements, including the do-nothing alternative.

The ROADSIDE program was adapted to analyze guardrail on Kansas LVR. Each jurisdiction must input their own jurisdiction - specific data to obtain good local results. This process is emphasized in this paper and is presented below. A sample of Kansas results are presented to illustrate the type of results possible.

The third computer screen in ROADSIDE allows input of the variable data specified to an alternative being evaluated. Following is a discussion of how each of the items, 2 through 15, was decided on for applying ROADSIDE in the embankment and fixed object analyses for Kansas:

- Item 2. Traffic Volume. The traffic volume was varied between 400 vehicles per day (vpd) to 100 vpd with a constant growth factor of 1% per year.
- Item 3. Roadway Type. A two-lane, two-way road was used for the analyses by setting an undivided roadway with one lane adjacent to the hazard in ROADSIDE. The lane width was assumed to be 3 meters.
- Item 4. Adjustment Factors. ROADSIDE allows adjustment to the baseline encroachment to account for roadway curvature and grade. For the analyses, a value of 1.0 was used.
- Item 5. Traffic Volume and Encroachments. ROADSIDE calculates this item by assuming splitting of the previously input traffic volume evenly by direction, applying the encroachment defined earlier, and adjusting the baseline encroachment by the factors in item.
- Item 6. Design Speed and Encroachment Angle. The following speeds were used in the calculations: 50, 60, 70, 80 and 90 km/hr. The program default encroachment angles were used in the analyses.
- Item 7. Hazard Definition. In ROADSIDE, a hazard is defined with a lateral offset (A) from the edge of the nearest driving lane, longitudinal length (L) parallel to the roadway, and width (W) - generally perpendicular to the roadway. Values used in the Kansas study are discussed below.

Lateral Offset

On Kansas unpaved rural roads, there is no way to describe or show a typical section from which to measure the offset. This must be determined in the field. Depending upon local blading practices, the usable roadway width (traveled way) may vary from one local jurisdiction to another and in fact may vary from before and after a section is bladed. The only practical solution is for the person in charge of road and street operation and maintenance to determine the record the outer limits of the normally traveled way.

The following parameters were used in the analyses:

- For embankment analysis

In the embankment analysis, 60 m (200 ft) was used for the length of both the guardrail and the embankment. Different lengths were tested with the ROADSIDE program and 60 m yielded the smallest height of fill at which guardrail became cost-effective. Thus, this value is conservative on the side of safety.

Length: 60 m (200 ft.) For both (guard and embankment), 6 m (20 ft) on culverts

Width of Guardrail: 0.3 m (1 ft)

Width of Embankment: variable depending on embankment height and cross slope.

Foreslopes: 1:2, 1:3, 1:4

Height: 0 to 10 m (0 to 32.8 ft)

Lateral Offset for Guardrail: 0.0, 0.3, 1.3, Sm

Lateral Offset for Embankment: 3 m (10 ft)

- For the fixed objects analysis

For the fixed objects analysis a 60 m (200 ft) section of guardrail was compared with a 0.3m (1ft) by 0.3m (1ft) fixed object.

Length. 0.3m (1 ft)

Width. 0.3 m (1 ft)

Lateral Offset of the Fixed Objects. 0, 0.3, 1, 2, 3, Sm

- Item 8. Initial Collision Frequency. These values are calculated by ROADSIDE based on previously input data.

- Item 9. Severity Index. Severity indexes, (SIs) are estimates of the societal costs associated with an average accident with a given feature. ROADSIDE uses the Sis to determine the cost of accidents. Five values are needed to perform the analyses. One for each: the upstream side, the upstream corner, the force, the downstream corner, and the downstream side of the texture. For both, embankment analysis and fixed objects analysis, the Sis used were taken from the ROADSIDE Users Manual, Appendix A (A Cost-Effectiveness Selection Procedure; a user's guide and documentation for the computer program ROADSIDE.)
- Item 10. Project Life and Discount Rate. For the purpose of this project, an anticipated life of 20 years and a discount rate of 4 percent were used.
- Item 11. Installation Costs. Based on the data provided by KDOT the installation cost was $82.50 linear meter ($25 per linear foot) for G4 (2W) - 6" x 8" wood.
- Item 12. Repair Cost/Accident. For the purpose of this project, $500 was used as the average cost of repairing hit guardrail.
- Item 13. Maintenance Cost/Year. Based on the data provided by KDOT, the maintenance cost was $3.00 per linear meter ($1.00 per linear foot).
- Item 14. Salvage Value. For the purpose of this project, the salvage value was assumed to equal $0.
- Item 15. Present Worth/Highway Department Costs. ROADSIDE calculates the total present worth (TPW) of accident costs and highway department costs incurred over a specified analysis period (the project life) using the following equation:

TPW = CA (KC) ± CI + ARC + CM (KT) - CS (KJ)

where:

CA = Accident cost based on initial collision frequency

KC = Factor to account for project life, discount rate, and traffic growth rate

CI = Installation cost

ARC = Present worth of accident report cost = 1 KC(CDi) (Cfi)

Cdi = Average collision damage repair costs for sides, corners, and face Cfi.

Initial collision frequencies for sides, corners, and face

CM = Annual maintenance cost

KT = Factor to account for the project life and the discount rate

CS = Salvage value of feature being studies

J = Factor to account for the project life and the discount rate

ROADSIDE also calculates annualized costs, which are obtained by multiplying present worth values by a capital recovery factor (CRF).

RESULTS

Results are from a cost-effectiveness analysis based on several assumptions, that are both input to the ROADSIDE program and inherent within the program; therefore, the results should be used with judgement after considering other, non-economic factors. A sample of Kansas results are presented below. Detailed results were incorporated in approximately 140 graphs and tables and several summary tables *(5).*

Roadside Obstacle

RCB Culvert - Straight Wings

Based on the total life cycle cost analysis, the guardrail was economically justifiable for speeds of 90 km/h, ADTs of 300 or higher and culvert end height of 2.4 meters. The results indicated that the guardrail was not economically justified if the culvert's lateral offset from the nearest driving lane was two or more meters.

RCB Culvert - Flared Wings

The study results indicated that, under all conditions, guardrail was not economically justified if the culvert's lateral offset from the edge of the nearest driving lane was more than three meters. For some other conditions, installation of guardrail was economically justifiable.

RCP Culvert - Pipe/Headwall

The study results indicated that the guardrail was not economically justified if the average daily traffic was 100. Guardrail was economically justifiable for some other conditions.

Utility Poles

Based on the total life cycle cost analysis, the guardrail was economically justifiable for speeds of 90 km/h, ADTs of 400 and lateral offset of 0.0 m and 0.3 m.

Embankments

The study results concerning guardrail installation on roadside embankments indicated that the guardrail was not economically justified for either 1:4 and 1:3 foreslopes with slope surface condition B, regardless of the design speed and ADT. For 1:3 foreslopes with slope surface condition C, ADT of 400, speed of 90 km/h and height of fill of four or more meters installation of the guardrail was economically justifiable. Guardrail was economically justifiable on most 1:2 foreslopes with surface condition B and C. (Surface conditions A, B and C relate to the condition of the roadside with C being the roughest.)

CONCLUSIONS

Application of the ROADSIDE microcomputer program produced valuable results that should provide for a more cost effective use of guardrail on rural, low-volume roads in Kansas. It is important to note that the procedures and input parameters used in this study were based on the latest Kansas information available at the time. Other jurisdictions should input parameters that apply to their jurisdiction. Also, considerations beyond cost-effectiveness may be important.

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The Importance of Measuring Fuel Consumption In Evaluating Electronic Clearance

Dennis Kroeger

Electronic screening of commercial vehicles at weigh stations is important to enforcement agencies and motor carriers as it allows both parties to use their resources more efficiently. This paper studies the effects of electronic screening on reducing the fuel consumption for motor carriers. The hypothesis that we tested was that a reduction or elimination of stops at weigh stations by participating transponder-equipped trucks will result in measurable fuel savings for those transponder-equipped trucks. The experiment demonstrates the impact of electronic screening on motor carriers by actually measuring fuel consumption between two trucks in the field under controlled conditions. The obtained results show that there are measurable fuel savings attributable to electronic screening. Key words: electronic screening, fuel consumption.

INTRODUCTION

With over 600 commercial vehicle inspection stations across the USA and the increasing emphasis on safety inspections for those vehicles, there are numerous occasions in which a commercial vehicle driver faces delays en route. Many of the nation's fixed inspection facilities were constructed 20 to 30 years ago. Consequently, the explosive growth in truck traffic has exceeded the station design specifications at many of these inspection stations. As truck arrivals exceed these stations' operational capacities, queues develop and drivers are delayed. Often, the backups require stations to close to avoid safety risks.

The national Intelligent Transportation System (ITS) program is designed to address these safety and productivity concerns along with focusing advanced technology on commercial vehicle operations (CVO). One part of this overarching program is to enhance mainline electronic clearance of CVO at the weigh stations. Several electronic clearance systems are in currently in operation across the country. These systems equipped with Automated Vehicle Identification (AVI) readers then identify the transponder-equipped trucks, verifying their size, weight, and credentials. When the information is read and verified, the trucks receive a signal, both visual and auditory. The signal directs the operator to either by-pass the weigh station or to enter the station (if there is a discrepancy in the size, weight, or credential information). The elapsed time of this communication from the truck to the weigh station, back to the truck, is less than one second. By electronic screening commercial vehicles on the mainline, thereby permitting compliant vehicles to bypass the weigh station, enforcement officials can better focus their resources on the non-compliant commercial vehicle operations. This paper describes the importance of measuring the fuel consumption of commercial vehicles as part of an overall evaluation of measuring the effects of electronic clearance at weigh stations.

This paper documents the application of a fuel consumption experiment. The paper illustrates the fuel consumption experiment through a case study of six typical weigh stations located on an interstate with high truck traffic volume. Although only six weigh stations were used in the case study, we have used the experiment to analyze electronic screening at for other weigh station designs.

METHODOLOGY

Our experiment is to determine if mainline electronic clearance produces significant fuel savings for motor carriers. The test used to make this determination applied accepted Society of Automotive Engineers' (SAE) guidelines. The prescribed method directed one truck to stay on the mainline and a second truck to enter the weigh station. The second truck then either stops or slow down at the scale, depending on the design of the weigh station. The fuel used by each truck was then precisely measured to determine the fuel used by each vehicle. The difference in fuel used was the estimated savings of fuel attributable to a truck bypassing a weigh station.

Test Procedures

The fuel consumption test was based upon the Society of Automotive Engineers Recommended Practice (SAE Type II Fuel Consumption Test). This experiment was performed to determine if the reduction or elimination of stops at weigh stations by trucks equipped with transponders results in measurable fuel savings for each participant truck. One truck, termed the control truck, always bypassed the weigh station. The other truck, termed the test truck, alternated between control runs in which the weigh station was bypassed, and experimental (or test) runs in which the weigh station was entered. At each of the test sites, the baseline fuel con-

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sumption difference between the two trucks was measured (when both trucks bypassed the weigh station.) Then the experimental fuel consumption difference was measured between the two trucks (when one truck bypasses the weigh station and one stops or slows at the weigh station.) This procedure includes two forms of control. First, during each run the control truck and test truck encountered almost identical conditions, therefore any observed differences were due to experimental or vehicle differences. Second, the use of baseline runs provided estimates of the fuel consumption differences due to vehicle variances (tire tread, engine performance, etc.) The baseline runs, therefore, provided a control for the experimental runs. For the purposes of consistency and reliability, the same equipment and drivers are used through out the duration of the experiment.

In an attempt to obtain "real-world" results as closely as possible, standard issue equipment was used for these fuel consumption experiments. The only modifications made to the vehicles were the addition of the portable fuel tanks, utilized for the precise measurement of the fuel usage. Prior to beginning the tests, the tractors were equipped with "quick-connect" fittings by the motor carrier, permitting the easy installation and subsequent removal of the portable fuel tanks and fuel coolers. No other alterations were made to the tractors or trailers.

There were three basic weigh station design types used for the fuel consumption experiment: the static scale design, the ramp weigh-in-motion (WIM) design, and the high-speed ramp WIM design. These are the most common design types encountered by commercial vehicle operators. To gain the most from the experiment, we decided to use the most efficient and least efficient design types in order to determine a proper range of fuel consumption. To that end, two static scale design types were chosen, two ramp weigh-in-motion design types were chosen. Finally, high-speed ramp WIM design was selected. To recap the site location decisions, these sites were chosen based upon their topographical layouts, varying traffic volumes, and efficiency in design. One set of static scales has flat terrain and contains moderate traffic volume. Conversely, the second set of static scales is hilly with very heavy volume of vehicle traffic. Likewise, the first set of ramp WIM scales is laid out on flat terrain, with heavy volume of traffic. Meanwhile, the second ramp WIM scale layout is hilly with a moderate amount of traffic. The high-speed ramp WIM design is the most efficient design layout of the group. It is on flat terrain with a light to moderate traffic volume. This design was termed "High-Speed" Ramp WIM because the design allows trucks to use the bypass lane at speeds of up to 45 mph (72 kph), which is considerably higher than other bypass lanes at other facilities.

Data Analysis

To evaluate the hypothesis that trucks bypassing weigh stations consume less fuel, the fuel consumption has been measured using the test procedures described earlier in this document. The goal is to provide a measure of the expected savings (gallons of fuel per weigh station bypassed) for different weigh station designs. One can formally test the hypothesis of no savings but this is no of much interest here. Instead this experiment focuses on providing a valid estimate along with estimates of the possible variation due to a variety of uncontrolled factors.

Input Data

Data worksheets filled out during the test runs included for each truck: condition (bypass or stop), distance traveled, fuel consumed, time stopped during the run. Wind speed and ambient temperature at the time of each run were also recorded. Analyses of previous pilot data and the present data suggest that wind speed and temperature are not needed for the analysis. This is expected due to the fact that the test and control trucks face the same conditions in each run. The basic measurement that we use for each run is the fuel consumed difference in gallons per weigh station between the control truck and the test truck.

Methods

The approach that is taken here uses two-sample statistical methods (comparing control or baseline runs to experiment runs). Symbols required to carry out the analysis are defined as:

M*b*=the mean observed fuel consumption difference between the control truck and the test truck during the baseline runs

M*e*=the mean observed fuel consumption difference between the control truck and the test truck during the experimental runs

S*b*=the standard deviation of the observed fuel consumption differences between the control truck and the test truck during the baseline runs

S*e*=the standard deviation of the observed fuel consumption differences between the control truck and the test truck during the experimental runs

N*b*=number of baseline runs

N*e*=number of experimental runs

The values are provided in Table 1. Note that if identical trucks were used, then the expected value of M*b* would be near zero. The values observed there are far from zero, which points out the importance of baseline runs for establishing the relative fuel usage of the two trucks. The difference between M*b* and M*e* is a measure of fuel savings due to bypassing trucks. M*b* is lower than M*e* because the test truck is using more fuel in the experimental runs when it stops (or slows) for the weigh station.

The two sample pooled-t-statistic-based methods are used for drawing conclusions. To be specific, the runs are viewed as a sample from a population in which the team is interested (the savings that would be observed in a much bigger experiment involving more trucks). The observed difference between M*b* and M*e* is an estimate of the fuel savings expected. A 95% confidence interval provides a range of plausible values for the expected fuel savings. The formula used to provide the interval is:

$$\textit{Confidence Interval} = (Mb - Me \pm t * S \sqrt{1/Nb + 1/Ne})$$

Where S is a pooled (combined) estimate of variability that uses both the experimental and baseline runs and the t * value is a number that can be obtained from the tables to insure the 95% confidence statement is accurate (the t * value is generally about 2.0). More details about this procedure can be found in a variety of statistics texts including *The Basic Practice of Statistics* by D.S. Moore, W.H. Freeman and Co., 1994. The pooled procedures require that Sb and Se be approximately the same. The are almost identical at four of the five sites. The difference is more substantial high-speed WIM scale type, but still within the range for which pooled procedures are generally applied.

Table 1 Fuel Consumption Baseline and Experimental Results at Each Station

Station	Run Type	Number of Runs	Mean in Gal.	Std. Dev. in Gal.	SE Mean in Gal.
Station #1	Baseline	12	0.2314	0.0399	0.115
Static	Experimental	26	0.0523	0.0390	0.0076
Station #2	Baseline	15	0.1762	0.0399	0.0103
Static	Experimental	15	0.0124	0.0393	0.0101
Station #3	Baseline	23	0.1324	0.0438	0.0091
WIM	Experimental	25	0.0228	0.0402	0.0080
Station #4	Baseline	25	0.0171	0.0601	0.0120
WIM	Experimental	25	-0.0445	0.0648	0.0130
Station #5	Baseline	35	0.1703	0.0364	0.0860
WIM	Experimental	34	0.1184	0.0255	0.0044

Table 2 Estimated Mean Fuel Savings Per Station Bypassed

Station	Station Type	Estimated Fuel Savings in Gallons	95% Confidence Interval in Gallons
Station #1	Static	0.16	0.134, 0.194
Station #2	Static	0.18	0.151, 0.207
Station #3	WIM	0.11	0.085, 0.134
Station #4	WIM	0.06	0.026, 0.097
Station #5	WIM	0.05	0.037, 0.067

It is noteworthy that the sample size and the run-to-run variability (S*b* and S*e*) affect the width of the confidence interval. Sample sizes were chosen with the goal of obtaining confidence intervals sufficiently narrow for accurate inference.

CONCLUSIONS

Table 2 provides the mean fuel savings in gallons per weigh station bypassed and a 95% confidence interval for each site. All confidence intervals exclude the value zero which means that the fuel savings are "statistically significant." This statement is of limited value since it would seem evident that some fuel savings accrue to trucks that bypass weigh stations. While the results of the experiments are still preliminary, however, we are able to discuss the magnitude of savings.

The static scales provide the most dramatic savings with. The high-speed ramp WIM scale type performs as advertised with minimal fuel savings per station bypassed. The savings accrued at the ramp WIM set of scales are less dramatic. The last result from the ramp WIM station is surprisingly low, even with the hilly terrain surrounding the facility's area. The confidence interval here is widest because there was a great deal of variability from run-to-run.

The value of bypasses to an individual truck or firm depends on the number and nature of stations passes. For example, suppose a truck bypassed 100 static scale stations over a month. With fuel at $1.11/gallon this would mean savings of approximately $15/month.

Here is another example of expressing the fuel savings: Suppose 100 trucks are electronically cleared to pass a static scale type weigh station, this would mean fuel savings of 16 gallons for those bypasses. Therefore, following these experiments we can say that there are measurable fuel savings attributable to electronic clearance of commercial vehicles at weigh stations.

ACKNOWLEDGMENTS

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Weigh Stations' Capacity Enhancement Alternatives: A Comparison of Mainline Electronic Screening and Physical Expansion

ALIREZA KAMYAB

A number of weigh stations are unable to keep pace with current truck traffic levels. In response, state enforcement agencies are compelled to seek capacity enhancements for the weigh stations. One response is to increase physical capacity by adding a ramp weigh-in-motion (WIM) scale, a sorter and a bypass lane. Electronic screening, on the other hand, is a feasible option for increasing capacity without expanding physical infrastructure of the weigh station. This study compares the impact of these two alternatives in terms of travel time savings and enhanced productivity for a selected high traffic weigh station using a simulation model. The results indicate that physical expansion could improve a weigh station efficiency to some extent in a shorter amount of time than electronic screening programs. Electronic screening, however, eliminates a weigh station inefficiencies gradually but permanently when the number of transponder-equipped trucks increase significantly.
Key words: ITS/CVO, WIM, simulation modeling, weigh station, traffic modeling.

INTRODUCTION

Weigh stations are the primary regulatory compliance check points for commercial trucks. A large number of weigh stations are, however, unable to keep pace with current truck traffic levels, resulting in less efficient use of government resources and increased travel time for motor carriers. Weigh station administrators frequently close weigh stations to prevent queues from extending out on to the mainline. This queue spillover results in an inordinate number of bypasses without compliance checks (unauthorized bypasses).

In response, state enforcement agencies are compelled to seek capacity enhancements for the weigh stations. One response is to increase the physical capacity of the weigh station by adding a ramp weigh-in-motion (WIM) scale, a sorter and a bypass lane. This enables enforcement officials to monitor all arriving trucks and allow those trucks that do not exceed a set weight threshold to bypass the static scale and re-enter the mainline. Electronic screening, on the other hand, is a feasible option for increasing capacity without expanding physical infrastructure of the weigh station. Using automated vehicle identification systems and mainline weigh-in-motion scales, electronic screening allows for mainline weight checks and more targeted inspection of trucks.

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The impact of electronic screening in terms of travel time savings for motor carriers and enhanced productivity of the weigh station was studied elsewhere (*1*). The evaluation of electronic screening at the case study weigh station indicates a substantial reduction in travel time and the number of unauthorized bypasses. However, from the states' perspective, the benefits of electronic screening are not realized until a substantial number of trucks have been equipped with transponders.

It is estimated that nationally only two percent of the trucks are currently equipped with transponders. The rate at which trucks will become equipped with transponders is difficult to predict. Those administrators who seek a more imminent solution to enhance the productivity of their congested weigh stations may consider the physical expansion alternative.

This paper examines the impact of the physical expansion alternative at a weigh station. The primary objective of this study is to conduct a side by side comparison of the two alternatives at a weigh station assuming same traffic conditions. The case study weigh station has no ramp bypass lane and currently operates under high truck traffic. This study shows that physical expansion is able to temporarily alleviate the existing operational problems at the weigh station by improving the traffic throughput within the station. Eventually the growth in truck traffic will again overcrowd the weigh station. The electronic screening evaluation, however, indicates a more fundamental solution to the weigh station's capacity problems by making the mainline part of the system.

As an integral element of Intelligent Transportation Systems for Commercial Vehicle Operations (ITS/CVO) programs, electronic screening has proven to be an effective tool in improving traffic throughput on roadways. This paper does not intend to question the effectiveness of electronic screening nor the ITS concepts at weigh stations. The purpose of this study is to examine an interim solution to improve a weigh station operations in the short term until the long term benefits of electronic screening are realized.

BACKGROUND: ELECTRONIC SCREENING

Electronic screening systems use Automatic Vehicle Identification (AVI) technology to identify a participating vehicle as it approaches a weigh-station. Typically, an AVI tag (a transponder) is read by a roadside reader. The roadside reader identifies the truck and links its identification to the truck's weight and axle spacing information that is collected by a mainline WIM scale. Based on the identification of the truck, the WIM measurement, and decision rules coded into the roadside computer, a determination is made as to whether

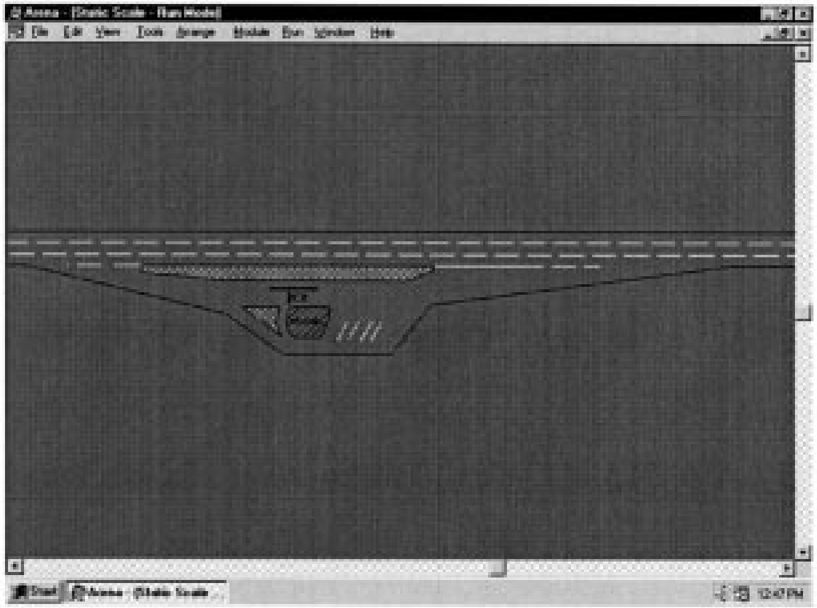

FIGURE 1 Static scale weigh station.

the truck is to be signaled into the weigh station or allowed to bypass. The in-cab transponder, in turn, signals the driver with either a green light to bypass or a red light to pull into the weigh station. Transponder-equipped trucks that are electronically cleared do not have to leave the mainline and thus benefit from fuel and time savings. By reducing the number of vehicles that have to pull into facilities that are operating at or near capacity, mainline screening also reduces frequency of full queues at weigh stations. Full queues result in either the line of trucks backing on to the mainline, a dangerous situation, or the waiving of trucks past the weigh station without performing compliance checks (unauthorized bypasses).

Electronic screening improves the efficiency of a weigh station. Because trucks participating in the electronic screening programs are not routinely stopped at weigh stations, they are able to minimize or entirely avoid the delay that results from manual checks. Enforcement officials do not routinely inspect compliant trucks participating in electronic screening. Because participating trucks are not waiting in the queue at the weigh station, the queue is diminished, resulting in fewer unauthorized bypasses.

The impact of electronic screening at weigh stations was examined by a new simulation model (*1*). The weigh station simulation is a microscopic, stochastic, model with a powerful animation capability. The simulation model is built in Arena simulation language (*2*). The model is developed for a conventional weigh station with a static scale and no ramp bypass lane as shown in Figure 1. It is built based on actual truck traffic patterns and geometry data collected at the weigh station site. The simulation results are compared to the real data, collected at the field, to validate the model.

The system performance of electronic screening at the weigh station is evaluated by conducting a "before and after" study using the weigh station simulation model. In the absence of an electronic screening system all trucks must enter the weigh station (base model). With the engagement of electronic screening systems, most of the transponder-equipped trucks are electronically cleared at the mainline (screening model). By comparing the results obtained from the simulation model run under the two described scenarios, the system performance of electronic screening at the weigh station is evaluated at different levels of transponder-equipped truck participation.

CASE STUDY: PHYSICAL EXPANSION

The case study involves a weigh station with a high volume of truck traffic (i.e., 440 trucks per hour). The collected field data at this site indicates that more than two thirds of trucks on the mainline are currently bypassing the weigh station due to a full queue at the weigh station (unauthorized bypasses). It also shows that under the weigh station's existing operation the average queue delay (i.e., delay time for being weighed) is about five minutes per truck.

The weigh station simulation model is modified to include a ramp WIM and a bypass lane (expansion model). The new weigh station design is shown in Figure 2. It is noted that a ramp bypass lane is added by widening the existing narrow lane adjacent to the static scale (see Figure 1) which is currently used for clearing trucks with wide or empty loads. This physical modification substantially reduces travel time and the number of unauthorized bypasses. However, more comprehensive physical and policy changes are required to completely eliminate the mainline unauthorized bypasses.

Similar to the electronic screening systems evaluation, the impact of physical enhancements at the weigh station is evaluated by conducting a "before and after" study using the weigh station simulation model. By comparing the base simulation model results with the ones obtained from the expansion model, the system performance of physical enhancements at the weigh station is evaluated. The study indicates that by allowing fifty percent of trucks to leave the weigh station via the bypass lane, the number of mainline un-

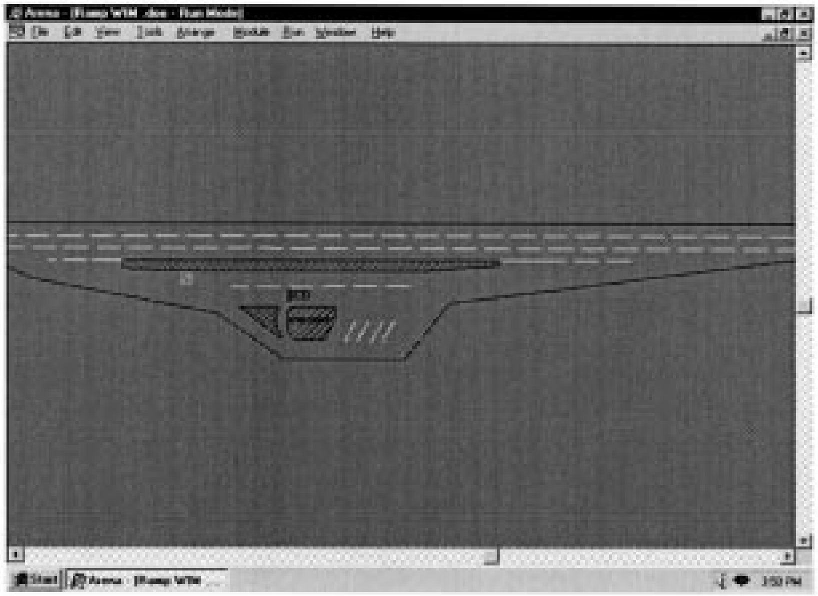

FIGURE 2 Static scale weigh station with a ramp bypass lane.

TABLE 1 Electronic Screening Versus Physical Expansion Alternatives

	Travel Time (sec/trk)			Unauthorized Bypasses (percent)		
Trans-ponder %	Screening	Expansion	Savings	Screening	Expansion	Effi-ciency
0	337	337	0	65	65	0
2	337	140	197	63	24	62
5	337	140	197	60	24	60
15	334	140	194	50	24	52
25	328	140	188	40	24	40
35	323	140	183	31	24	23
45	305	140	165	22	24	(8)
55	274	140	134	13	24	(37)
65	189	140	49	6	24	(75)
80	147	140	7	0	24	(100)

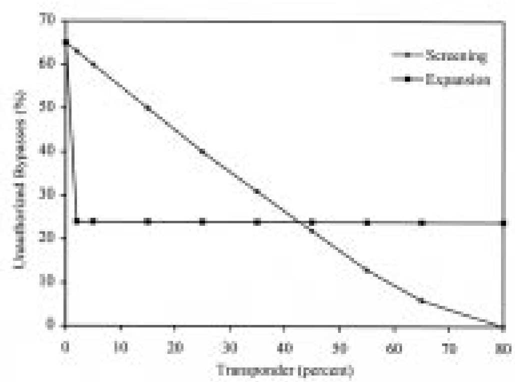

FIGURE 4 Comparison of the alternatives - unauthorized bypass.

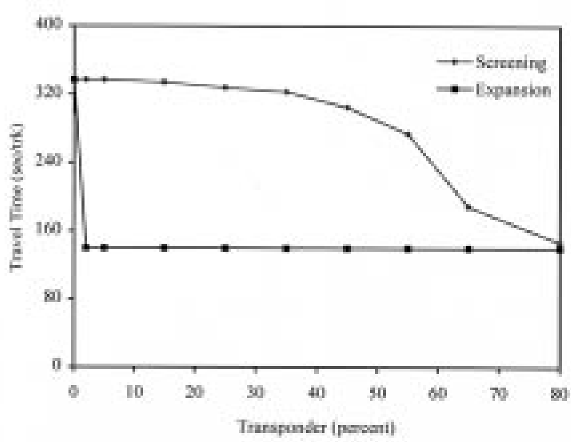

FIGURE 3 Comparison of the alternatives - travel times.

authorized bypasses and travel times are reduced by about 60 percent.

CAPACITY ENHANCEMENT ALTERNATIVES

The “before and after” study of the two proposed alternatives indicates that they both are capable of improving the efficiency of the weigh station. Table 1 presents a side by side comparison of the alternatives in terms of travel times and percent of unauthorized bypasses. The percentage of trucks with transponders is treated as a variable in the electronic screening model. The table shows electronic screening programs gradually decrease both travel time inside the weigh station and number of unauthorized bypasses. The physical expansion results, on the other hand, indicate a sudden drop in travel time and unauthorized bypasses. Table 1 also shows the travel time savings and added enforcement efficiency-deficiency of the physical expansion model as compared to the screening model results. The enforcement deficiency of the physical enhancements alternative is noted when the transponder usage reaches 45 percent in electronic screening programs.

The differences between the two alternatives are more visible in Figures 3 and 4. The area between the screening and expansion travel time, shown in Figure 3, presents the additional travel time savings that physical expansion at the weigh station could provide before the transponders market penetration reaches the 80 percent mark.

In a hypothetical situation, assume a progressive transponders market penetration rate of 80 percent in nine years. During this time period, the addition of a ramp bypass lane results in 54 years of additional travel time savings for the more than twelve millions trucks which will be traveling through the case study weigh station in nine years (i.e., an average of 2.3 minutes of travel time savings per truck).

Figure 4 compares the two alternatives in term of number of unauthorized bypasses. This figure shows that once more than forty percent of trucks participate in electronic screening programs, the resulting number of unauthorized bypasses in electronic screening case plunge under the ones in the physical expansion case. In other words, the screening weigh station design will become more efficient than the expansion model at the forty percent transponders market penetration.

CONCLUSIONS

The weigh station simulation model holds great potential as an evaluation tool for decision makers. Simulation demonstrates and quantifies the effect of electronic screening and physical expansion for a particular weigh station factoring in its unique geometrical and functional characteristics.

The weigh station simulation results indicate the effectiveness of the two proposed weigh station capacity enhancement alternatives in reducing the travel times and number of unauthorized bypasses. The comparison of these two alternatives indicates that physical expansion could improve a weigh station efficiency to some extent in a relatively shorter amount of time than electronic screening programs. Electronic screening, however, eliminates a weigh station inefficiencies gradually but permanently when the number of transponder-equipped trucks increase significantly.

The superior operability of electronic screening should also be emphasized. A ramp WIM sorts out the arriving trucks based on a set weight threshold. Those trucks that do not exceed the threshold weight are signaled by an overhead sign to return to the mainline through the bypass lane. The electronic screening system, on the other hand, monitors the weights and safety records of all approaching trucks on the mainline. The overweight trucks and those flagged for credential and/or safety problems are signaled by in-cab transponders to pull into the weigh station for a thorough inspection. Therefore, in terms of potential benefit to the state, electronic screening is the preferred method.

Electronic screening is seen as a key Intelligent Transportation Systems (ITS) function in pursuit of the Federal Highway Administration's ITS for Commercial Vehicle Operations (CVO) program vision (*3*). If one assumes that the majority of trucks eventually will be equipped with transponders, adding physical enhancements at a weigh station must carefully be examined prior to any investment. A thorough economic analysis should be carried out to determine the economic feasibility of adding a bypass lane as an interim solution and whether the resulting travel time savings and added enforcement efficiency can justify the implementation costs.

ACKNOWLEDGMENTS

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Use of Pavement Temperature Measurements for Winter Maintenance Decisions

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Formation of ice and frost on roadways and bridges presents a significant potential impediment to safe winter travel in Iowa. Roadway surface temperatures are not measured routinely by the National Weather Service and are not part of public forecasts of winter conditions, but highway maintenance personnel must make frost suppression and anti-icing decisions based on expectations of future roadway temperatures. Pavement temperatures are now measured at numerous locations in the state of Iowa and reported in real time to maintenance offices. One difficulty in use of such data is the question of how representative measurements made at one location are for other roadways in the vicinity. We have analyzed January pavement temperature data from urban/rural sites for both bridges and roadways in/near Cedar Rapids and Des Moines to evaluate nighttime trends and differences of temperatures at different locations and under different weather conditions. Preliminary results show that urban roadway pavement temperatures near both Des Moines and Cedar Rapids are 2 to 5°F higher than rural roadway pavement temperatures under clear sky conditions but only 1 to 2 or 1 to 3°F higher under cloudy conditions or when cloud cover is changing.

INTRODUCTION

Literature Review

Northern latitudes of North America and western Europe experience frequent snow, sleet, ice, and frost events from late autumn to early spring. Impacts of these conditions on highway safety have stimulated numerous studies of road surface temperatures (*1, 2, 3*). Topography is a key factor controlling the variation of road surface temperature (RST). Winter nighttime RSTs can vary more than 25.4°F (10°C) across a road network depending on factors such as exposure, altitude, traffic and changes in the road-surface characteristics. Such variable pavement temperatures can create significant variations in surface traction when moisture is present on the surface and the range of pavement temperatures span the freezing point of water. Numerous studies (*4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14*) have led to development of road-surface models to predict the occurrence of frost, wet, dry, and icy conditions on the roadways.

Present Study

This study focuses on analysis of pavement temperatures in Iowa. We have examined differences between urban and rural patterns of temperature and temperature changes under different types of weather conditions. The cooling part of the temperature-change cycle is most critical for maintenance decisions, so we focus on pavement temperature behavior from late afternoon to early morning.

Data from roadway weather information systems (RWIS), maintained and disseminated under the auspices of the Iowa Department of Transportation (IDOT), provide a valuable resource for numerous winter maintenance decisions. We analyzed nighttime pavement temperatures as reported by RWIS sensors located in and near Des Moines and Cedar Rapids under different conditions of cloud cover. Temperatures reported in this study are given in US customer units because maintenance personnel are most likely to use these units in operation.

METHODOLOGY

Data

Des Moines and Cedar Rapids each have two RWIS sites, one located generally southwest of the highly populated urban area and the other positioned in a downtown location. The downtown Cedar Rapids site has four pavement sensors located on I-380 in the vicinity of the Cedar River. Its rural (southwest) site is located on US Highway 30 near a railroad overpass, and it also has four pavement sensors. The downtown Des Moines site has three sensors located in the vicinity of the Des Moines River on I-235. Its rural (southwest) site has four pavement sensors located on I-35 over the Raccoon River and Highway 5. Sensors at all urban and rural sites are placed on roadway approaches, bridge decks over land, and bridge decks over water (Des Moines rural site has one sensor in each of two bridges over water, BW1 and BW2). Cloud cover for Des Moines and Cedar Rapids was obtained from the January 1997 Local Climatological Data (LCD) records maintained by the National Climatic Data Center.

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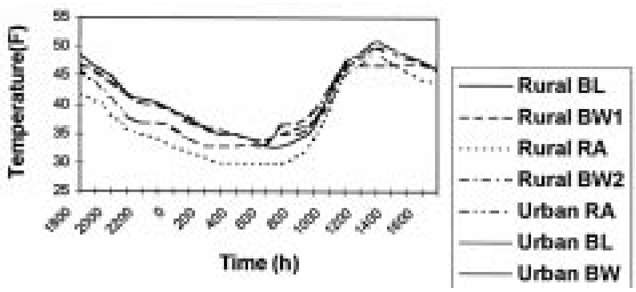

FIGURE 1 Time series of Des Moines urban and rural RWIS pavement temperatures under calm/clear skies for 01/02/97 - 01/03/97.

Procedure

RWIS pavement temperature data recorded at irregular intervals for the period 1-31 January 1997 were extracted from IDOT archives and linearly interpolated to produce an hourly temperature dataset. This dataset served as the basis for computing temperature differences, cooling rates, and lag times between urban and rural sites. Average values were stratified according to conditions of cloud cover and cloud-cover change such as clear sky/calm wind, transition from overcast to clear skies (30-50% cloud cover), transition from clear to overcast skies (75% cloud cover), and complete overcast conditions. For both cities, in general, 3-6 cases were used for each cloud-cover category.

ANALYSIS AND SUMMARY

We excluded from our analyses periods when no major changes in large-scale weather systems were the dominant influence on changes in pavement temperature.

Des Moines Pavement Temperatures

We analyzed and compared nighttime pavement temperatures for roadway approaches (RA), and bridge decks over land (BL) and water (BW) between downtown and rural Des Moines RWIS sites under different classifications of cloud cover. Figure 1 shows an example of diurnal variations in pavement temperatures for a calm/clear case.

Monotonic decrease in temperature from mid afternoon to early morning as shown in Figure 1 is a typical pattern of observed pavement temperatures, with clear skies giving the most extreme rate of temperature decrease. Under clear sky conditions, the downtown roadway approach pavement temperature exceeded the southwest site temperature by 3.9°F, the bridge deck over land downtown was warmer by 3.4°F, and the bridge deck over water downtown was warmer by 2.9°F. With complete overcast conditions, the roadway approach temperature downtown was warmer than its counterpart rural site by 1.6°F, the bridge deck over land downtown was warmer by 1.4°F and the bridge deck over water downtown was warmer by 1.2°F.

When complete overcast conditions gave way to clear skies (30-50% cover), the downtown roadway approach and bridge deck over land pavement temperatures were warmer than the southwest site by 2.0°F and 2.6°F respectively, and the downtown bridge deck

TABLE 1 Average Pavement Conditions Between the Des Moines Urban and Rural RWIS Sites for Different Classifications of Cloud Cover, January 1997

	Clear and Calm Skies			Overcast to Partly Cloudy Skies (30-50% cover)			Clear to Mostly Cloudy Skies (75% cover)			Complete Overcast Skies		
	RA	BL	BW	RA	BL	BW	RA	BL	BW	RA	BL	BW
Temp. Difference (°F)	3.9±1.7	3.4±2.5	2.9±2.3	2.0±1.1	2.6±1.4	1.4±1.5	2.6±1.9	1.2±0.8	1.4±1.4	1.6±1.6	1.4±1.1	1.2±1.2
Cooling Rates R:	1.1±1.0	1.3±0.9	1.2±1.0	0.9±1.3	1.0±1.3	1.0±1.1	0.9±1.3	1.0±1.4	0.9±2.6	0.3±1.0	0.2±1.0	0.2±0.8
($°Fh^{-1}$) U:	0.8±1.2	1.0±1.0	1.1±1.0	0.8±1.1	0.8±1.1	0.9±1.2	0.9±1.3	0.9±1.1	0.9±1.2	0.2±0.9	0.2±0.8	0.2±0.8
Mean Urban Lag Time (h)	4.9	3.4	2.6	2.5	3.3	1.6	2.9	1.3	1.6	8.0	7.0	6.0

R: Rural cooling rate
U: Urban cooling rate

over water pavement temperatures were 1.4°F warmer than its counterpart rural site. When skies became 75% cloud covered, the roadway approach downtown was 2.6°F and the urban bridge deck over land was 1.2°F warmer than the comparable rural site. The downtown bridge deck over water was warmer (1.4°F) than the rural bridge deck over water.

For Des Moines, a pavement temperature anomaly was observed to occur immediately preceding sunrise (0600-0730 LST). Under all classifications of cloudiness, the urban roadway approach and bridge decks downtown were warmer than the rural west site by 5-10°F. The cause for this pavement temperature anomaly is unknown and requires further study.

The rate at which the pavement cools is a significant factor in forecasting when wet surfaces might freeze. Under clear skies, cooling rates in Des Moines ranged from 0.8-1.1°F h-1 for approaches, 1.0-1.3°F h-1 for bridge decks over land, and 1.1-1.2°F h-1 for bridge decks over water. Transition to partly cloudy conditions produced cooling rates of 0.8-0.9°F h-1 for approaches, 0.8-1.0°F h-1 for bridge decks over land, and 0.9-1.0°F h-1 for bridge decks over water. The transition to mostly cloudy skies produced cooling rates of 0.9°F h-1, 0.9-1.0 °F h-1, and 0.9°F h-1, respectively. When skies were overcast the cooling rates were only 0.2-0.3°F h-1, 0.2°F h-1, and 0.2°F h-1, respectively. As a general rule, clear skies allowed fastest cooling, while completely cloudy skies suppressed the nighttime cooling rate. In addition, the approaches had the smallest cooling rates, while the bridge decks over water had the greatest cooling rates.

Maintenance personnel may be able to take advantage of the urban/rural pavement temperature difference in refining the timing of urban roadway treatments. For instance, if the time of ice formation due to pavement cooling at the rural site is noted, the urban pavement temperature and cooling rate can be used to predict time of freezing at urban sites. By dividing the urban/rural temperature difference by the urban cooling rate, we obtain an estimated lag time for the urban location to cool to the temperature of its rural counterpart. Table 1 shows the mean and standard deviation of temperature differences, cooling rates, and urban lag times for the roadways (RA), bridge decks over land (BL), and bridge decks over water (BW) for the different classifications of cloud cover.

In summary, analyses of the Des Moines January 1997 data show that the downtown pavement temperatures were consistently warmer than the rural-site temperatures, usually by 2.5°F and as much as 3-5°F under clear skies. Under clear skies, the urban time lag was largest for approaches, and least for bridge decks over water. With 30-50% cloud cover, the lag was largest for bridge decks over land, and least for bridge decks over water. When cloudiness increased to 75%, the lags were highest for roadways, and lowest for bridge decks over land. For overcast conditions, the lags were very large for all sensor locations.

Cedar Rapids Pavement Temperatures

We evaluated and compared nighttime pavement temperatures for roadway approaches, bridge decks over land, and bridge decks over water for Cedar Rapids downtown and southwest (rural) RWIS sites for different classifications of cloudiness for the same period covered by the Des Moines analysis.

Data available for Cedar Rapids, although fewer than Des Moines, offer an independent comparison of urban/rural temperature differences and cooling rates. Under clear skies, the roadway approach temperature downtown typically was warmer than the southwest site by 1.6°F although differences as large as 3°F were recorded. The downtown bridge deck over land was approximately 1.8°F warmer than the rural site. For overcast conditions the downtown roadway approach pavement temperature exceeded the rural site temperature by 0.4°F, and the bridge deck over land was warmer by 0.7°F. For partly cloudy skies with 30-50% cloud cover, downtown pavement temperatures for both roadway approach and bridge deck over land were modestly warmer (0.4°F for approaches and 0.8°F for decks over land). When skies became approximately 75% cloud covered, the downtown roadway approach was consistently 1.5°F warmer than the comparable location outside the city. The urban bridge deck over land also was warmer (2.5°F) than the rural deck over land.

Downtown Cedar Rapids cooling rates generally were greater than rural-site rates. Under clear skies cooling rates ranged from 0.8-0.9°F h-1 for approaches and were 1.0°F h-1 for bridge decks over land. When skies were overcast cooling rates were only 0.2°F h-1 for approaches and bridge decks. Transition to partly cloudy conditions (30-50% cloud cover) and transition to mostly cloudy conditions (75% cloud cover) produced cooling rates of 0.4-0.6°F h-1 for approaches and 0.6-0.7°F h-1 for bridge decks. As a general rule, clear skies allowed fastest cooling, while completely cloudy skies suppressed the nighttime cooling rate.

In summary, the Cedar Rapids data show that urban pavement temperatures can be expected to exceed rural pavement temperatures, with a difference of 1.6°F being typical under clear conditions. Urban temperature lags typically were 1.8-2.0 h for clear skies, 1.0 h for roadways and 1.3 h for bridge decks under partly cloudy skies, and 3.0 h for roadways and 4.2 h for bridge decks under mostly cloudy conditions. For overcast conditions, the lags were 2.0 h and 3.5 h for roadways and bridge decks, respectively.

CONCLUSION AND DISCUSSION

In conclusion, we have seen that RWIS pavement temperatures can differ significantly between urban and rural locations and that cloud cover can have a significant influence on cooling rates at all locations.

Data for Des Moines show that downtown pavement temperatures were consistently warmer than the rural-site temperatures, usually by 2.5°F for cloudy conditions and as much as 3-5°F under clear skies and calm conditions. Cedar Rapids data confirmed the urban heat island effect although the magnitude of the difference was consistently less. Des Moines area cooling rates were the greatest with clear skies during the nighttime hours and the rates for bridge decks over land (1.0-1.3°F h-1) exceeded its rates for approaches and bridge decks over water. Cedar Rapids cooling rates were of comparable magnitude. For clear sky conditions, the urban time lags ranged from 2.6-4.9 h in Des Moines where the approaches had the greatest values and bridge decks over water had the lowest values. Cedar Rapids lag times were about half as large. When skies were overcast the cooling rates were greatest (0.2-0.3°F h-1) for approaches at both cities. For complete overcast conditions, the urban lags ranged from 6.0-8.0 h in Des Moines and about half as much in Cedar Rapids.

We emphasize that these results are preliminary since they cover only January and not other winter months which may experience different patterns of temperature cooling. Also, other January months may give patterns that depart from the limited period studied herein. Despite these limitations, we conclude that pavement temperatures offer roadway maintenance personnel guidance for treating roadways for frost, snow, and ice conditions.

ACKNOWLEDGMENT

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Economic Evaluation of Advanced Winter Highway Maintenance Strategies

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Highway agencies face demands to maintain or improve the existing winter roadway level of service. The benefits of advanced winter highway maintenance strategies now become more attractive. This paper examines the potential benefits of applying advanced winter highway maintenance strategies. The Vermont Agency of Transportation conducted "Smart Salting," a study of de-icing salt effectiveness at different pavement temperatures. A recommended salt application curve was developed. As the pavement temperature goes down, it takes more salt to melt the ice. Iowa DOT snow removal and material costs were used to calculate the theoretical differential cost using the Vermont Study's recommendations and comparing to actual practice. A saving in materials exists when the Vermont Study material application rate curve is used for pavement temperatures above 25°F. Iowa DOT weather and pavement forecast verification reports were reviewed. The reports consisted of scenario-based questioning, and focused on cost savings and losses. The report calculations did reveal a loss for the 1996-1997 winter season, but preliminary figures for the 1997-1998 winter season are anticipating savings. Advanced winter maintenance strategies reduce the cost of winter maintenance. There is a cost to the roadway user when winter maintenance is lacking. Local economies will suffer, traffic accidents will escalate, and most activities of individuals, industries, utilities, schools, and governments are handicapped in social and economic ways. Key words: maintenance, economic, evaluation, costs, materials.

INTRODUCTION

Many highway agencies and state departments of transportation (DOTs) are facing staff cutbacks, are being called upon to maintain the existing or improve the level of service on their roadway system, and they face additional challenges involving increased productivity, quality, and environmental sensitivity. This paper will complete a study of the potential benefits from the application of advanced winter maintenance technologies—technologies which, when working together, have the potential to provide economic benefits to both the state DOTs and to the highway users. The potential benefits supported by this study include reduction in winter chemicals and abrasives, equipment usage, and labor costs. This paper will examine the use of pavement temperature provided by the advanced technology applications. According to the Transportation Research Board, "Demands on highway agencies for fast and effective deicing, however, sometimes results in indiscriminate salting. However, new developments in winter maintenance including deicer application techniques (e.g. salt prewetting), plowing and spreading equipment, and weather and roadway monitoring (e.g., pavement sensors) are making these priorities less confusing" (*1*).

Pavement temperature is the controlling item in the treatment of highways during winter storms (*2*). Using this fact, pavement temperature data provided by advanced technologies may be used to customize the rates of material application and the type of material utilized to match road conditions, and thus provide treatment for only those sections of the road which require it. This paper will examine two methods to use pavement temperature in advanced winter highway maintenance strategies—the first method involves observation of pavement temperature from stationary sensors and moving vehicles, and the second method involves pavement temperatures obtained from weather forecasts. This paper will use a salt application curve adapted from "Smart Salting: A Winter Maintenance Strategy" provided by the Vermont Agency of Transportation (*3*). Vermont has used the salt application curve to effectively treat their roads with a quality at or above that achieved by conventional treatment methods. This curve is included at the end of this paper. This study will compare the actual amounts of material used on highways to those amounts prescribed by the Vermont study, and determine if a potential material (and thus cost) savings estimate exists.

To estimate potential savings in labor and equipment costs, weather and pavement forecast verification reports for Council Bluffs, Des Moines, and Cedar Rapids maintenance garages were reviewed for the winter of 1996-1997. These reports contained sections outlining savings or losses in both material and labor, related to forecasts provided concerning a particular winter storm event. These savings or losses were obtained from a comparison between conventional National Weather Service (NWS) and media forecasts, and specially tailored forecasts provided by Surface Systems, Incorporated (SSI). From the savings or losses in labor costs, savings or losses in equipment were also calculated for each garage for each winter storm event. Then, using the road mileage (in lane miles) for which each garage is responsible, a value of savings or losses per lane mile of road was calculated to enable equal comparison among garages.

THE VERMONT STUDY

During the winter of 1993-1994, the Vermont Agency of Transportation (VAT) conducted a study concerning the application of pave-

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TABLE 1 Melting Capacity of Salt (*3*)

Temperature (°F)	Pounds of Ice Melted Per Pound of Salt
30	46.3
25	14.4
20	8.6
15	6.3
10	4.9
5	4.1
0	3.7

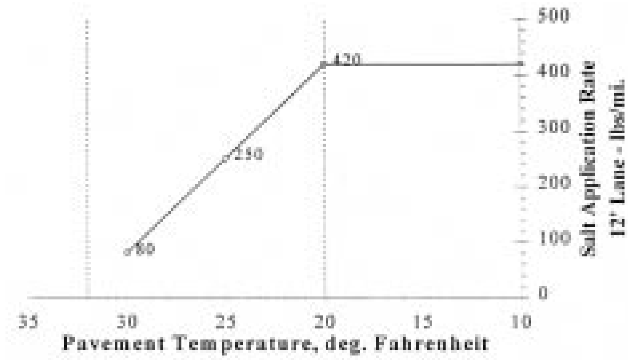

FIGURE 1 Vermont study recommended application rates.

ment temperature information to winter highway maintenance. Titled "Smart Salting," the Vermont Study was a combination anti-icing and deicing strategy. It called for winter maintenance crews to do two things. First, determine pavement temperature before and during a storm through the use of infrared thermometers mounted on supervisors' vehicles; and second, determine salt application rates based upon the relationship between pavement temperature, melting capacity of salt, and the thickness of ice or snow on the pavement.

The anti-icing concept, as identified in the Vermont Study, refers to the application of liquid chemicals and materials to road surfaces early in the storm or during plowing operations to prevent the bonding of snow/ice to the road surface. The lack of a bond between snow/ice and the road surfaces makes the task of removing the snow and ice much easier. The "Smart Salting" strategy was adopted statewide in Vermont for the winter of 1994-1995.

The Vermont Study generated a Snow and Ice Control Plan, including a figure correlating recommended salt application rates with pavement temperatures. This figure is separated into areas for 1/4" snowpack or 1/8" ice; 1/8" snowpack or 1/16" ice; and 1/16" snowpack or 1/32" ice. Prewetted salt requires less application amount per lane mile than does dry salt because the liquid element in the prewetted solution speeds up the reaction process to remove snow/ice from the road by creating a slurry which sticks to the pavement better than does dry salt. The Vermont Study also identified an "economic salting range" which extends from 30°F down to 20°F. The Iowa DOT estimates that 75 to 80 percent of winter storms occur with pavement temperatures above 20°F. For temperatures below this range down to 10°F, the salt becomes less effective in its ability to melt ice or snow: only spot treatment is recommended unless snow pack or ice begins to form ñ in which case salt should again applied. Ultimately, salt can melt snow/ice down to 6°F.

IOWA DOT SNOW REMOVAL COSTS

Iowa DOT 1996 budget figures for snow and ice removal indicate that, on average, statewide material usage costs average $35,000 per hour, and statewide labor and equipment costs average $19,000 per hour and $16,000 per hour, respectively. These three figures add to

Material + Labor + Equipment = Cost / hour

$35,000 / hour + $19,000 / hour + $16,000 / hour = $65,000 / hour

Thus the average snow removal cost for the state of Iowa is $65,000 per hour. This figure assumes that, during any given winter storm which moves across the state and involves all the maintenance areas, the entire state of Iowa is involved in some way in the treatment process.

IOWA DOT MATERIALS

Conversations with maintenance supervisors in Iowa revealed that normal salt application rates are 200 pounds per lane mile (pplm). Three hundred pplm is used for the heavy snowpack and generally occurs 20% of the time. Since the greater application rate is generally used 20% of the time, the lighter application rate is then used 80% of the time. Thus, using the proportion method for total application rate, the average application rate becomes

(0.20) (300 pplm) + (0.80) (200 pplm) = 220 lbs salt

Thus the average application amount is 220 pounds of salt per lane mile of road.

According to the salting curve developed by the VAT, salt application rates could be as low as 80 pplm at 30°F. Current practice in Iowa doesn't include this relatively low application rate. One reason for the current Iowa practice is that the proper treatment of roads is dependent upon the proper amount of material and the trend in temperature. A lapse in proper treatment may result in deterioration of the road condition and may require a considerable amount of time, material, and equipment to re-establish the previously existing level of service. Another reason for the current Iowa practice is apprehension in the recommended low application amounts by maintenance supervisors and equipment operators. The belief that "if you can't see the salt on the road then it's not enough" exists for many supervisors and equipment operators.

Therefore the material cost in salt to treat one lane mile of road, at 220 pplm, is

(220 pplm) (1 ton / 2000 lbs) ($32.50 / ton) = $3.58 per lane mile

Assuming a pavement temperature of 30°F, the Vermont Study recommends a salt application rate of 80 pplm. The material cost would then be

(80 pplm) (1 ton / 2000 lbs) ($32.50 / ton) = $1.30 per lane mile

The total material cost savings when comparing the Vermont Study recommendations to the present rates used by the Iowa DOT is

$3.58 pplm - $1.30 pplm = $2.28 per lane mile savings

According to the Iowa DOT, the Iowa DOT is responsible for maintaining 25,698 lane miles of road. Applying the material cost savings to one pass over the entire Iowa highway network at 30°F, for example, represents savings of

($2.28 pplm savings) (25,698 lane miles) = $58,591.44 savings

Greater savings in salt are realized at temperatures closer to freezing (generally above 25°F) and at lower temperatures, the savings are not as great because the Vermont Study recommends higher application rates with decreasing temperature. At lower temperatures the Vermont Study's recommended application rates are similar to the ones used in practice. For example, at 26°F the Vermont Study recommends a salt application rate of 220 pounds per lane mile, the same as the average application rate used by the Iowa DOT. Below 26°F the recommended application rates are equal to or greater than the ones already used in practice.

PAVEMENT FORECAST VERIFICATION REPORTS

To estimate potential savings in labor and equipment costs, weather and pavement forecast verification reports for Council Bluffs, Des Moines, and Cedar Rapids maintenance garages were reviewed for the winter of 1996-1997. The Matrix Management Group in Seattle, Washington administered these reports. The reports consisted of scenario-based questioning, focusing on cost savings and losses. The purpose of these reports was to measure the effectiveness of RWIS. The purpose of the weather and pavement forecast verification reports was to determine the accuracy of specifically tailored forecasts (through advanced technology) and their utility to Iowa DOT maintenance garages in enhancing snow and ice control operations. The reports also contained cost savings or losses in materials and labor for each winter storm event.

According to the reports, "to determine a loss or savings (in cost), a [maintenance] garage must determine what they would have done with only National Weather Service (NWS), television, radio, or other general media forecasts available. Actual winter maintenance decisions made using Surface Systems, Incorporated (SSI) [ScanCast] forecasts are then compared to what the maintenance managers would have done without the tailored forecast services. Additionally, the value of having RWIS equipment and atmospheric sensors that provide site-specific highway weather and pavement conditions must also be considered in overall savings/loss determinations" (*4*). Savings or losses in both material and labor were provided in the reports, and these figures were tabulated into a spreadsheet for easier calculation. Using the total lane miles of road for which each maintenance garage is responsible, a statistic of savings per lane mile was used to provide equal cost comparisons among maintenance garages.

Savings or losses in costs of equipment were derived from the labor information through some assumptions. The first assumption was a staffing of one equipment operator per maintenance truck so that a one-to-one relationship exists between labor hours saved or lost, and equipment operation hours. The second assumption dealt with the cost of equipment operation. According to 1998 Iowa DOT budget figures, the average cost to operate medium- and heavy-duty maintenance trucks equipped for snow treatment and removal is $11.36 per hour and $17.67 per hour, respectively. These figures are true averages because they were calculated based on trucks of varying ages and reliability. In addition, nearly 40% of the maintenance truck fleet are medium-duty trucks and nearly 60% are heavy-duty trucks. Thus, the average equipment operation cost becomes

(0.40) ($11.36 per hour) + (0.60) ($17.67 per hour) = $15.15 per hour

This cost figure was multiplied by total labor hours saved or lost in each winter storm to find the equipment savings or losses. See Table 2 for savings resulting from the use of advanced technology for Winter 1996-1997.

A summary of the reports for the 1996-1997 winter season revealed an overall loss when using the advanced technology (total losses of over $15,000, or a loss of $2.38 per lane mile). Most of these losses occurred in the last two dates, for the same winter storm. There is an explanation. Since the 1996-1997 season was the first time the verification reports were used, everyone was learning how to utilize them. And, in spite of the upward trend in weather forecast accuracy, situations still do occur where certain storms elude forecasters and prove to be "forecast busters." The latter explanation appears to be the best explanation for the losses, as preliminary figures for the 1997-1998 winter season indicate overall cost savings and no "forecast buster" storm events. During the 1996-1997 "forecast buster" storm system, weather forecasts called for increasingly heavier snowfall as the storm proceeded but, due to warmer than expected pavement temperatures, snow did not accumulate as forecast. (Both the NWS and SSI issued "busted" forecasts.) This situation made it very difficult to manage crews and apply salt and sand in a timely and effective manner. The totals for the winter season are calculated without and with the last storm system of the season. These totals indicate that, including the last storm system, an overall loss was realized from the use of advanced technology. Not including the last storm system, however, result in savings.

CONCLUSION

Winter maintenance is an important issue facing state DOTs. During a time when budgets are decreasing, DOTs are being called upon to provide improved levels of service. Thus, close attention must be paid to the condition of roads. According to Hanbali, "without close attention to the effective removal of snow and ice from streets, roads, and highways during periods of snow and icy conditions, local economies will suffer, traffic accidents will escalate, and most activities of individuals, industries, utilities, schools, and governments are handicapped in social and economic ways" (*5*). This paper explored the economic evaluation of the use of advanced winter highway maintenance strategies, in terms of material, labor, and equipment expenses. The results obtained from the Vermont Study indicate that, above about 25°F, potential savings in materials exists when the material application rate curve is used.

Pavement forecast verification reports were examined to determine the savings or losses in material, labor, and equipment, for maintenance garages in the Iowa DOT, when using advanced technology for winter maintenance decision-making. Although these reports revealed a loss for the 1996-1997 winter season, the losses were due mostly to a "forecast buster" storm system near the end of the season. Preliminary figures for the 1997-1998 winter season revealed a savings when using the advanced technology.

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TABLE 2 Cost Savings Resulting from Advanced Technology, Winter 1996-97

Storm Event Date(s)	Garage	Lane Miles of Road	Material Savings	Labor Hour Savings	Labor Savings	Equipment Savings	Total Savings	Savings Per Lane Mile
12/23/1996	Cedar Rapids	285.19	$825.00	0.0	$0.00	$0.00	$825.00	$2.89
12/25/1996	Des Moines N	424.06	1,620.00	50.0	587.50	757.50	3015.00	7.11
12/25-26/1996	Cedar Rapids	285.19	0.00	0.00	0.00	0.00	0.00	0.00
01/09/1997	Cedar Rapids	285.19	0.00	0.00	0.00	0.00	0.00	0.00
01/14-15/1997	Des Moines N	424.06	0.00	0.00	0.00	0.00	0.00	0.00
01/14-15/1997	Cedar Rapids	285.19	0.00	44.0	686.72	666.60	1397.32	4.90
01/23-24/1997	Council Bluffs N	210.34	0.00	0.00	0.00	0.00	0.00	0.00
01/23-24/1997	Des Moines W&N	863.86	540.00	32.0	741.39	484.80	1,798.19	2.08
01/24/1997	Cedar Rapids	285.19	825.00	0.0	0.00	0.00	825.00	2.89
01/26/1997	Council Bluffs N	210.34	0.00	0.0	0.00	0.00	0.00	0.00
02/03-04/1997	Des Moines W	439.80	0.00	-96.0	-1,999.04	-1454.40	-3,549.44	-8.07
02/03-04/1997	Des Moines N	424.06	0.00	-60.0	-1,249.40	-909.00	-2,218.40	-5.23
02/03-04/1997	Council Bluffs N	210.34	0.00	0.00	0.00	0.00	0.00	0.00
02/26-27/1997	Des Moines W&N	863.86	540.00	0.00	0.00	0.00	540.00	0.63
04/10-12/1997	Cedar Rapids	285.19	1080.00	33.0	901.77	499.95	2514.72	8.82
Sum		5781.86	5430.00	3.0	-331.06	45.45	5147.39	0.89
04/09-12/1997	Council Bluffs N	210.34	-2700.00	-20.0	-326.60	303.00	3,349.60	-15.92
04/10-12/1997	Des Moines W	439.80	-3,472.80	-372.0	-7,633.64	-5,635.80	-17,114,24	-38.91
Sum		6432.00	-742.80	-389.0	-8291.30	-5,893.30	-15,316.45	-2.38

port. Special thanks goes to Lee Smithson, Dennis Burkheimer, and Tom Donahey, all from the Maintenance Division, for their professional and sincere help in gathering information for this paper, as well as their review of the paper.

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Impact of Bridge Deck Cracking on Durability

JEFF PAPE AND FOUAD FANOUS

This paper gives the purpose and background of a current Iowa Department of Transportation (IDOT) research project entitled "Impact of Bridge Deck Cracking on Durability." Within this report, information is developed about the history of bridge deck deterioration and the use of epoxy coated rebars. Also, the procedures used to determine the impact of deck cracking on the condition of bridge structures evaluated in this project are described. Since the project has not yet been completed, conclusions and generalizations drawn from the research are only preliminary. Initial observations of the first 20 bridges evaluated have shown that the conditions of epoxy coated rebars at cracked locations can be much worse than those at uncracked locations. This can be attributed to the fact that deck cracking allows chlorides to penetrate the surface of epoxy coated rebars. Since many defects were found in the epoxy coatings of rebars in this study, this makes the reinforcing steel susceptible to corrosion and deterioration.

INTRODUCTION

Background

Concrete bridge components constructed with uncoated reinforcement and exposed to chloride salt solutions can suffer accelerated deterioration. These problems stem from the heavy use of de-icing chemicals (2.5 to 5.0 tons/lane/mile/year) on bridge decks in many States (*1*). Due to concrete's permeability and its natural tendency to crack, these de-icing chemicals can infiltrate the concrete and come into direct contact with the steel reinforcement, resulting in corrosion. Because steel expands 3 to 6 times its original volume when it corrodes, this can create areas of delaminations and spalls in the concrete (*2*). The delaminations and spalls further increase the corrosion rate of the steel by allowing even more chloride to penetrate the surface of the concrete. The problems can be so harmful to the structural capacity of the bridge that many decks required class A repairs (replacement of the upper portion of concrete and in some cases the top mat reinforcement) or class B repairs (replacement of the entire deck) after about 20 years of service.

In an effort to minimize corrosion of the reinforcement and the corresponding delaminations and spalls, the Iowa Department of Transportation (IDOT) and many other transportation departments started using epoxy coated rebars in the top mat of reinforcing in the mid-1970's and in both mats in the mid 1980's. Although the performance of epoxy coated rebars in corrosive environments is superior to typical black steel rebars, large full depth cracks have caused some concern as to the condition of the reinforcement and epoxy coating in these areas.

In a study conducted by the Federal Highway Administration in 1996, the performance of epoxy coated rebars in bridge decks was evaluated in various states and in some parts of Canada (*3*). The study found that epoxy coated rebars were performing well, except in some circumstances. For example, the study determined that defects in the epoxy coating at cracked locations and other areas with high chloride concentration can result in corrosion of the reinforcement, which could cause major problems in the future. There was also some evidence that exposure to high chloride concentrations tended to make the epoxy coatings more brittle and weakened the bond between the epoxy and steel.

A study was conducted by the Iowa Quality Subcommittee on Structures which evaluated the condition of epoxy coated rebars at cracked locations. The study found that corrosion of epoxy coated rebars was occurring at cracked locations. Although the findings were important, they only represented the condition of the bridge decks at the time of the study, and further research on this area was recommended. In addition, since the IDOT's epoxy coating specification is different than the national standard, the performance of Iowa's bridge decks with epoxy coated rebars needed to be evaluated.

Research Objective

The ultimate objective of this research is to determine the impact of deck cracking on durability. The main objectives of this research, which started in April 1997 consist of conducting a literature review, visually inspecting several bridge decks, collecting and sampling test cores at several locations on bridge decks, determining the extent to which epoxy coated rebars deteriorate at cracked locations, and determining if beams deteriorate below cracked locations. The results will demonstrate the effect of deck cracking on durability. In addition, the results obtained from this research will be used as a guide for maintenance engineers to determine an optimal time to conduct preventative maintenance or overlay bridge decks to mitigate class A and B repairs.

The objectives of the research are to be accomplished in two phases. Phase I, which will be completed by August 1998, consists primarily of detailed field and laboratory studies to determine the extent of corrosion of epoxy coated rebars in various bridge decks across the State. Phase II will complement Phase I and will com-

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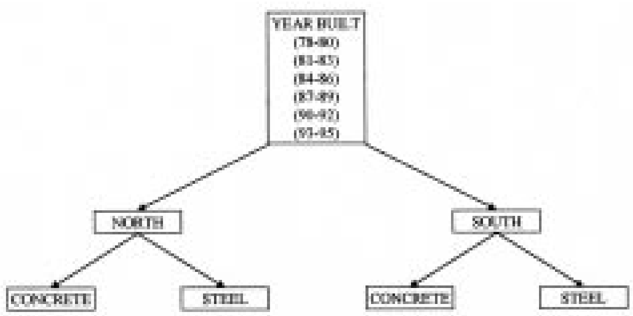

FIGURE 1 Bridge grouping.

plete the ultimate objective of this research, i.e., to evaluate and determine the impact of deck cracking on durability. Phase II is to consist of a more detailed analysis of a few bridges selected from those evaluated in Phase I.

BRIDGE SELECTION

At the time this project was initiated, the IDOT's bridge records indicated that there were 711 bridges built between 1978 and 1995 in Iowa with epoxy coated rebars in the top mat or both the top and bottom mats. In deciding which bridges to select for evaluation in this project, the characteristics of each of these bridges were obtained from the IDOT's database. The bridge records obtained included bridge type, bridge length and width, length of maximum span, number of spans, year built, ADT, ADTT, deck condition rating, superstructure condition rating, substructure condition rating, the location of ECR, and more.

The effects of many of the above listed conditions on the deck condition rating of each bridge were analyzed. The analyses showed that the deck condition rating was impacted most significantly by the age of the structure, the geographic location of the structure, the type of structure (concrete or steel), the ADT, and the ADTT. For this reason, the selection of bridges was grouped on the basis of age (1978 to 1980, 1981 to 1983, 1984 to 1986, 1987 to 1989, 1990 to 1992, or 1993 to 1995), geographic location (northern or southern Iowa), and type of structure (concrete or steel). This grouping scheme is shown in Figure 1. The average daily traffic was not included in the grouping process since this would have restricted the sample size of each group so much that many of the groups would be too small to be represented in the sampling process.

Since the long term durability of bridge decks with epoxy-coated rebars was the focus of this project, more older bridges were selected than newer bridges.

RESEARCH PROCEDURES

Field Evaluation

With the exception of a few bridges, four cores were taken from each bridge deck.

Two cores from each bridge were taken directly at crack locations, while the other two cores were taken from locations of the deck that showed no signs of cracking. The two "cracked" and the two "uncracked" cores were taken from different locations of the deck. One of the "cracked" cores and one of the "uncracked" cores were taken near the gutter line of the deck, while the other two were taken near the centerline of the deck. To simplify traffic control, all cores on each deck were taken from only one side of the bridge centerline, which was arbitrarily chosen.

Reinforcing bars in each bridge deck were first located using a pachometer. As often as possible, cores were taken at locations where longitudinal and transverse top mat rebars intersected. Cores were drilled from approximately 4 to 8 inch depths, and on several occasions bottom mat reinforcing bars were also drilled.

While the cores were being drilled, concrete powder samples at five locations across each bridge deck were collected. Two samples were obtained at each location. One sample at each location contained concrete powder drilled from a depth of 0.5 in. to 1.5 in. The other sample contained concrete powder drilled from a depth of 2.5 in. to 3.5 in.

Laboratory Evaluation

After the cores were drilled from the bridges, they were evaluated in detail. The condition and general properties of the cores and rebars were described by the procedures in the following sections.

Physical Properties

Classification of the physical properties of the cores consisted of various measurements and observations. Measurements were made on the concrete cover over reinforcing bars, the diameter of the rebars, the lengths of the rebars in the cores, the total depths of the extracted cores, the orientation of the rebars in the cores, and the orientation of cracks within the cores. In addition, factors such as the number of pieces that the core was broken into from the coring process, the number of rebars collected during coring, the type of rebar in each core, and whether or not a reinforcement tie was present were also noted.

Crack Dimensions

A microscope with variable magnification was used to determine the crack dimensions in cores taken from cracked locations of bridge decks. To obtain accurate measurements, samples were cut at ap-

TABLE 1 Rebar Rating Descriptions (*4*)

Rating	Description
0	No evidence of defects or corrosion.
1	One or more defects in the epoxy coating which don't show evidence of corrosion.
2	One or more defects in the epoxy coating which show some evidence of corrosion.
3	Corrosion area less than 20% of total ECR surface area.
4	Corrosion area between 20% to 60% of total ECR surface area.
5	Corrosion area greater than 60% of total ECR surface area.

TABLE 2 Coating Bond Rating Description (*4*)

Rating	Description
1	Well adhered coating that cannot be peeled or lifted from the substrate steel.
3	Coating that can be pried from the substrate steel in small pieces, but cannot be peeled off easily.
5	Coating that can be peeled from the substrate steel easily, without residue.

proximately 90° to the crack orientations. These samples were then polished with various grades of sandpaper. This procedure made it possible to record distinct crack width measurements that weren't altered by the chipping off of concrete near cracks during coring.

To record the crack widths, the polished surfaces were placed under the microscope and digital pictures were taken through the microscope at 0.5 in. incremental depths along the cracks. These pictures were then inputted into a computer program which could calibrate the pictures and allow the user to measure crack widths on the computer screen. A total of three crack width measurements were taken at each incremental depth in each core taken from a cracked location.

Chloride Content

Three or four concrete powder samples of at least 20 grams were collected from each core at different depths. The powder samples were drilled horizontally with respect to the deck surface using 3/8 in. diameter drill bits. For each core, one powder sample was drilled at the mid-depth of the lowest top mat reinforcing bar. The second and third powder samples were drilled at the third points between the deck surface and the rebar. In cores which contained a bottom mat reinforcing bar, a fourth powder sample was drilled at the mid-depth of this rebar. An x-ray diffraction instrument was used to determine the total chloride content in each of the concrete powder samples.

Rebar Condition

An important part of the core evaluation involved describing and classifying the condition of the rebars within each core. Although many of the rebars were separated from the cores while being drilled from the bridge decks, most of the rebars were still embedded in the concrete cores and had to be broken out in order to be inspected. Each rebar was evaluated for several characteristics, including the amount of corrosion, number of defects in the epoxy coating, and the amount of discoloration of the coating.

Each rebar was given a rating from 0 to 5. The rebar rating was categorized as shown in Table 1. (Note: The corrosion percentage for each bar was based on the surface area of the small bar sample collected and does not represent the entire length of the rebar.)

Epoxy Color

When field coring was initiated, it was noticed that many of the rebars had distinct areas where the epoxy coating was much darker than normal. In order to investigate if the discoloration had any impact on the effect of the epoxy coating, the darkest area of each rebar was compared to a color chart and given a rating which signified the color that matched the epoxy most closely.

Epoxy Coating Hardness

The epoxy coating hardness was tested in order to determine if there was any significant correlation between the epoxy hardness and other characteristics, such as chloride content, bridge age, corrosion, etc. The coating hardness of each rebar was tested using the Pencil Hardness Test described in NACE TM0174 - Section 6.1.5.

Epoxy Coating Bond

The knife adhesion test was used to determine the degree of bond between the epoxy coating and the steel on each rebar. The knife adhesion test was performed on the most discolored area of each extracted rebar. The knife adhesion test is described in NACE TM0185 - Section 5.3.2.1. The epoxy coating was rated according to Table 2.

Epoxy Thickness

The thickness of the epoxy coating was measured using two different techniques. A few samples were measured by encapsulating one end of the rebars in a plastic resin which hardened around the epoxy coated rebars. The ends of the rebars with the hardened resin were then sanded off to expose an unaltered epoxy cross section which could be read clearly under a microscope.

Other samples were measured while still intact inside the cores. The epoxy thickness measurements of these samples were taken with the microscope and digital camera while the crack width measurements were being taken. Since the cross sections of the samples were sanded while still encased within the cores, the rebar and epoxy cross sections were relatively distinct, which allowed for accurate measurements.

The epoxy thickness measurements were not taken for all of the rebars in the study due to the variability of the epoxy thickness around most of the rebars. The epoxy thickness measurements ranged from about 0.05 mm (2.0 mils) to about 0.28 mm (11.0 mils). The thicknesses of the epoxy coatings around the ribbed areas of the rebars were, in general, much less than other areas. Since the thin areas of epoxy would probably have a large impact on the effectiveness of the coatings, measurements on just a few locations of each rebar may not be representative of the coating.

Scanning Electron Microscope

Four epoxy-coated rebars and a cross section from one core were analyzed under a scanning electron microscope (SEM). A number of different characteristics were measured with the SEM, including the chloride content at different locations in the concrete, epoxy coating thickness, and the various elements that made up the concrete, steel, and epoxy-coating. Additionally, the darkened areas seen on the epoxy of some of the rebars was closely examined for deterioration. These showed microscopic pattern cracking on the surface of the epoxy, which could possibly affect the corrosion protection offered by the epoxy coating.

PRELIMINARY FINDINGS

From the evaluation of cores from the first 20 bridges in the study, some preliminary observations were made. The following are for informational purposes only and may not reflect the final conclusions of this project.

One of the most interesting findings is that all of the rebars that were evaluated as having a rebar rating of 3, 4, or 5, i.e., bars which had surface corrosion undercutting the epoxy coating, came from cores that were taken through cracks. Of all the rebars taken from uncracked areas of the bridge decks, none had rebar ratings higher than 2.

The rebar ratings of 0, 1, and 2 represented relatively good rebar conditions, although the defects in the epoxy coatings of rebars rated 1 or 2 could lead to corrosion problems in the future. Thus, it appeared that the presence of cracks in the deck surface had a large impact on the condition of the rebars below these cracks.

Although some of the bars obtained had significant corrosion on the steel surface, a large buildup of corrosion by-product was not seen. Also, no delaminations or spalls were evident on the decks where these rebars were cored.

These questions will be further addressed in Phase II of this project. Successful completion of this project will assist bridge engineers in making decisions on when to overlay bridge decks.

ACKNOWLEDGMENTS

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NUDECK - A New Prestressed Stay-In-Place Concrete Panel for Bridge Decks

SAMEH S. BADIE, MANTU C. BAISHYA AND MAHER K. TADROS

An improved stay-in-place (SIP) precast prestressed concrete panel system is presented. This system eliminates the major drawbacks of conventional stay-in-place precast panels and still maintains their structural and economical efficiencies. The proposed precast panel covers the entire width of a bridge. Therefore, it eliminates the necessity of forming for the overhangs and results in reduction of the time and labor required installing a larger number of individual panels between girder lines. The panel has a full-length gap at the girder lines to maximize the space needed to accommodate the shear connectors of the supporting girders. The precast panel is continuous over the girder with an innovative reinforcement system. Shear keys and reinforced pockets are used in the direction of traffic to provide continuity between panels. Testing shown that the proposed system has superior structural performance to the conventional SIP panel system. All materials used in the production the panel are non-proprietary and readily available. This makes this system cost competitive with that of conventional SIP precast deck system. Details of the system, including design procedure, construction steps and experimental verification, are given. Key words: concrete, prestressed, precast, stay-in-place, bridge, deck.

INTRODUCTION

The great majority of bridges built in the United States have a concrete deck slab. Most of these slabs are cast-in-place (CIP). Many bridge deck construction systems have been developed either for the construction of new bridges or for the rehabilitation of deteriorated bridge decks. Among these systems is the conventional precast stay-in-place (SIP) prestressed concrete deck panel system. This system has been employed successfully in Florida, Texas, Missouri and several other states. The precast SIP deck panel system provides a thin solid precast prestressed concrete panel of 76 to 102 mm (3 to 4 in.) to function as a form for the CIP topping and also to house the positive moment reinforcement. These panels are produced in 1219 to 2438 mm (4 to 8 ft) widths depending on the available transportation and lifting equipment. The precast panels are butted against each other without any continuity between them. They are set on variable thickness bearing strips to allow for elevation adjustment. This system has the advantage of high construction speed compared to the full-depth cast-in-place deck system because of elimination of field forming. However, it suffers from reflective cracks over the transverse joint between the precast SIP panels due to lack of continuity in the longitudinal direction. Also, the prestressing strands are not fully used due to the lack of developed length especially for small girder spacing. Design, detailing, field implementation and test results are available (*1,2,3,4*).

The objective of this paper is to present an improved precast prestressed stay-in-place concrete panel to overcome some the drawbacks of the existing precast SIP panel system. The improved precast stay-in-place deck panel covers the entire width of a bridge. The precast panel is pretensioned from end to end. Each panel acts as a continuous member over the girder in the transverse direction. Transverse and longitudinal continuity is achieved by means of an innovative technique described in the following sections.

DESCRIPTION OF THE SYSTEM

To provide a detailed description of the system, a bridge of 13411-mm (44-ft) width is considered. The deck consists of three 3658-mm (12-ft) spans plus two 1219-mm (4-ft) overhangs. The four supporting steel girders have 305-mm (12-in.) wide top flange. The system consists of a 114-mm (4.5-in.) precast prestressed SIP panel and a cast-in-place concrete topping. The CIP topping thickness can vary from 90 to 144 mm (3.5 in. to 4.5 in.) based on the girder spacing, minimum specified concrete cover and type of live load. Figure 1 shows the cross section of the deck system.

Figure 2 gives a plan view of the SIP precast panel. It covers the entire width of the bridge. The panel length can vary from 1220 to 3658 mm (4 to 12 ft) according to the transportation and lifting equipment available in the field. In this study, the panel length was chosen at 2438 mm (8 ft). At the girder positions there is a full-length gap for a width of 203-mm (8-in.) to accommodate the shear connectors. The gap-width was determined based on having steel girders with one line of shear connectors. However, the width can be increased for the case of using more than one line of shear connectors or using concrete girders. High strength concrete is used to cast the panel. Specified concrete release strength is 27.58 MPa (4.0 ksi) and the specified 28-day compressive strength is 68.95 MPa (10.0 ksi).

The panel is pretensioned from end to end with sixteen, 13-mm (1/2-in.) diameter, low-relaxation, strands of 1,862 MPa (270 ksi). The strands are provided in two layers and uniformly spaced at 12 in. (305 mm) spacing as shown in section A-A in Figure 3.

A minimum clear concrete cover of 1 inch (25 mm) is used for both the top and bottom layers of strands. In order to maintain the gap over the girder positions and to transmit the prestressing force from one part to another over the gaps, when releasing the prestressing force, 28- #19 (#6) reinforcing bars are used in two lay-

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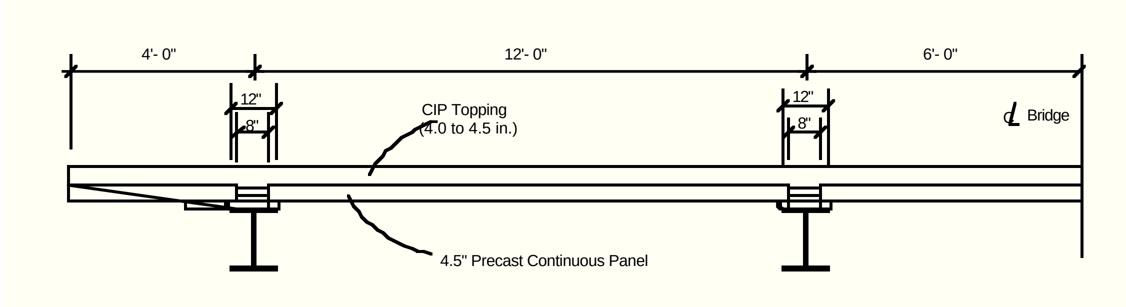

FIGURE 1 Cross-section of the bridge (1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

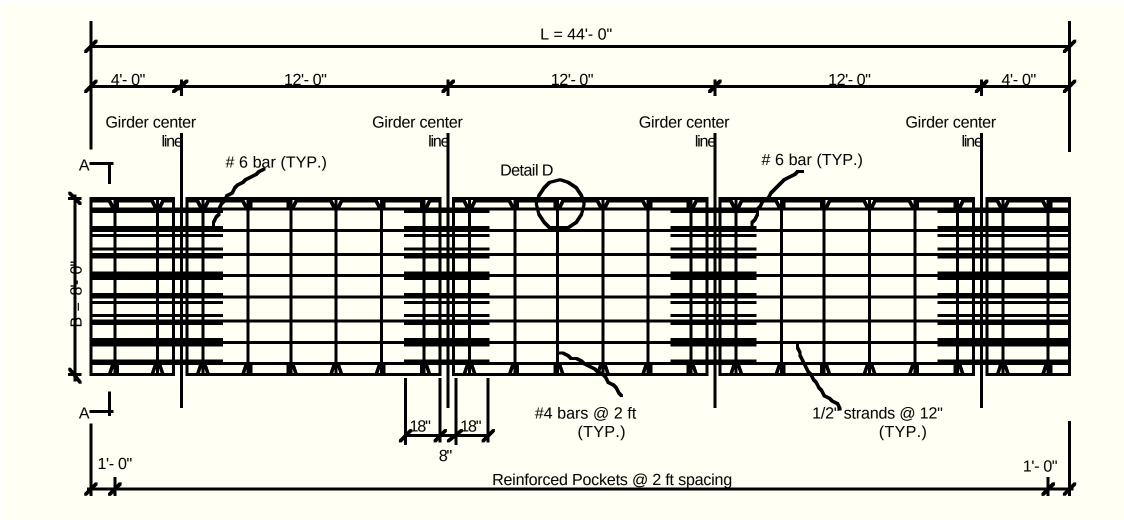

FIGURE 2 Plan view of the SIP panel (1.0 = 304.8 mm, 1.0 in. = 25.4 mm).

ers, as shown in section C-C in Figure 3. These bars have 457-mm (18-in.) embedment length to transmit the compression force from one part of the panel to the next part over the gaps.

To maintain continuity in the longitudinal direction between the adjacent precast panels, shear keys and reinforced pockets are provided. Section 1-1 in Figure 4 shows the dimensions of the proposed shear key. Reinforced pockets are spaced at 610 mm (2 ft) on center. Detail (D) and Section 2-2 in Figure 4 gives the dimensions of the reinforced pocket.

To avoid field forming, a 20-gage 152 mm x152 mm (6 in. x 6 in.) metal sheet is used as a stay-in-place form at the pockets. The panel is reinforced longitudinally with #13 (#4) bars spaced at 610 mm (2 ft). To provide for tension development for the #13 (#4) bars, they were spliced using an innovative confining technique. The splice consists of a loose 229 mm (9-in.) long #13 (#4) bar and a spiral whose size is shown in Figure 5. This technique was separately evaluated (*7*) with small tension specimens and found to produce the full bar yield strength of 414 MPa (60 ksi).

The precast SIP panels are leveled, when set over the supporting girders, using a simple leveling device as shown in Figure 6. The leveling device consists of a 1/2-inch (13-mm) thick plate. The plate has a hole of 22-mm (7/8 in.) diameter. A 19-mm (3/4-in.) nut is welded to the bottom surface of the plate. The plate is mounted between the top flange of the girder and the lower layer of reinforcement of the precast panel. A 178-mm (7-in.) high, 19-mm (3/4-in.) diameter bolt is inserted through the nuts to level the precast panel by it turning up and down. Once the panels are placed over the girders and adjusted with the leveling devices, gaps over the girders are grouted with a flowable mortar mix. The mortar mix should have a compressive strength of 27.58 MPa (4.0 ksi) at time of casting the topping slab. The mortar provides a compression block needed to resist the negative moment over the girders due to loads

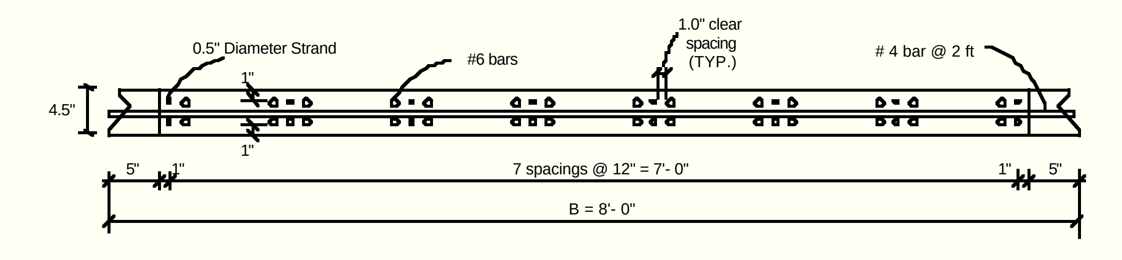

FIGURE 3 Section A-A(1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

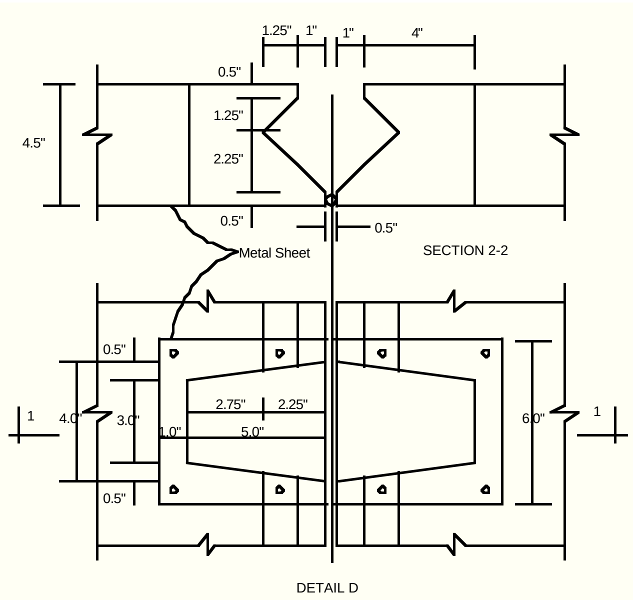

FIGURE 4 Transverse joint details (1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

imposed by the concrete paving machine and the self-weight of the concrete topping. It also provides bearing for the precast panel over the girders as shown in Figure 6.

DESIGN PROCEDURE

AASHTO Standard Specifications, 16th Edition, 1997, is used to design the system. The design procedure consists of two different sections: (1) the precast panel (non-composite section); and (2) the composite section. The precast section is designed to support its self-weight, the topping slab self-weight, a construction load of 2.394 kPa (50 lb/ft2) and the loads provided by the concrete paving machine. The composite section is designed to support a superimposed dead loads of 1.197 kPa (25 lb/ft2), barrier self weight, and the live loads. An HS25 design truckload is considered as the live load. For the design of the precast panel, two stages were considered: (1) release of prestress; and (2) casting of the topping slab. At release stage, compatibility and equilibrium equations are applied at the section at the gap to calculate the compressive stress gained in the #19 (#6) bars and the tensile stress lost in the prestressing strands. Thus,

Equation (1):

$$\varepsilon = \frac{A_p f_{pi}}{A_s E_s + A_p E_p}$$

Equation (2):
Compression stress in the reinforcing bars = $\varepsilon (E_s)$

Equation (3):
Tensile stress in the prestressing strands = $f_{pi} - \varepsilon (E_p)$

where: ε = the elastic strain loss in the gap, f_{pi} = tensile stress in the strands just before release, A_s = the cross section area of the reinforcing bars, A_p = the cross section area of the prestressing strands, E_s = the Modulus of Elasticity of the reinforcing bars, E_p = the Modulus of Elasticity in the prestressing strands.

Using Equations (1) to (3) results in ε = 1.164x10^{-3} mm/mm (in./in.), compression stress in the reinforcing bar = 233 MPa (33.76 ksi), and tensile stress in the prestressing strands = 1171 MPa (169.91 ksi). Similar analysis at mid-span between the girder lines needs is conducted to determine the tensile stress in the prestressing strands at that location. The reinforcing bars in the gap must be adequate to satisfy two design criteria: (1) preserve as much prestress in the strands as possible; and (2) transfer that prestress to the adjacent concrete without too much stress concentration. The first criterion was already covered in the preceding paragraph. Satisfaction of the second criterion is not totally clear to the authors. A conservative approach is to use the tension development length as the minimum required embedment into the concrete. The buckling length of the #19 (#6) bars at the gap is also checked to protect these bars from buckling.

At topping slab casting stage, three sections are checked. The first section is the maximum positive moment section between the girders, which is designed as prestressed concrete section. Thus, service concrete stresses and ultimate flexural capacity of the SIP panel are checked. The second and third sections are the negative moment sections at interior and exterior girder lines. These sections are designed as conventionally reinforced concrete sections.

Finally after the CIP topping cures, at service stage, the three sections mentioned previously are checked against superimposed dead and live loads taking into account the composite action.

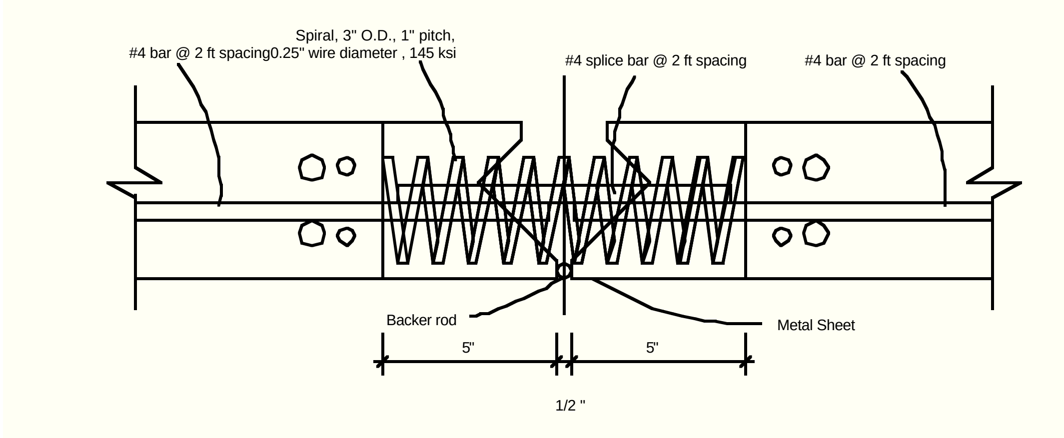

FIGURE 5 Reinforced pocket detail (1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

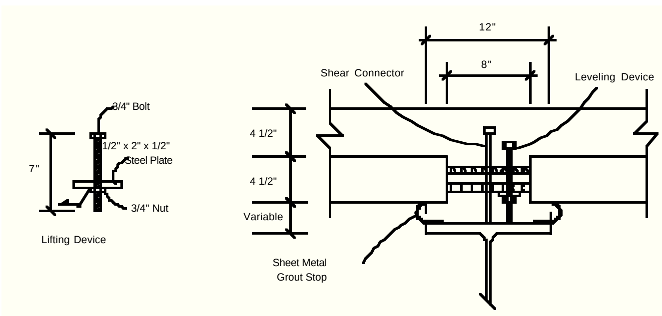

FIGURE 6 Leveling device (1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

FIGURE 7 Shear-key and pocket forming.

TESTING PROGRAM

A 6096-mm (20-ft) wide and 2438-mm (8-ft) long bridge was constructed in the structural lab. The bridge consisted of two steel girders spaced at 366 mm (12 ft) and two 122-mm (4.0-ft) overhangs. Two 6096 mm x 1219 mm (20 ft x 4 ft) precast SIP panels were produced in the lab. Wood forming was used to form the shear keys while polystyrene foam was used to form the reinforced pockets, as shown in Figure 7. Wood forming was used to form for the gap over girder lines, as shown in Figure 8. The top surface of the panel was roughened using a silk brush to a height of approximately 13 mm (0.5 in.) to provide for composite action between the precast panel and the CIP topping.

Figure 9 shows details of the panel after forming is completed. Figure 10 shows the stability of the SIP panel during handling. In order to study the behavior of the NUDECK system, two tests were conducted. These are a cyclic load test and an ultimate load test. Figure 11 shows details of the test setup. The test setup simulates two HS25 trucks spaced at 4 ft (1219 mm).

STRUCTURAL BEHAVIOR UNDER CYCLIC LOAD

The cyclic load test was performed up to 2x106 cycles. At 0.7x106 cycles, one hairline crack was noted over each girder line. The number, size, and length of cracks reported in the proposed system were much less than those reported in the conventional SIP panel system, which was tested earlier (*5*,*6*). These cracks closed after removing the load. Strain gages mounted on the top surface of the CIP topping showed that the CIP topping gained some compression stress due to the creep of the SIP precast prestressed panel. This helped to recover the cracks and to minimize their number, size, and length. No reflective cracks over the transverse joints between the SIP panels were noted.

Behavior At Ultimate Load

The test specimen was loaded incrementally until compression crushing in the CIP topping took place at mid-span between the

FIGURE 8 Gap forming.

FIGURE 9 Completed forming.

FIGURE 10 Stability during handling.

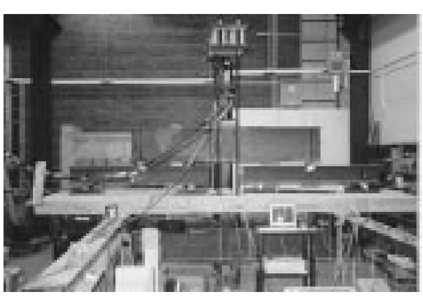

FIGURE 11 Test setup.

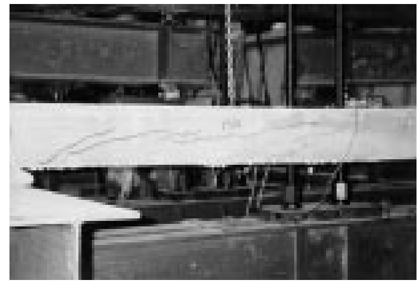

FIGURE 12 Shear cracks at ultimate.

girders. One-way shear cracks were observed in the thin non-reinforced concrete, which was filling the transverse shear key, as shown in Figure 12. However, the shear cracks did not extend beyond the transverse shear key. This was confirmed when the deck was removed for disposal. No sudden failure occurred. After removing the load, the deck returned to its original shape. No residual deflection was noted. This means that the system has a ductile behavior even after failure. Comparing the behavior of this system with the conventional SIP panel system showed that the proposed system has almost double capacity of that of the conventional SIP panel system, it has ductile behavior, and it has less deformation. Up to the failure moment, no reflective cracks were reported and no slippage took place in the splice connecting the longitudinal #13 (#4) bars. Testing program shows that making the proposed SIP panels continuously in the longitudinal and transverse direction leads to better performance, elimination of reflective cracks, and better distribution of live loads.

CONCLUSION

An improved precast stay-in-place deck panel system has been developed. The proposed system has the following advantages compared to the conventional precast SIP deck panel system:

(1) It has higher construction speed because fewer number of pieces need to be handled and field forming of the overhangs is eliminated; (2) The materials used in the production of the panel are non-proprietary and are inexpensive; (3) Elimination of the reflective cracks at the transverse joints; (4) It has superior structural performance under cyclic load; and (5) The system has almost double the capacity of the conventional SIP panel system.

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Bridge Deck Deicing

Sherif Yehia and Christopher Y. Tuan

Concrete bridge decks are prone to ice accumulation. The use of road salts and chemicals for deicing is cost effective but causes damage to concrete and corrosion of reinforcing steel in concrete bridge decks. This problem is a major concern to transportation officials and public works due to rapid degradation of existing concrete pavements and bridge decks. The use of insulation materials for ice control and electric or thermal heating for deicing have been attempted and met limited success. Conductive concrete may be defined as a cementitious composite, which contains a certain amount of electronically conductive components to attain stable and relatively high electrical conductivity. When connected to a power source, heat is generated due to the electrical resistance in the cement admixture with metallic particles and steel fibers. Based on the results of a transient heat transfer analysis, a thin conductive concrete overlay on a bridge deck has the potential to become a cost effective deicing method. Small-scale slab heating experiments have shown that an average power of about 48 W/m2 was generated by the conductive concrete to raise the slab temperature from -1.1°C (30°F) to 15.6°C (60°F) in 30 minutes. This power level is consistent with the successful deicing applications using electrical heating cited in the literature. The work described in this paper is part of an on-going research project being conducted for Nebraska Department of Roads. Two large slabs are under construction for bridge deck deicing experiment in natural environment to monitor power consumption and deicing performance. The construction costs and experimental data will be used to evaluate the cost effectiveness of using a conductive concrete overlay for bridge deck deicing or anti-icing. Key words: conductive concrete, deicing methods, steel fibers, concrete bridge deck, heat transfer.

INTRODUCTION

Traditionally, removing ice from pavement can be accomplished by a combination of several methods, such as plowing, natural melting, traffic movement, and chemical treatment. Most highway winter maintenance depends on using chemicals and fine aggregates as a primary means for deicing and anti-icing (*1*). Various deicing chemicals are available commercially. The most cost-effective product is sodium chloride. However, using chloride has caused damage to concrete and corrosion of reinforcing bars in concrete bridge decks.

The search for improved deicing methods has been a research focus for quite some time. The use of insulation materials and electric or thermal heating has been attempted; however, those techniques were either not cost-effective or could not meet the bridge deck strength requirements. Existing deicing and anti-icing methods have been surveyed and compared in this study.

Xie et al.[2,3,4] at the Canadian National Research Council have developed an innovative concept of using an "electrically conductive" concrete mix. When connected to a power source, heat is generated due to the electrical resistance in the cement admixture with metallic particles and steel fibers and can be used for deicing and anti-icing. Coke breeze (i.e., steel shaving from steel fabricators) and steel fibers are mixed in the cement to increase the electrical conductivity, while maintaining adequate mechanical strength of the concrete. The feasibility of using a conductive concrete overlay for bridge deck deicing has been investigated. Different power supply schemes, such as using solar energy with a backup battery, microwave power, and DC power, are being evaluated for cost-effectiveness. Results from small-scale experiments using conductive concrete mixes for heating concrete decks are presented herein.

LITERATURE SURVEY

Using Deicing Chemicals

The most common deicing chemicals used by highway agencies is sodium chloride (NaCl). The recent statistics indicate that about 10 million tons of sodium chloride is used in a winter in the United States (*1*). Sodium chloride, often referred to as road salt, is usually used alone or mixed with fine aggregates. A recent interview with Nebraska Department of Roads officials has revealed that the deicing operation in Omaha uses road salt mixed with sand. The application rate ranges between 200 to 300 lbs/12 ft lane-mile and about 2000 tons of salt and 5000 tons of sand are usually used in a winter season.

Using chloride deicing salt causes many problems, which include damage to concrete pavement and bridge decks (e.g. surface scaling and corrosion of reinforcement), corrosive damage to automobile bodies, and pollution due to concentrations of sodium and chloride in roadside soils and water runoff (*5,6,7,8,9,10,11*). Furthermore, salt produces osmotic pressure causing water to move toward the top layer of the slab where freezing takes place (*6,7,9*). This action is more severe than the ordinary freezing and thawing. Commonly used deicing chemicals are compared in Table 1.

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TABLE 1 Comparison of Deicing Chemicals

Deicing Chemical	Temperature Range	Application Rate	Approximate Cost
Sodium chloride[1,11,12] (NaCl)	-10°C to 1°C (14°F to 34°F)	13 to 68 g/m² (170 to 890 lb/12ft lane-mile)	\$29/m³ (\$26/ton)
Calcium chloride[1] ($CaCl_2$)	-25°C(-13°F)	Not used alone in the U.S.A.	\$294/m³ (\$267/ton)
Salt mixed with Calcium chloride[1,12] ($CaCl_2$)	-17°C to 0°C (0°F to 32°F)	21-50 l/m³ salt (5 to 12 gal/ton)	\$108/m³ (\$98/ton)
Calcium Magnesium Acetate[1,11,12] (CMA)	-5°C to 0°C (23°F to 32°F)	15 to 39 g/m² (200 to 500 lb/12ft lane-mile)	\$738/m³ (\$670/ton)
Urea[1,11]	-9°C (16°F)	26 to 136 g/m² (340 to 1780 lb/12ft lane-mile)	\$145-\$290/m³ (\$130-\$260/ton)
Magnesium chloride[11]	-15°C (5°F)	8 to 11 g/m² (100 to 150 lb/12ft lane-mile)	Not Available
Formamide[11]	-18°C (0°F)	Not Available	\$290-\$435/m³ (\$290-\$390/ton)
Tetrapotassium[1,11] pyrophosphate (TKPP)	-4°C (25°F)	49 g/m² (640 lb/12ft lane-mile)	\$435/m³ (\$390/ton)

TABLE 2 Comparison of Different Heating Systems

Heating	Approximate Cost[a]	Annual Operating Cost[a]	Power Consumption
Infrared heat lamp[11]	\$8.9/ft² (\$96/m²)	Not available	7 W/ft² (75 W/m²)
Electric heating cable[11,22]	\$5/ft² (\$54/m²)	\$0.45/ft² (\$4.8/m²)	30 to 40 W/ft² (323 to 430 W/m²)
Hot water[23,24]	\$15/ft² (\$161/m²)	\$250/storm (3 in. snow)	44 W/ft² (473 W/m²)
Heated gas[25]	\$35/ft² (\$378/m²)	\$0.20/ft² (\$2.1/m²)	Not available
Conductive concrete overlay[b]	\$4.5/ft² (\$48/m²)	\$0.5/ft² (\$5.4/m²)	48 W/ft² (516 W/m²)

[a]Cost figures were quoted directly from the literature, and conversion to present worth was not attempted.
[b]Costs and energy consumption are estimates based on the limited data obtained in this study.

Insulation Against Freezing

One method to reduce salt usage is to provide insulation against frost and ice formation (*13*,*14*). This concept was used to insulate the underside of a bridge deck and the subgrade of highway pavements and airfield runways. The main objectives were to reduce heat loss from the surface and prevent ice and frost formation, and to decrease the number of freeze-thaw cycles and salt usage. Since 1962, the polystyrene foam (Styrofoam) has been used in Michigan (*14*), Iowa (*14*), Minnesota (*14*), Missouri (*15*), Nebraska (*16*) and Alaska (*17*), to insulate beneath the roads and airfields to prevent subgrade freezing. Canada (*18*), Sweden (*19*) and Britain (*20*) also experimented using polystyrene foam for insulation under highway pavements which effectively prevented frost action in the subgrade.

Heating Systems

Heating systems (*11*,*21*,*22*,*23*,*24*,*25*) for use in pavements have typically been embedded resistive electrical heaters or pipes containing a heated fluid. The circulating fluid systems generally use fossil fuel energy sources. Different heating systems are compared in Table 2.

Electrically Conductive Concrete

Conductive concrete may be defined as a cement-based composite that contains a certain amount of electronically conductive components to attain stable and relatively high electrical conductivity. Some of the applications are: 1) electromagnetic shielding often required in the design and construction of facilities and equipment to protect electrical systems or electronic components; 2) radiation shielding in nuclear industry; 3) anti-static flooring in the electronic instrumentation industry and hospitals; and 4) cathodic protection of steel reinforcement in concrete structures.

Xie et al. (*2*,*3*,*4*) summarized several researchers' efforts in investigating some conductive concrete composition. The conductive concrete cited in the literature can be classified into two types: 1) conductive fiber-reinforced concrete, and 2) concrete containing conductive aggregates. The first type has higher mechanical strength but lower conductivity with a resistivity value of about 100 Ω.cm. The reason for the lower conductivity is due to the small fiber-to-fiber contact areas. The second type has a higher conductivity with a resistivity value of 10 to 30 Ω.cm, but relatively low compressive strength (less than 25 MPa). Lower mechanical strength is due to the high water content required during mixing to offset the water absorption by conductive aggregates, such as carbon black and coke. Xie et al. (*2*,*3*,*4*) patented a new conductive concrete mix developed at the Institute for Research in Construction, National Research Council of Canada. With the newly developed mix, both high conductivity and mechanical strength can be achieved simultaneously. However, this mix has not been utilized in actual field applications. The material costs of conductive concrete are compared against those of conventional concrete in Table 3.

TABLE 3 Material Costs of Conductive Concrete Versus Conventional Concrete

Material	Cost/lb	Conductive Concrete	Conventional Concrete
		Cost/yard^3	
Steel fiber	\$0.40	\$80.0	0
Conductive material (coke breeze, steel shaving, etc.)	\$0.10	\$70.0	0
Sand	\$0.0024	\$2.6[a]	\$2.4
1/2 in. limestone	\$0.0024	\$3.9[a]	\$4.7
Cement	\$4/(sac 94 lb)	\$35[a]	\$32
Total		\$191.5	\$39.1

[a]Due to the use of conductive materials, more sand and cement and less limestone were used than in conventional concrete.

TABLE 4 Physical and Thermal Properties of Conductive Concrete

Composition	Mass Density (kg/m^3)	Heat Capacity (kJ/kg-°K)	Thermal Conductivity (W/m-°K)
Steel	7850	0.42	47
Conventional Concrete	2300	0.88	0.87
Conductive Concrete	3133	0.71	4.4

SIMPLIFIED HEAT TRANSFER ANALYSIS

Conductive concrete may be considered as a "composite," whose constituents are steel fibers, steel shaving, and regular concrete. Based on the volume fraction of the steel fibers and shaving contained in the composite, expressions of "apparent" physical and thermal properties of conductive concrete may be derived from those of the constituent materials (*26*).

The "apparent" physical and thermal properties for a conductive concrete mix with 15 percent of steel fibers and shaving by volume can be derived from those of steel and concrete. These physical and thermal properties of conductive concrete are compared with those of steel and conventional concrete in Table 4.

Heat Transfer Analysis for Bridge Deck Deicing

With the apparent physical and thermal properties of the conductive concrete (with 15% of steel fibers and shaving by volume) determined, a simplified heat transfer analysis has been conducted to determine the power consumption in using conductive concrete overlay for bridge deck deicing.

A hypothetical case is proposed here with realistic parameters given as follows: ambient temperature T_a= -10°C (14°F), initial overlay temperature T_{ov} = -10°C (14°F), wind blowing across bridge deck at 24 km/hr (15 mph), a 3.2 mm (1/8 in.) thick layer of ice on deck surface, and a 51 mm (2 in.) thick conductive concrete overlay on top of a 152 mm (6 in.) thick regular concrete deck. The power consumption and the associated cost of deicing a concrete deck of 1 m (3.3 ft) by 1 m (3.3 ft) surface area, as illustrated in Figure 1, are determined based on energy balance. The bottom face of the conductive concrete overlay must be thermally insulated to prevent heat loss by conduction into the concrete deck. The four sides of the overlay element can be considered to be adiabatic boundaries. The effect of radiant heat transfer is ignored in the analysis. A stepwise transient heat transfer analysis was conducted with 1 kW of power input to the conductive concrete overlay. The time step, Δt, of the analysis was 10 sec.

The temperature at the bottom surface of the conductive concrete overlay, at the interface between ice and conductive concrete,

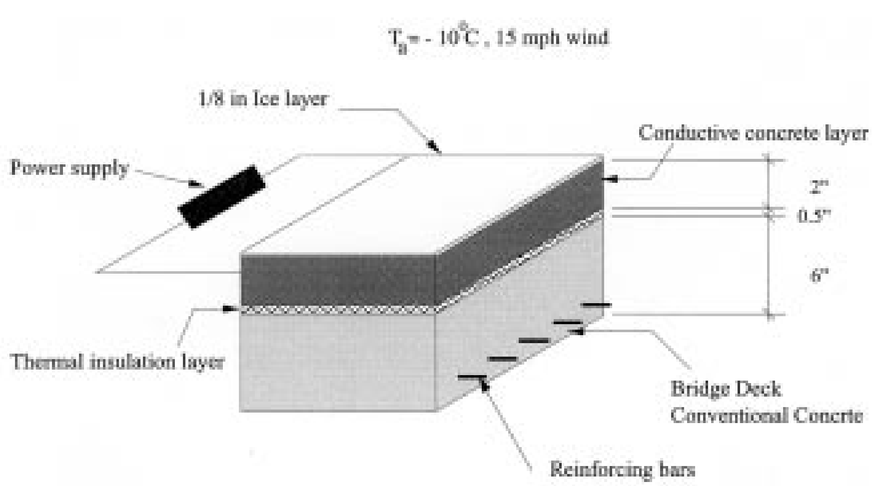

FIGURE 1 Concept of using conductive concrete overlay for bridge desk deicing.

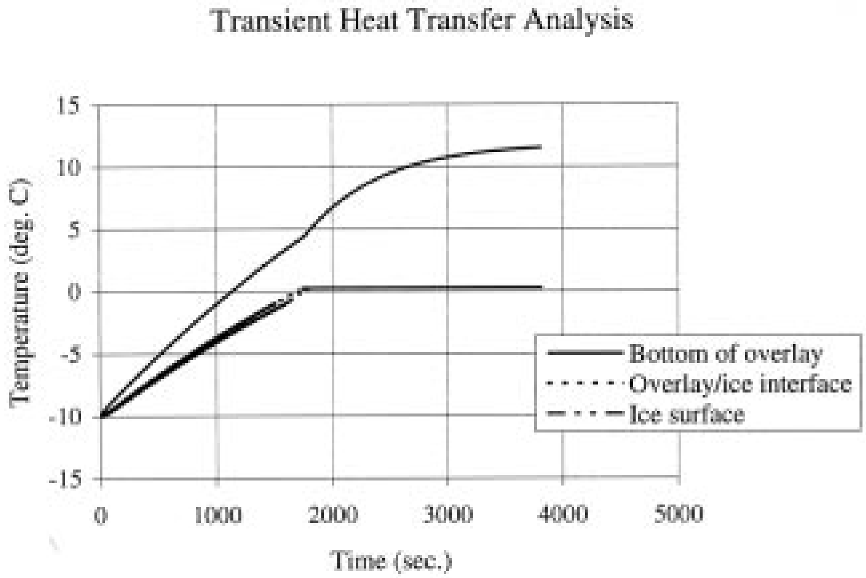

FIGURE 2 Temperature time-histories from a transient heat transfer analysis.

and at the ice surface are updated at the end of each time step based on conservation of energy and the solution process was continued until the average temperature in the ice reached 0°C. The ice would start melting at this point and continue to absorb heat for phase change into water. During the phase change, the temperature of the ice remains at 0°C. The stepwise solution algorithm was modified slightly to accommodate phase change and the solution was continued until the ice layer was completely melted. The time histories of the temperature variations for the case studied are presented in Figure 2. If the thermal energy generation in the conductive concrete overlay was 1 kW/m2, it would take about 30 minutes for the ice to start melting. It would take about an hour from the ice layer to melt completely. The highest temperature reached at the bottom of the conductive concrete overlay was 11.5°C (52.7°F). The cost of energy consumption was calculated to be about \$0.05/m^2, if the average energy cost is \$0.05/kW-hr for the United States. Based on the analysis results, it is very feasible to use a conductive concrete overlay for bridge deck deicing.

LABORATORY EXPERIMENTS

Test Specimens

Over fifty trial mixes of conductive concrete have been prepared using steel fibers with aspect ratios between 18 to 53 and steel shaving. Electric resistivity (*27*) and compressive strength were determined for each batch. The volume fractions of the steel fibers and shaving in the concrete mix have been optimized to provide the required conductivity and adequate compressive strength. The workability and surface finsihability are similar to those of conventional concrete.

Small Slabs Heating

A number of small slabs (1 ft x 1 ft x 1 in) were used to determine the required power to heat the slab. Steel plates were cast in the slabs for electrodes in the small slab heating tests. All tests were conducted in a room temperature of 74°F. Two thermocouples were installed in each slab to measure the mid-depth and surface temperature, both located at the center of the slab. The experimental results from six slabs showed that the temperature at the mid-depth in the slab increased at a rate of approximately 1°F per minute with 35 volts of DC power. The current going through the conductive concrete specimen varied from about 0.2 A to 5 A. Figure 3 shows the changes in the core and surface temperature of the 1 ft by 1 ft slabs with time. Some of the slabs were placed in a refrigerator before testing, and the results showed consistent heating of the slabs with different initial temperature. The power input was variable because there was no constant power control on the power supply. Figure 4 shows the thermal energy consumption versus average slab temperature curves for the test slabs. An average power of about 48 W/m2 was generated by the conductive concrete to raise the slab temperature from -1.1°C (30°F) to 15.6°C (60°F)in about 30 minutes. This power level is consistent with the successful deicing applications using electrical heating cited in the literature. It was noted that the specimen had a higher electrical resistance at lower temperature, while the voltage was kept constant. This phenomenon was also reported by Whittington (*28*).

SURVEY OF POWER SOURCES FOR CONDUCTIVE CONCRETE HEATING

Various power sources for heating the conductive concrete overlay have been surveyed and are being tested for feasibility studies.

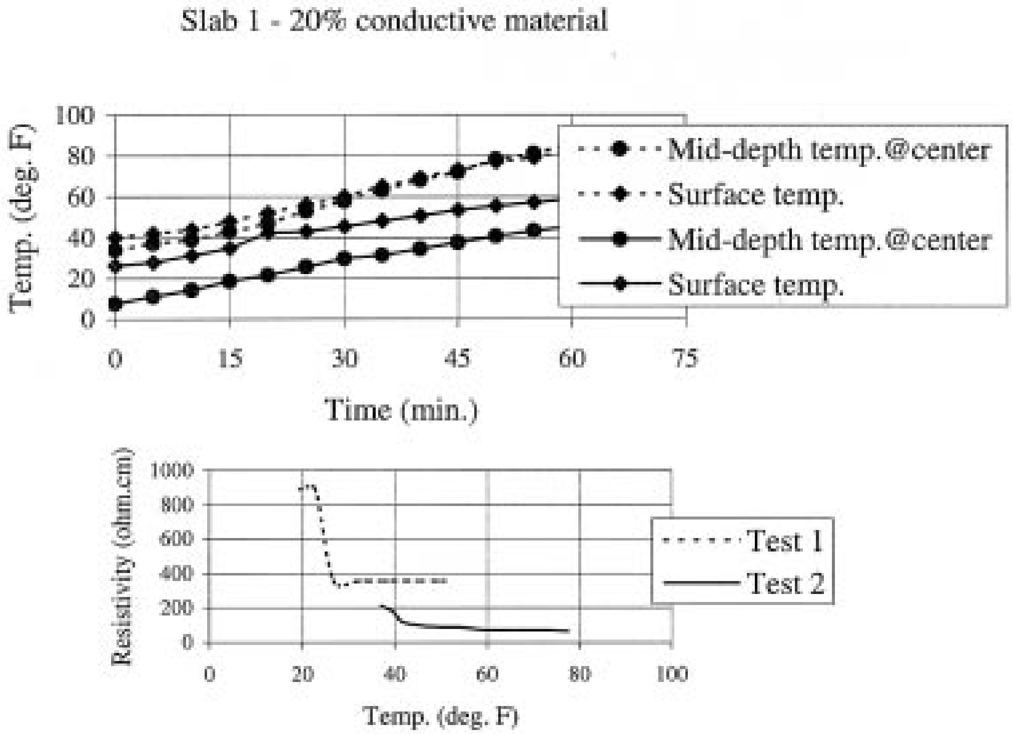

FIGURE 3a Temperature time-histories from heating tests (Slab 1).

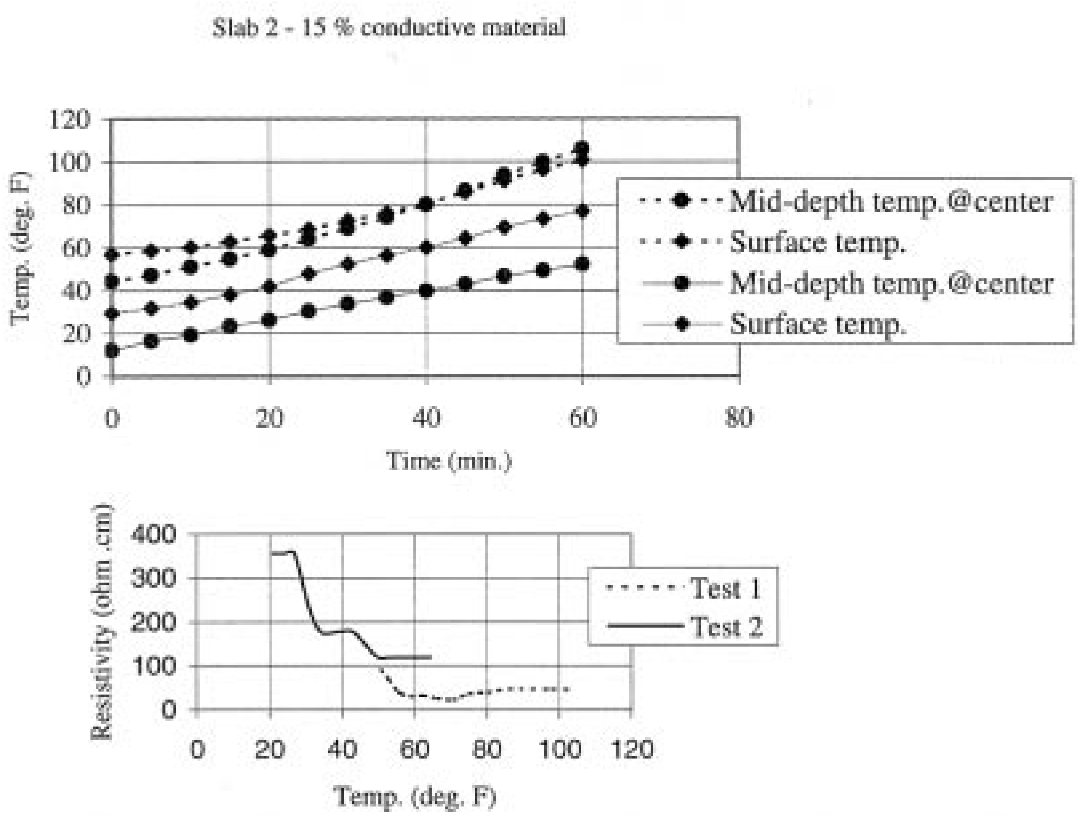

FIGURE 3b Temperature time-histories from heating tests (Slab 2).

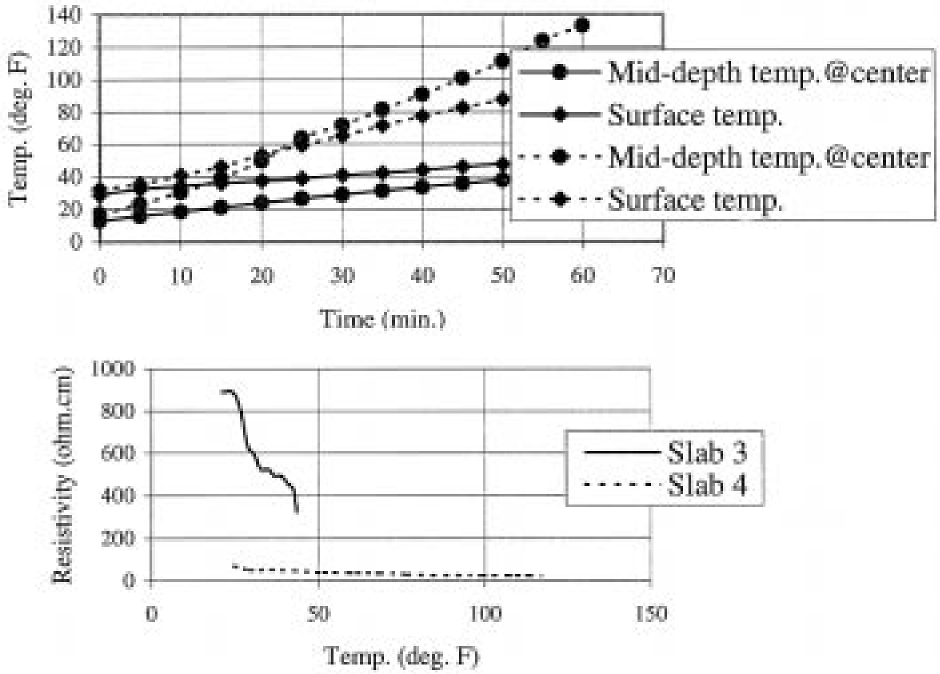

FIGURE 3c Temperature time-histories from heating tests (Slabs 3 and 4).

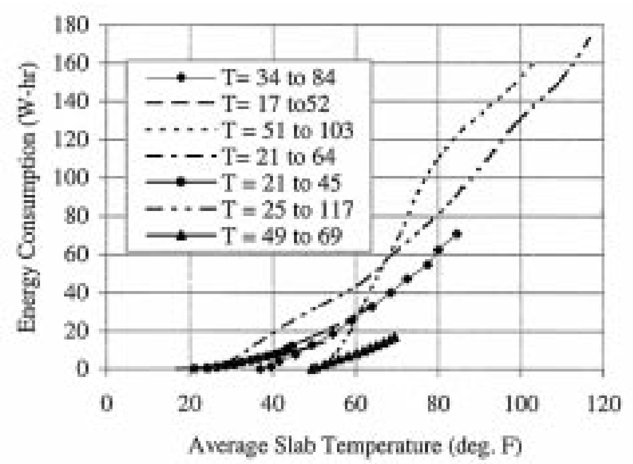

FIGURE 4 Energy consumption in small slab heating tests.

TABLE 5 Summary of Test Results

Composition	Compressive Strength[a] (psi)	Electric Resistivity[a] (Ω-m)
Conventional concrete	9425 (65 MPa)	5.4×10^5
Concrete with 2% steel fibers by volume	7918 (54 MPa)	5.4×10^5
Concrete with 15 to 20% steel fibers and shaving by volume	5000-6000 (35-40 MPa)	5 to 10

[a]Values were evaluated at the 28th day.

DC Power Supply

The simplest power source for heating the conductive concrete overlay is DC power. Through a regulated power supply, an AC power can be transformed to the required voltage and current depending on the resistance of the specimens. The voltage should not exceed 48 volts, which is the safe threshold of a human being.

Photovoltaic Power Generation

One alternative to power conductive concrete overlay is to use photovoltaic (PV) power generation. PV cells are made of silicon and first developed in the mid-1950s (*29*). PV systems are either grid-connected or stand-alone. Grid connected systems are connected to local utility lines and require inverters to convert the electricity from DC to AC. Stand-alone systems are not connected to the electric power grid, and generally use 12, 24, or 48 volt DC power. A stand-alone system with a backup battery has the potential to become a viable power source for the conductive concrete overlay.

Radio Frequency (RF) and Microwave

Another power source under investigation is the use of radio frequency (RF) and microwave heating to prevent ice formation on bridges. In direct electrical heating, a DC or AC power is applied to a conductive concrete overlay on the bridge surface to generate heat to melt the ice. RF power may be used to focus the heat to the ice formation directly.

The conductive concrete surface layer, together with the bridge sides, constitutes a lossy RF resonator with snow/ice or water forming on the surface. With sufficient concrete conductivity and proper arrangements of the conductive layers, RF excitation may generate enough heat for direct absorption by the ice formation. This scheme is similar to the heating process of a microwave oven. The feasibility of this approach depends on the RF properties of the conductive concrete mix. The RF characteristics of the conductive concrete mix is being studied at the two ISM (Industrial, Scientific and Medical) frequencies of 915 MHz and 2450 MHz in the L-band and S-band, respectively, which have been allocated by the FCC for commercial and industrial microwave applications.

CONCLUSIONS

The existing deicing methods for bridge deck and roadways have been surveyed and compared. Although using road salt is the most cost effective deicing method, it has detrimental effects on concrete structures and causes environmental concerns. The use of insulation materials for ice control and embedded electric or thermal heating for deicing have been attempted, however, they could not provide consistent deicing function or they were expensive to operate and difficult to maintain. A transient heat transfer analysis conducted has illustrated that the concept of using a conductive concrete overlay for bridge deck deicing is highly feasible and could become a cost effective deicing and anti-icing method. The experimental data from the small-scale slab heating tests have showed that the conductive concrete can achieve the desired electric conductivity and adequate mechanical strength. The heating was stable and uniform with 15 to 20 percent of conductive materials by volume. Small-scale slab heating experiments have shown that an average power of about 48 W/m2 was generated by the conductive concrete to raise the slab temperature from -1.1°C (30°F) to 15.6°C (60°F) in 30 minutes. This power level is consistent with the electrical heating applications cited in the literature for successful deicing.

The work described in this paper is part of an on-going research project being conducted for Nebraska Department of Roads. Two large slabs are under construction for bridge deck deicing experiment in natural environment to monitor power consumption and deicing performance. The construction costs and experimental data will be used to evaluate the cost effectiveness of using a conductive concrete overlay for bridge deck deicing.

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A "Trigger Mechanism" Concept for Arterial Street Improvement Programming

Barry D. Lundberg and Virendra Singh

The City of Lincoln, Nebraska, with a current population of 215,000 is growing at about 1.3 percent each year. Increasing traffic volumes are a growing concern to public officials and citizens alike. The streets serving the older sections of the city are of particular concern because of the divisions of opinion regarding the need for widening these streets. In 1995, the Mayor of Lincoln appointed a "citizen task force" to study the need for traffic improvements on five key segments of these streets and recommend a program for improvement. The task force sought a congestion measure which could be applied (as a matter of public policy) to determine when, and to what extent, improvements would be made to these streets, i.e., the term "trigger mechanism." The primary measure of congestion agreed upon by the task force was "average travel speed" along these streets. This measure would be used to determine the level of congestion and to establish threshold values which would "trigger" successive levels of study and improvement. Key words: congestion improvements, trigger mechanism, congestion measures, arterial traffic.

INTRODUCTION

In early 1995 the Mayor of Lincoln, Nebraska, appointed a citizen task force to study the improvement of traffic flow along segments of five arterial streets serving several older neighborhoods in the City's core area. The task force was officially named the Congestion Management Task Force (CMTF) and was mandated by the Mayor to: a) develop transportation system improvements that would reduce or prevent traffic congestion on the study streets, and b) recommend a "trigger mechanism" for determining the conditions under which such transportation improvements would be implemented.

In September 1995, Wilbur Smith Associates (WSA) was contracted by the City of Lincoln to assist the CMTF and the City's transportation staff in studying the extent of traffic congestion on these street and the development of a "trigger mechanism" process which could be used to determine the need for and timing of specific improvements. WSA engaged HWS Consulting Group, Inc., of Lincoln, Nebraska, to provide local traffic and municipal engineering assistance and the Lincoln-Lancaster Mediation Association to provide facilitation of key CMTF workshops.

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CONGESTION MANAGEMENT TASK FORCE SURVEY

At the outset of the study, the consulting team conducted a survey of the CMTF members to determine the member's thinking regarding key problems, issues and solutions to the traffic problems on the study streets. The results provided good insights as to the areas of agreement and disagreement between the CMTF members. The task force members indicated that primary consideration be given to signal timing, lane additions and intersection improvements. The information provided by the CMTF members helped the study team fashion a process and approach to the project which maximized effective use of the time and data available. The survey helped insure that the study effort would be consistent with the goals and objectives of the CMTF.

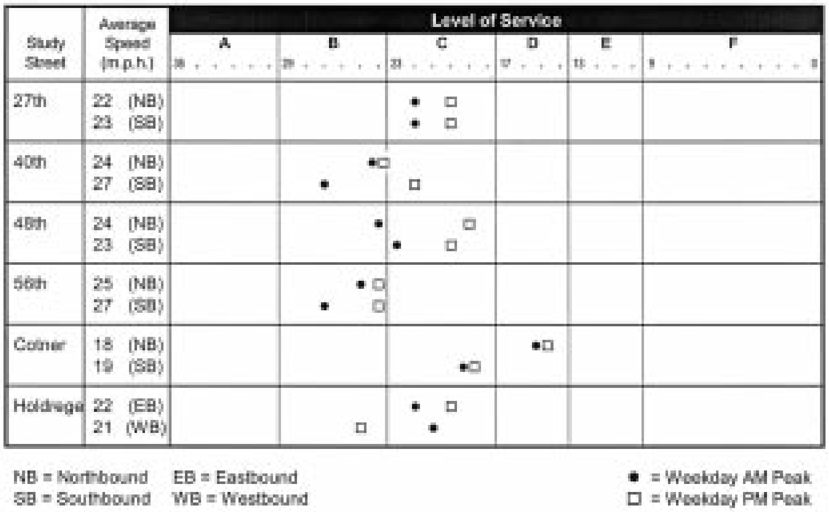
STUDY AREA CHARACTERISTICS

The character of the study corridor neighborhoods is that of mature and well established residential areas with some mixed neighborhood commercial development along with a scattering of public or community support facilities. The entire study area is homogeneous in nature with very little differential in the neighborhood ambiance throughout the corridor. At the time the study was conducted the study area was heavily forested with a strong linear planting of trees along all major thoroughfares. The age and size of the trees were such that the majority of these plantings provided a full canopy across many of the streets. One of the main concerns of the CTMF was the extent to which major traffic flow improvements (i.e., lane widening) would impact the existing trees populating the parkways and front yards along the streets.

CONGESTION MEASUREMENT AND TRIGGER MECHANISMS

It was important at the outset of the study to discuss the concept of congestion and to create consensus among the CMTF members on the best way to define and measure congestion on the study streets. This was considered key to future evaluation of alternatives and the development of the trigger mechanism(s) needed for development of an acceptable congestion mitigation strategy. During the first CMTF workshop, the consulting team and the CMTF members reviewed and evaluated various ways of defining and measuring congestion. There was strong agreement that the decision process used by travelers in making mode and route selections is primarily influenced by travel time. Based on this discussion, it

TABLE 1 Present LOS Based on Actual Average Speed



was agreed that "average speed" should be the primary measure of effectiveness (MOE) for evaluating congestion and improvement options along the study streets. *Travel time* is the common thread, both as a direct measure, and as an element of other indicators. It was also agreed that vehicle delay which is used to estimate Level of Service (LOS) at signalized intersections would be used as a secondary MOE for this study.

TRAFFIC VOLUMES

To determine how the study streets were operating, speed and delay runs were conducted during the morning peak hour, the weekday noon hour, the weekday evening peak hour and a Saturday mid-day time period. The number of runs per period varied according to the variance between each run. Typically, four to six runs were conducted and averaged. The average includes the highest 15 minute period within the peak hour as well as remaining time on either side of the peak 15 minute period. The overall results of the speed and delay runs are shown in Table 1.

Cotner Boulevard is operating in the C-D LOS range with the weekday evening, peak and Saturday mid-day peak being the worst time period. Both are for northbound traffic. (Cotner Boulevard's relatively poor performance is due to the fact that it is a short segment between O Street and Randolph and is highly influenced by the heavy traffic at the intersection of Cotner and O Streets.) Interestingly, the other study corridors operate better (e.g., a higher average level of speed than perceived by many citizens in Lincoln. Many citizens remark that these streets are very congested during peak periods and that major improvements are necessary. Based on actual field data collected these streets are working quite well and within the acceptable level established by the CMTF.

Traffic volumes on the study streets has remained fairly level with present day ADTs ranging from 10,000 to 17,000; however, the City's long range traffic forecasts predict considerable traffic growth and congestion on these streets.

THE "TRIGGER MECHANISM" CONCEPT - A NEW DECISION-MAKING PARADIGM

Making street improvement decisions using the principles inherent in the trigger mechanism concept is vastly different from the traditional approach of programming improvements based on long-range forecasts. The traditional approach often produced a "build it and they will come" result. Use of the trigger mechanism concept facilitates incremental decision-making based on measured need and trends. This can have profound implications for not only development and prioritization of projects but for resource allocation and rational accommodation of actual traffic demand throughout the City'' entire street system.

The trigger mechanism model for the study streets, and its basic operating principals and parameters are shown in Figure 1.

Incorporating the trigger mechanism concept into street improvement decisions represented a significant addition to the City's planning and transportation system development and management process. It was recognized, however, that streets of different classifications, serving different purposes (e.g., residential streets vs. collector streets vs. residential arterials vs. general community arterials, etc.) would have increasingly higher freeflow speeds. *Thus, a different threshold of "trigger" speeds would need to be established for different classifications of streets.*

The City of Lincoln adopted an average speed of 18 miles per hour as the trigger for initiating a study that could result in street improvement projects, with 16 miles per hour being the point at which the study recommendations would be implemented. Travel time and intersection delay studies would be conducted annually to monitor operational performance of these streets consistent with the principle of this trigger mechanism concepts. The parameters of the City's monitoring program will be based on:

- A statistically significant sample size for a 95% level of confidence.
- Floating car technique, where the driver floats with the traffic by attempting to safely pass as many vehicles as pass the test ve-

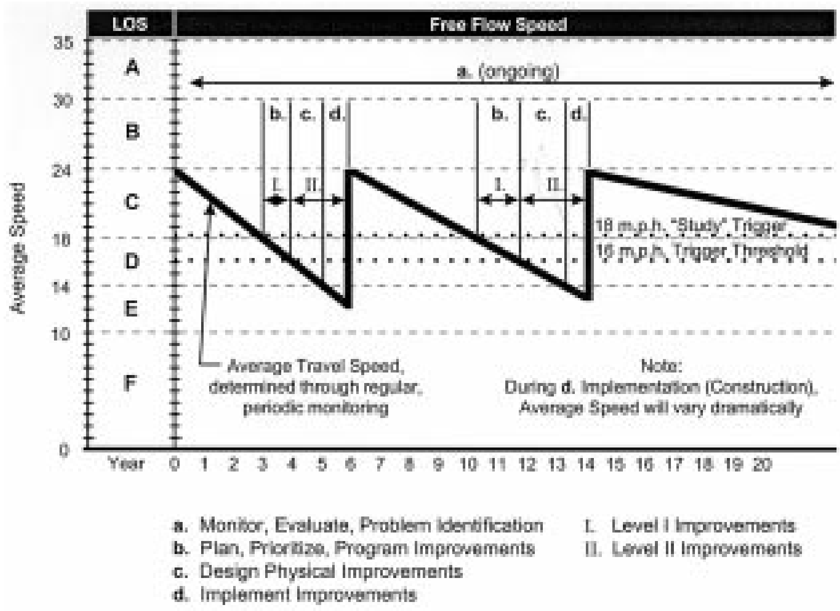


FIGURE 1 Trigger mechanism model.

hicle.

- Results should focus on "Average Speed" throughout the corridor.

IMPLEMENTATION PROGRAM

Using average speed as the primary measure of congestion, the six study corridors are operating within the acceptable level of service established by the CMTF. However, several intersections along several of these streets were operating at unacceptable LOS peak traffic conditions. Accordingly, the improvements that are needed and appropriate now and in the short run are a combination of traffic signal timing improvements, promoting ridesharing (including transit), maximum practical use of other transportation system and travel demand management techniques, and selective intersection capacity improvements. However, the longer-range traffic forecasts (2015) based on possible community-wide population growth and commercial development in or near the study corridors, suggest that at some future date (perhaps within the next 10 to 15 years) capacity improvements along one or more of these streets may be needed.

A program to effectively deal with the traffic flow conditions on the six study streets was recommended. As a result the City of Lincoln is in the process of revising the 1994 Comprehensive in the following ways:

- Review projects in the Plan and the C.I.P. and place in one of the following categories: "Corridor Improvement Study," "Project Development Area" or a "Construction Project."
- The Plan should preclude acquisition of right-of-way in a "Corridor Improvement Study" or a "Project Development Area," but authorize acquisition of right-of-way for a "Construction Project."
- All corridors with an average speed of 18 miles per hour would be proposed, at a minimum, for inclusion in Step-1 (year 2015 Street and Road Network) as a "Corridor Improvement Study" or a Project Development Area."
- All "Corridor Improvement Studies" and "Project Development Areas" would first consider all improvements that do not require additional right-of-way, before a "Construction Project" is included in the 1-20 Year Road Program or C.I.P.
- All corridors with an average speed of 16 miles per hour and for which additional right-of-way is being recommended would be proposed, at a minimum, for inclusion in the Plan as a "Construction Project."

Finally, because corridor improvement projects will be initiated by an objective measure, namely the 18 miles per hour average speeds, it is critical that the data used is accurate and verifiable. Implementation of a consistent methodology to measure future performance of the corridors is contingent upon adequate funding. Accordingly, the City of Lincoln is considering annual allocation of $200,000 of Street Construction Funds for conducting travel-time and delay studies. The travel-time and delay studies will be based upon the following issues.

CORRIDOR IMPROVEMENT STUDY CONCEPT

When one or several paralleling arterial streets have reached the threshold average travel speed of 18 miles per hour, with the operating performance steadily dropping, a comprehensive corridor improvement study (CIS) should be triggered. The consultant team recommended that the CIS should have the following important features:

1. Logical Termini: A CIS should cover a route length which extends to a logical and functional terminus on each end, even though the entire length of the corridor may not be in need of improvement at the time of the study.
2. Comprehensiveness: The study should consider all reasonable transportation elements and strategies, ranging from transportation system management (TSM) including traffic signal timing activities to physical improvements.
3. Community Involvement: The public, particularly in the corri-

dor area, should be given the opportunity to provide input and constructive suggestions at the outset of planning and during the development and evaluation of alternatives for improving the performance of the corridor. This should be a truly collaborative process, with each involvement of the directly affected neighborhood.

4. Environmental Orientation: Environmental considerations need to be an integral part of the study process and, in the case of residential arterials such as the study streets, should focus on issues of neighborhood impact such as residential displacement, front yard encroachment, and tree removal.

CONCLUSIONS AND DIRECTIONS

1. Average travel speed should be used as the primary tool for measuring and describing the level of congestion along major corridors, as well as the primary measure of effectiveness for the development and evaluation of congestion reduction actions. This measure is well understood and accepted by citizen groups and local policy makers.
2. The use of "average travel speed" for measuring congestion and planning congestion improvement programs requires a routine program of speed and delay monitoring, sharing this information with the public and policy makers, and rational application of this information with the community's transportation system improvement process.
3. It is important to undertake a community and neighborhood public involvement program to formulate consensus regarding the "local" definition of congestion (i.e., what is an unacceptable average speed for different corridors and classes of streets, as well as unacceptable levels of service for signalized intersections), and a process for determining corridor improvements (such as the use of "trigger mechanisms" and the CIS described earlier in this paper.)
4. In addition to the use of average speed as the primary determinant of congestion along a corridor, and the primary measure of effectiveness relative to improvement options, conventional level of service (LOS) analysis should be used to determine traffic flow problems and solutions for individual intersections.
5. Successful application of the trigger mechanism process described in this paper hinges on the willingness of the community's engineers, planners, and policy makers to approach the question of congestion and its mitigation from the point of view of the traveling public and the values of neighborhood residents simultaneously with a commitment to balance these interests.

Neighborhood Traffic Control Planning for Small Cities

WILLIAM TROE AND LINDA HARTMAN

Development and implementation of neighborhood traffic control plans are becoming common place for larger metropolitan areas. However, many of the same concerns which larger communities have with through traffic and higher speed traffic along local streets in residential areas are also observed in smaller communities. The most significant difference between smaller and larger communities is the sensitivity to various levels of traffic. What may be a perfectly acceptable level of traffic through a residential area in a large city is not likely acceptable in Casper. The traffic and speeds associated with the thresholds of acceptability in communities similar to Casper still require a basis in traffic engineering practice. In order to facilitate review of the state of the practice the MPO retained HDR Engineering to assist in organizing and educating the community in traffic calming practice and measures.

STUDY ELEMENTS

The purpose of the study was to:

- Prepare a Neighborhood Traffic Control Application Handbook for the MPO
- Provide outreach to the MPO, city staff and the community on What is Traffic Calming
- Establish local guidelines for when it may be appropriate to implement a program in a neighborhood
- Work with the MPO and the city to identify a pilot neighborhood for study, establish a control plan and implement the plan elements
- Provide the MPO and the Ad Hoc Traffic Control Committee with the tools required to continue the program.

INTRODUCTION

Through the long range transportation planning process, the Casper (Wyoming) Area Metropolitan Planning Organization (MPO) established a set of functional classification criteria. A total of 14 elements (*1*) were reviewed in establishing where in the functional hierarchy a specific corridor would fall. An issue associated with using such a large number of criteria to attempt to provide a definitive description of the purpose of a street, is that establishing a unique definition of each of the classifications becomes very difficult. Application of each of the criteria results in creation of a general hierarchy, however, the resulting scale does not contain distinct thresholds for which crossing results in migration to another classification.

Conflicts between the classified function of a street and the actual function arise when motorists observe a different level of utility in using the corridors than was assumed in the long range planning functions. In general for a community the size of Casper, the significance of the conflicts created as a result of providing adjacent land access directly from an arterial is lower than the significance of the conflicts associated with using local streets as a through route. To address the conflicts of through traffic using local street (residential neighborhood streets), the MPO has undertaken the task of conducting a *Neighborhood Traffic Control Study*.

Neighborhood Traffic Control Study Goals

The goals of the *Neighborhood Traffic Control Study* are:

- To establish a set of procedures and guidelines which the MPO and/or each of the local jurisdictions (including Natrona County), can follow in assisting neighborhoods in implementation of control measures
- Identify the universe of appropriate traffic control concepts from which the MPO and neighborhood groups can select for implementation
- Provide a set of guidelines for determining whether a traffic problem exists within a specific neighborhood.

Neighborhood Traffic Control Committee Representation

Representation on the committee by staff from the police department, the fire department and public services is essential to the success of the neighborhood traffic control program. Historically in communities that have attempted to implement a program of traffic calming measures, the success of the plan rests very much on staff from these city departments. Implementation (actual construction) of the calming measures typically is the responsibility of the Public Services Department. The Fire Department and Police Department have historically looked at calming measures from a different viewpoint than residents/stakeholders and traffic engineers. The emergency response personnel viewpoint focuses more on with impacts to response times associated with the calming measures. Reducing traffic volumes in residential areas has the potential to improve response times, but many of the most effective calming

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techniques tend to result in slightly higher response times to area residences and businesses. Participation by these groups ensures:

- That any recommended calming measures requiring construction, have received internal department approvals prior to the plan being sent to the Councils and the MPO Policy Committee for approval. Each of the departments from which approvals are required are represented on the ad hoc committee.
- The "hard" questions of "Are the measures needed, will the measures likely result in the desired outcome, and will implementation of the measures adversely impact emergency vehicle response times?" are asked.

NEIGHBORHOOD TRAFFIC CONTROL PLAN IMPLEMENTATION PROCESS

The intent of establishing a neighborhood traffic control plan and implementation of design and operating concepts focusing on the neighborhood is based in the following goals of the program:

- Improving safety/comfort for pedestrians, bicyclists and motorists through controlling vehicle operating speeds and minimizing through traffic in residential neighborhoods
- Avoid neighborhood intrusion through providing for and maintaining acceptable levels of service on *arterials* and *collectors*. This is completed through the long range planning process.

Through working with local planning and public works staff, and through researching neighborhood traffic control policies and plans implemented and/or studied for other communities, a set of guidelines for implementation of a plan in Casper were developed. The intent of the program is to address traffic issues on local streets in residential neighborhoods. The program is not intended to address congestion or functional classification issues associated with collector or arterial roadways. Those issues are more readily addressed through the long range transportation planning process. As a means of assisting in defining the types of roadways included within this program the following checks have been developed:

- Is the street functionally classified as a collector or arterial street?
- Is the street a part of the official truck route map?
- Does the street have direct access to the interstate?
- Does the street provide more than two through lanes, or include turn lanes?

If the answer to each of these questions is NO, it is likely reasonable and feasible to include the corridor in a neighborhood traffic control study. The general process for assessing the need, determining the appropriate set of concepts to achieve a goal and providing for local input is displayed in Figure 1. The process can essentially be divided into three phases:

- Project Initiation/Problem Definition
- Alternatives Analysis/Recommendations
- Implementation

Project Initiation/Problem Definition

The primary purposes of the initial stage of the neighborhood traffic control plan is to provide education to the local residents about

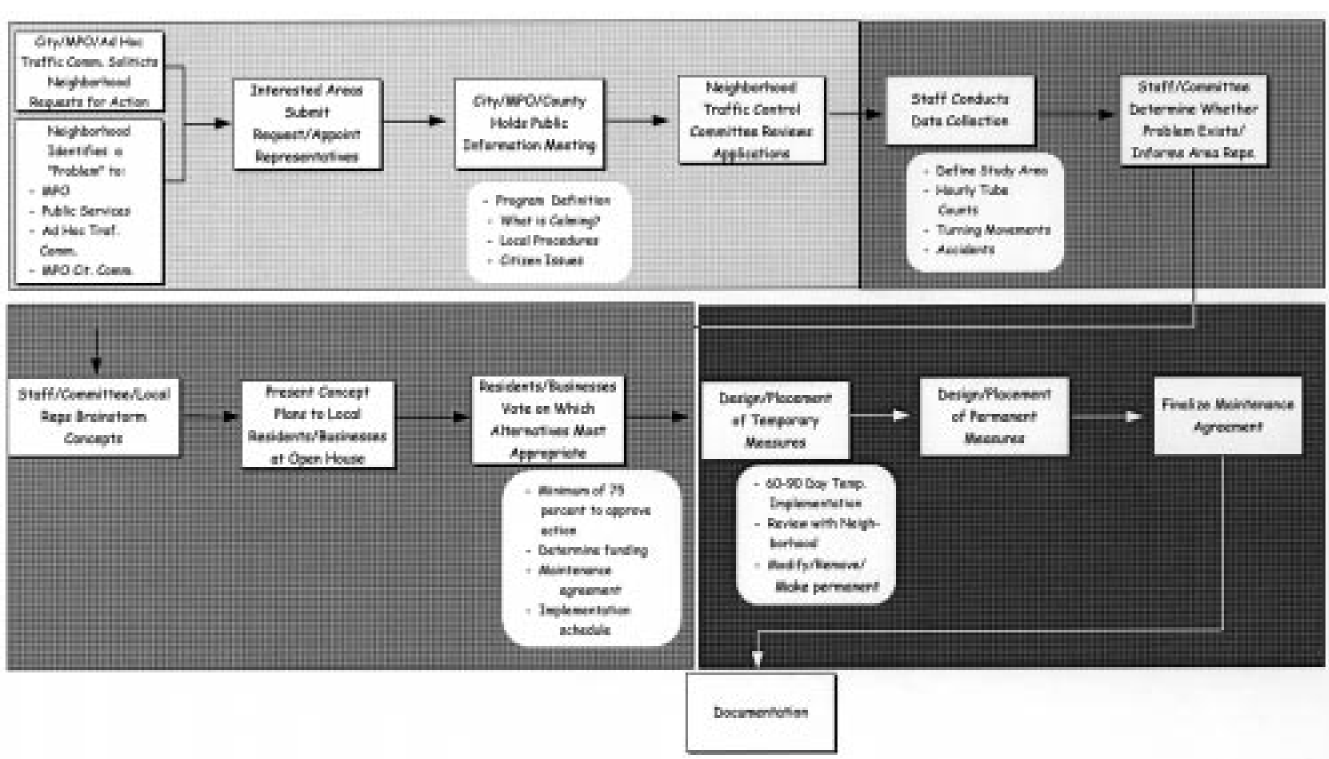

FIGURE 1 Neighborhood traffic control planning process.

the plan and to gather information about the particular issues and traffic operations in specific areas. The key elements of the initial stage are documented below.

Request for Action

It is likely that the number of neighborhood organizations or associations desiring action by the cities or county to provide relief for unwanted traffic in residential areas will outpace the ability to provide, or assist in providing services. Which residential areas desire agency assistance is not necessarily a decision which can be made by city or county staff. Thus, a process for identifying the areas which desire assistance was developed. The alternatives reviewed in the selection process included:

- On a *semi-annual* basis, a notice of intent to address neighborhood traffic issues will be posted by the Ad Hoc Traffic committee. Through the request for action, local associations and organization will be invited to submit a request for study of their particular neighborhood or subarea of interest.
- A representative or group of residents/stakeholders within the neighborhood requests action through the Public Services department. Currently, complaints/comments on neighborhood traffic issues (speeding/traffic/accidents) are brought up to police personnel, Public Services personnel, council members, etc. In general, comments which are brought up to police officers are forwarded to the Public Services department staff.
- A representative or group of residents/stakeholders within the neighborhood requests action through the City Council.

Data Collection

For each of the neighborhoods where an application or request for study was received, basic traffic and travel data would be collected in order to allow staff to determine whether a significant problem does or does not exist. The basic traffic data to be collected includes:

- Hourly traffic counts using mechanical tube counters set by city staff
- Accident data for the latest three-year period
- Number of residents adjacent to each corridor in the study area
- Summary of the 85th percentile speeds through the corridors in the study area.

Neighborhood Meetings/Workshops

Through the implementation process the Ad Hoc Neighborhood Traffic Control Committee would advertise and host a series public information meeting or workshops. The purpose of the meetings would be to:

- Provide local residents and property owners with a definition of the program
- Define what is neighborhood traffic calming
- Provide an outline of the local implementation procedures
- Gather information on local citizen issues.

Problem Definition

The purpose of this step in the problem identification process is provide a first level of screening in which the question of whether a problem which can be alleviated through implementation of traffic calming measures exists or not. In general, perceived neighborhood traffic problems can be categorized into one or a combination of the following:

- Traffic volume problem
- Vehicle speed problem
- Accident problem

Through review of the alternate methods of determining whether a traffic problem exists is has been recommended that the use of the traffic volume threshold and the speed threshold be used as the primary descriptors. The traffic noise, pedestrian activity and accident indices would be used more as support material to the conclusions drawn through use of the volume and speed indices. Thus, the perceived neighborhood traffic problem would be supported by technical analysis if:

- Traffic volumes observed in the corridor or study area are greater than the calculated "home-based" traffic associated with the residences adjacent to the corridor or within the residential study area
- Observed 85th percentile operating speeds in the corridor exceed 30 MPH for those areas with a posted or prima facie speed limit of 30 MPH or less.

Alternatives Analysis

Through the alternatives analysis the process, the Ad Hoc Traffic Committee, MPO/City staff, emergency response staff and representatives from the study area would meet to discuss the feasibility of the various alternative calming techniques in solving the identified problems. The range of calming techniques that are included in the Casper neighborhood traffic control toolbox, have been developed through review of the successes and failures of devices in other communities (*2,3,4,5 6,7,8,9,10,11,12*).

Traffic Control Measures Workshops

The process for developing a set of traffic control measures is documented below:

- An initial workshop is held where the techniques included in the toolbox are discussed with representatives from the study area.
- It is unlikely that a neighborhood control plan can be developed for an area through a single workshop. In most case a second workshop would be held to:
  -Review the designs prepared by city or MPO staff
  -Refine the "package" of neighborhood controls to be implemented
  -Establish a schedule for implementation of the neighborhood traffic control measures
  -Establish the roles and responsibilities of staff, Ad Hoc Committee and neighborhood representatives at the neighborhood open house to be held to present the preliminary findings.

Alternatives Open House

Following the alternatives development workshops, an open house will be held for the neighborhood residents. The purpose of this open house to provide local residents with an opportunity to comment on the design concepts developed in the workshops by city, MPO staff, ad hoc traffic committee members and local area representatives.

Implementation

Implementation of ANY neighborhood traffic control measures must have overwhelming support from city/MPO staff, emergency response personnel and residents of the implementation area. Thus, a survey would be distributed at the open house and as part of the open house notice through a door-to-door campaign in the immediate neighborhood. Through questions asked in the survey, the level of support/opposition to the proposed concepts will be gathered. Prior to implementation of the concepts on a temporary basis, a minimum of 75 percent of the adjacent residents/land owners must be in support of the concepts, the means of funding the concepts and the terms of any maintenance agreements.

The public services department has the responsibility of final design of the control concepts. After completion of the final design, public services staff will be responsible for implementation of the measures on a temporary basis.

The proposed control measures will be implemented in the field for a period of 60 to 90 days. After which:

- A survey inquiring about the usefulness and level of support for the measures. As with the initial survey, a minimum of 75 percent of the neighborhood residents must support permanent installation. Through the survey information on small design modifications would also be obtained.
- Traffic count data would be collected by city staff to determine the level of impact associated with implementation of the control measures.
- Vehicle speed data would be collected by city staff to determine the level of impact associated with implementation of the control measures.

Should the minimum level of support from the neighborhood be obtained (75 percent of residents/land owners), permanent installation of the control measures would be scheduled with city public services. At this time any maintenance agreements included with the plan implementation would be finalized and filed with the project documentation.

FINANCING PROPOSED CONTROL MEASURES

Alternatives discussed for funding design, implementation and maintenance of the alternate measure include:

- Funded through local construction/maintenance budgets of the Public Services Department
- Costs for design and construction are funded through creating local assessment districts in the affected area
- Costs for design, construction and maintenance are funded through a combination of city construction/maintenance funds and local assessments
- In low and moderate income areas, design, construction and maintenance costs could be funded through use of Community Development Block Grant (CBDG) funds.

DOCUMENTATION

The following documentation would be included as part of the neighborhood traffic control plan:

- The request for action published by the MPO and ad hoc committee
- Responses by the neighborhoods to the request for interest
- Traffic, speed and accident data collected as part of the study
- Notes from the alternatives workshops
- Responses to the surveys distributed for to obtain input on the concepts from the neighborhood residents and land owners
- Documentation of the concepts requested to be implemented
- Results of the survey conducted after the temporary implementation period
- Any maintenance agreements developed through the planning process.

KEY FINDINGS OF THE STUDY

Implementation of the neighborhood traffic control planning process is in the early stages. A test neighborhood was selected by the MPO Policy Committee and MPO staff. The neighborhood was selected because of its location in the city and a history of resident complaints about high levels of though traffic and speeding. Through the early stages of implementation a number of findings have been established, including:

- Establish a review committee with representation from public works, emergency services (police/fire) and planning.
- Selection of local representation: Selection of the appropriate local representative is essential during the study phase. The representative must be willing to listen to other neighborhood residents and city staff.
- Facilitate early involvement by a broad cross section of the study area: The most challenging aspect of the plan implementation was obtaining a reasonable level of consensus in the neighborhood. It has been an observation that a seemingly relatively homogeneous neighborhood can contain a vast range of ideas and expectations.
- Establish a funding mechanism/policy prior to public involvement.
- Narrow, through a local committee, the calming techniques deemed appropriate for implementation in the community: Providing too many alternate concepts can reduce the ability to obtain reasonable consensus.

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Quick-Response Bypass Forecasting for Small Urban Communities Using an Economic Gravity Model for External Trip Analysis

MICHAEL D. ANDERSON AND REGINALD R. SOULEYRETTE

The Iowa Department of Transportation routinely assesses the potential traffic impact of highway bypasses on small cities in the State. For a given city, this labor-intensive process may take from one-to-three person months to complete. With the implementation of Iowa's Commercial and Industrial Network, bypass analyses are becoming more frequent, and are now required for six-to-eight cities per year, placing a strain on limited planning staff resources. This paper describes a GIS-based modeling approach which can streamline and automate the process, thereby reducing resource requirements or increasing the number of studies that can be completed within a given time. The automated process also serves to promote consistency across projects. This paper also validates the new approach by comparing manual-based estimates of impact to those predicted by the new methodology. Key words: bypass planning, traffic forecasting, GIS.

INTRODUCTION

The National Highway Cooperative Research Program (NCHRP) defines a highway bypass as an existing roadway that previously passed through town which is realigned through rural land outside the city limits, see Figure 1 (*1*). The Iowa DOT five year transportation improvement program includes 34 bypass locations and ten more are under consideration (*2*). The bypass planning technique currently employed by planning staff includes a somewhat subjective and labor intensive spreadsheet-based assessment of historical traffic trends sometimes based on 30 year old origin-destination studies. The studies result in hand-generated traffic flow maps and associated tables and reports. As each bypass plan may require one to three (or more) person-months to complete, staff resources are becoming strained.

To assist the Iowa DOT in meeting the needs of an increasing number of bypass studies, the Center for Transportation Research at Iowa State University has developed a geographic information system (GIS) based methodology for bypass analysis. The methodology uses datasets maintained by various government organizations and other commercially available sources. The GIS-based methodology provides a series of interactive steps that develop a "quick-response" travel demand model which forecasts traffic volumes and turning movement counts. The methodology reduces the subjectivity associated with the existing process by employing a conventional urban-modeling approach to determine travel patterns in a matter of days. This paper describes and tests the methodology in a case study of one Iowa community where a highway bypass plan was recently completed. The results of the case study demonstrate the GIS-based methodology can provide similar traffic assignments in a fraction of the time.

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MODEL DEVELOPMENT

There are six steps involved in the GIS-based quick response model development methodology. They follow the similar structure used to develop travel models using traditional strategies. The steps include: data collection, defining model structure, digitizing the attributed network, calculating productions and attractions, validation of the base model and developing alternative bypass scenarios, and forecasting future traffic volumes. These steps are demonstrated in the following section.

Step 1: Data Collection

The initial step required for model development involves collecting appropriate datasets. The ability to store, query, manipulate and perform calculations on the datasets in a GIS environment affords several time-saving and quality control advantages. For the

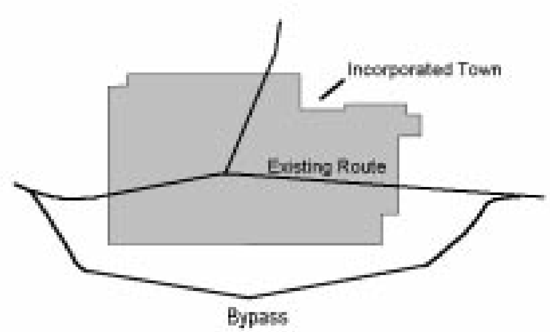

FIGURE 1 Typical bypass configuration.

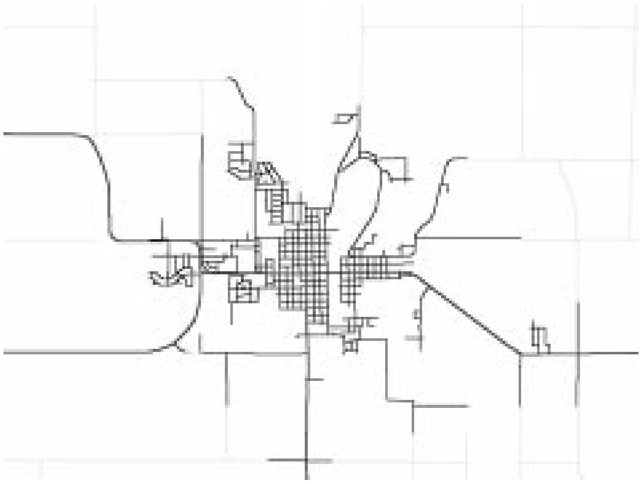

FIGURE 2 Roadway data for an Iowa community.

current study, all data sets were incorporated into the MapInfo GIS program (MapInfo Corp., Troy, NY).

Street Data

Roadway network data are available from a number of sources:
- TIGER street files
- local government agencies
- State DOT
- other commercially available sources.

TIGER street files include street names and address ranges for roadway segments. This information can be used to geocode address specific data. Iowa DOT street and highway maps contain geometric and operational data for all roadways for the state (speed limits, roadway volumes, and available lanes). An example Iowa DOT map is shown in Figure 2.

Socio-Economic Data

Socio-economic data to support travel modeling are available from several sources:
- employment data from state agencies monitoring workforce statistics
- census data
- white pages databases available at most software outlets.

Employment data from government agencies are typically considered private and are protected from dissemination, at least in their most disaggregate form, by law. Many public planning agencies can obtain permission to use these data, however, provided they do not disclose the identity and characteristics of individual employers. Data containing business addresses, number of employees and business function (through the use of a industrial codes) can easily be geocoded and used to calculate zonal attractions. Census data, available at block and block-group levels, include population, household occupancy and income levels. "White-pages" data generally contain addresses for all households and businesses in the study area which can be geocoded in the same manner as the employment data and aggregated to a traffic analysis zone level. An example of a popular package geocoded to the TIGER street network is shown in Figure 3 (circles indicate businesses and stars represent households).

Step 2: Define Model Structure

After collecting roadway and socio-economic data, the next step involves defining the travel demand model network through the establishment of traffic analysis zones and travel model streets. This process requires personal judgment and experience to define the appropriate model structure and selecting the location, number, size and shape of the traffic analysis zones will not be addressed in this paper. Census block-groups can provide a suitable structure for zones in some of the larger communities, however, for most smaller communities, block-groups will be too aggregated to be effective zones. In these areas, using high volume roadways as zonal edges represents another alternative. The user should develop a new GIS table for the zones with columns for zone number and all the variables required to perform the trip generation calculations.

After developing the GIS table structure, the user needs to digitize polygons to define the traffic analysis zones. Note that it is important to locate the zones and number the zones in the same fashion as the centroids (which will be placed later). An example table showing traffic analysis zones is provided as Figure 4.

While the traffic analysis zones are being defined, the user needs to select the major streets in the network included in the travel model. This information can be determined from traffic count maps or queried within the GIS if the data are available. As mentioned, the Iowa DOT roadway database is one potential dataset containing traffic volumes, which can be accessed through various query statements. It is recommended that the network roadways are selected

FIGURE 3 White pages software geocoded to the TIGER Network.

and stored as a new table or a sketch is made on a paper map to assist in digitizing the network.

Step 3: Digitizing the Attributed Network

Following the initial network definition, the next step digitizes the attributed link-node network. To automate this step, a MapBasic program was written that "walks" the user through a series of steps and enters default data values (note that any values entered can be manually changed by the user). For agencies without MapInfo, the network can be digitized within the GIS of choice attributing the node and link tables with the appropriate information. This requires digitizing points at all centroid and intersection locations and lines for each roadway in the model with the attribute information following the travel model structure.

Step 4: Calculate Productions and Attractions

With the zone table and socio-economic data stored in the GIS, the user can calculate zonal productions and attractions for the community. We begin by outlining two methods to perform the internal-internal calculation. First, if the user has census and employment data and has developed zones closely following block groups, then the user should follow the NCHRP 187 methodology to calculate production and attractions based on the given equations. Second, if the zones were not developed to follow the block-groups and either the census and employment data are not available, the user should begin by aggregating the number of households and businesses from the "white pages" software to the zonal level. These zonal totals can then be used to calculate the number of productions as the number of households multiplied by a factor representing total trips per household (9.2 trips per household is the national average) and calculate the attractions as a ratio of the zonal businesses to total businesses multiplied with total number of attractions.

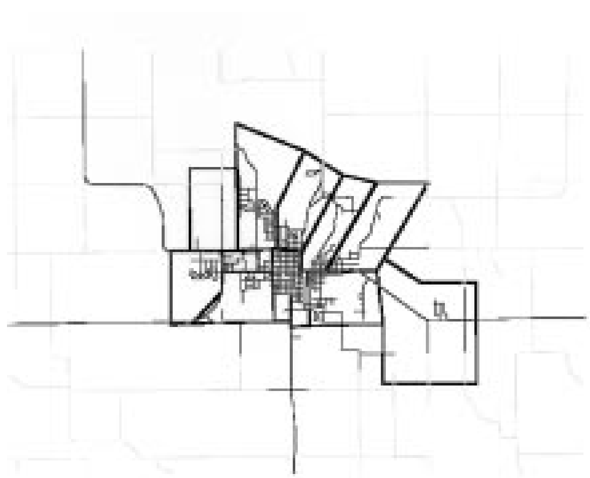

FIGURE 4 Traffic analysis zones developed for Waverly, Iowa.

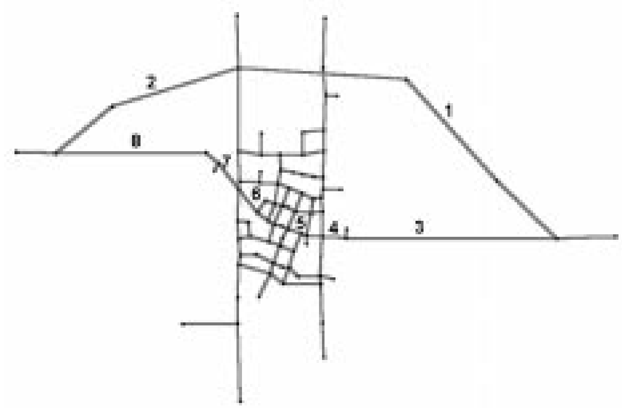

FIGURE 5 Location of Carroll traffic counts.

After the internal-internal calculation, the model requires external-external and external-internal data. The main requirement for the external trips is defining the percentages of vehicles which will stop in the study area versus the vehicles passing through to an ultimate destination. A novel methodology for determining the percentage of vehicles into or through town uses an economic gravity model of community attractiveness. The economic model defines the relationship between the attractiveness of the study community versus surrounding communities. This model is defined by the following equation (*3*):

$$\mathrm{PROB}_{ax} = \frac{\left(\dfrac{POP_x}{D_{ax}}\right)}{\left(\Sigma\dfrac{POP_x}{D_{ax}}\right)}$$

where,

PROB_{ax} = the probability that a customer located at a, which is D_{ax} miles from x, will patronize x,

POP_a = population of city a,

D_{xa} = distance from city a to city x.

The fundamental application of this model is based on the central place hierarchy stating that towns of comparable size should have similar business functions with larger towns possessing more amenities to compete for resident's business. Incorporating this principle into the project requires examining the communities in surrounding the study area to identify towns matching the study area's socio-economic characteristics. Entering probability that a customer from one town will patronize another into a spreadsheet program allows the user to develop the appropriate splits for external traffic.

Step 5: Validate Base Model and Develop New Scenarios

The initial model should be run through the travel demand software and the assigned volumes should validate the model using traffic counts. Some manipulation of network attributes might be required to improve validation. After validating the base model, the model should be altered to include the highway bypass and the travel model used to assign traffic. To assist the users, another

TABLE 1 Differences between DOT and GIS Methodologies for Carroll

Location	DOT volume	GIS-based volume	Difference
On northern bypass			
1 east bypass	2,900	2,846	-54
2 west bypass	2,100	2,500	400
On residual roadway			
3 east of Carroll to Monterry Drive	4,800	5,245	445
4 Monterry Drive to Grant Road	6,950	8,197	1,247
5 Grant Road to Main Street	10,800	9,021	-1,779
6 Main Street to US 71	11,100	12,682	1,582
7 US 71 to Burgess Avenue	5,700	5,645	-55
8 Burgess Avenue to west of Carroll	3,500	3,175	-325

MapBasic program was written to automate the development of new model scenarios.

Step 6: Forecast Future Traffic Volumes

The final step is to develop forecasts for future traffic in the area. To perform this step, traffic expansion factors normally used should be applied to the assigned volumes on the network links.

CASE STUDIES

The GIS-based bypass development methodology case study was performed for the community of Carroll, Iowa. Projected traffic volumes resulting from the GIS-based methodology were compared to the results of the existing forecasting method used by the Iowa DOT (*4*). Forecasted volumes for both the bypass and residual roadway were similar using the GIS-based methodology to the existing DOT procedure. The differences, which generally range from less than ten to twenty percent are outlined in Table 1 with Figure 5 showing the traffic count locations. These differences generally do not have serious implications for roadway design, but may have a limited effect on estimates of economic impact performed after the traffic study.

CONCLUSIONS

The proposed methodology for developing highway bypass plans within a geographic information system (GIS) environment seems promising, as it provides consistent, reproducible results with fewer staff resources. Using the GIS-based modeling approach, bypass traffic forecasts were obtained in less than one week (the original forecasts took three months). As the new model and data are maintained in a GIS, analysis and display of data are greatly enhanced. In addition, an analyst using the methodology is able to quickly analyze traffic conditions for many different alignment and access scenarios.

ACKNOWLEDGMENTS

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PCC Pavement Deterioration and Expansive Mineral Growth

HYOMIN LEE, ANITA M. CODY, ROBERT D. CODY AND PAUL G. SPRY

In order to evaluate the importance of newly-formed minerals in the premature deterioration of Iowa highway concrete, a two-phase study was undertaken. In the first phase, we performed petrographic and SEM/EDAX analyses to determine chemical and mineralogical changes in the aggregate and cement paste of samples taken from Iowa concrete highways that showed premature deterioration. In the second phase, we experimentally simulated environmental changes occurring in highway concrete after different deicer chemicals were applied in order to evaluate the role of deicers in premature deterioration. In highways exhibiting premature concrete deterioration, ettringite, $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$, completely fills many small voids and lines the walls of larger voids. Microscopic ettringite is also commonly disseminated throughout the paste of many samples. Severe cracking of cement paste is usually associated with ettringite locations, and strongly suggests that ettringite contributed to deterioration. Pyrite, FeS_2, is present in coarse/fine aggregates in several concretes. Sulfate ions released by its oxidation contribute to ettringite formation. In poorly performing concretes containing reactive dolomite aggregate, brucite, $Mg(OH)_2$, resulting from partial dedolomitization of the aggregate, was most common. No cracking was observed to be spatially associated with brucite, but most brucite crystals are microscopic in size and widely disseminated in the cement paste of less durable concretes. Expansion stresses associated with its growth at many microlocations may be relieved by cracking at weaker sites in the concrete. In the experimental phase of the study we found that each deicer salt can cause characteristic concrete deterioration by altering dedolomitization rims at the coarse-aggregate paste interface, by altering cement paste, and/or by forming new expansive minerals in the paste. Magnesium in deicer solutions produces the most severe paste deterioration by forming non-cementitious magnesium silicate hydrate and brucite. Chloride in deicer solutions promotes decalcification of paste and alters ettringite to chloroaluminate. Acetate seems to accentuate Mg-induced deterioration. Magnesium chloride, calcium magnesium acetate (Ca_3Mg_7Ac), and magnesium acetate were the most deleterious. Key words: concrete, deterioration, deicers, highways.

INTRODUCTION AND PROBLEM STATEMENT

Iowa highways constructed of concrete containing carbonate coarse aggregate from certain quarries sometimes have service lives of less than 10 years. Premature deterioration can be caused by many factors. The current report investigates the role that potentially expansive minerals have in their premature deterioration and whether deicer applications accentuate deterioration.

Considerable progress has been made in reducing premature failure of highway concrete, but problem concretes such as those studied here continue to fail prematurely. One area of uncertainty about the causes of failure is the significance of newly-formed minerals in concrete. Two common secondary minerals, brucite, $Mg(OH)_2$ and ettringite, $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$ or $Ca_6Al_2(SO_4)_3(OH)_{12} \cdot 26H_2O$, are often implicated in premature deterioration, and the cause of deterioration is often attributed to expansion and cracking related to their growth. The importance of expansion in the deterioration of concrete by growth of these minerals is still controversial and not accepted by all workers (*1*). Primary ettringite, which grows when concrete is still plastic, easily pushes other materials aside and is not harmful, but ettringite that forms long after concrete has hardened (delayed ettringite) may produce damaging expansive pressures (*2*). Delayed ettringite formation is especially enhanced by the availability of sulfur, because ettringite's other components, calcium, aluminum, and water are abundant in Portland cement concrete. Sulfur can be derived from gypsum added to the concrete to delay setting, from sulfate-containing ground or surface waters that infiltrate into the concrete, from sulfate impurities in road salt, or from oxidation of sulfide minerals that occur in coarse or fine aggregate.

PROCEDURES

Personnel of the Iowa Department of Transportation collected cores from seven different Iowa concrete highways that contain limestone and/or dolomite coarse aggregate from different sources and have different service records (Table 1). Each of the four-inch diameter concrete highway cores was cut into small rectangular blocks, approximately 2cm x 2cm x 4cm. Similar blocks were cut for use in the experimental phase of the study. Polished thin-sections were made from blocks from the top (1 in. from top of the road surface) and bottom (1 in. from the bottom) portions of each core. Petrographic analyses of thin-sections were conducted with both transmitted and reflected light utilizing a standard petrographic polarizing microscope. These analyses were used to identify specific areas to be studied by scanning electron microscope and to supplement observations of features difficult to observe with scanning electron microscopy such as color changes on coarse aggregate margins.

An Hitachi S 2460 reduced-vacuum scanning electron microscope was used for electron microscopy. Back-scattered images

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TABLE 1 Concrete Core Locations and Other Data for Iowa Highway Concretes

Core Location	Year*	Coarse Aggregate Source	Portland Cement
I-35, Cerro Gordo Co.	1974	Portland West quarry, Shellrock Fm.	Northwestern I
US 30, Linn Co.	1981	Crawford Lee quarry, Spring Grove Member, Wapsipinicon Fm.	Lehigh I
IA 9, Howard Co.	1974	Dotzler quarry, Spillville Fm.	Lehigh I
IA 21, Iowa Co.	1982	Crawford Lee quarry, Spring Grove Member, Wapsipinicon Fm.	Martin Marietta (?)
US 63, Howard Co.	1971	Nelson quarry, Cedar Valley Fm.	Dewey I
US 20, Dubuque Co.	1988	Sundheim quarry, Hopkinton Fm	Davenport I
IA 100, Linn Co.	1989	Crawford Lee quarry, Spring Grove Member, Wapsipinicon Fm.	Continental III
US 63, Tama Co.	1972	Smith quarry, Coralville Member, Cedar Valley Fm.	Lehigh I
US 151, Linn Co.	1947	Paralta quarry, Otis Member, Wapsipinicon Fm.	Mixed (Medusa, Lehigh, Dewey, Atlas, Alpha)
US 218, Benton Co.	1971	Garrison quarry, Coralville Member, Cedar Valley Fm.	Davenport I
US 20, Dubuque Co.	1988	Sundheim quarry, Hopkinton Fm.	Davenport I

*Year the highway was constructed.

were taken and energy dispersive analytical x-ray (EDAX) area mapping was performed for Si, Al, K, Na, O, Ca, Mg, S, Cl, and Fe. EDAX point analyses were obtained at high magnification for qualitative mineral identification. An accelerating voltage of 15 kV was generally used for imaging whereas EDAX point analyses were obtained at 20 kV.

RESULTS AND DISCUSSION

Introduction

The majority of core samples were concretes constructed with dolomite coarse aggregate. Previous research divided them into two groups, durable and non-durable concretes, based on their service records (*3*,*4*). The term “durable concrete” was used for the highway concretes that had extended service lives of >40 years before significant deterioration, and “non-durable concrete” was used for concretes with service lives of <16 years. The highway concrete containing Sundheim aggregate (Table 1) is classified as durable; all others were non-durable. Our use of the two terms has no necessary correspondence with ASTM-defined durability. Durability correlates with dolomite coarse aggregate reactivity. Poorly performing concretes contain fine-grained, poorly-crystallized, microporous dolomite that has reacted with surrounding concrete paste to produce dark and light-colored partially-dedolomitized rims surrounding the dolomite aggregate fragments (*4*).

Part One: Newly Formed Minerals in Iowa Highway Concrete

Abundant brucite and ettringite were observed in most of the highway concretes studied, and large amounts of calcite mineralization occurred in the outer regions of partially dedolomitized dolomite aggregate rims. Ettringite was the most abundant secondary mineral, followed by calcite, and then by brucite. ettringite and brucite were the only potentially expansive substances identified by petrographic microscope and electron imaging methods.

The abundance and size of brucite crystal masses are closely related to dolomite coarse aggregate reactivity. In highway concretes containing reactive dolomite aggregate, abundant, <20 µm diameter, brucite masses commonly occur in the cement paste near dolomite aggregate/cement paste interfaces (Figure 1). These crystals are not usually associated with void spaces. Larger, irregular micro-nodules are also disseminated in the paste at many locations far from dolomite aggregate particles. The latter occurrence indicates that significant quantities of dolomite-derived Mg^{2+} migrate considerable distances before precipitation. In highway concrete with non-reactive dolomite or limestone aggregate, much fewer and smaller masses of brucite are observed. Brucite occurrences show no obvious spatial correlation with cracks in either cement paste or coarse aggregate.

In our Iowa highway concrete samples, ettringite chiefly occurs in air-entrainment void spaces where it grows as needle-like crystals projecting from the void walls. It occurs in two forms in these air-entrainment voids (Figure 2). The first type is void-fill ettringite

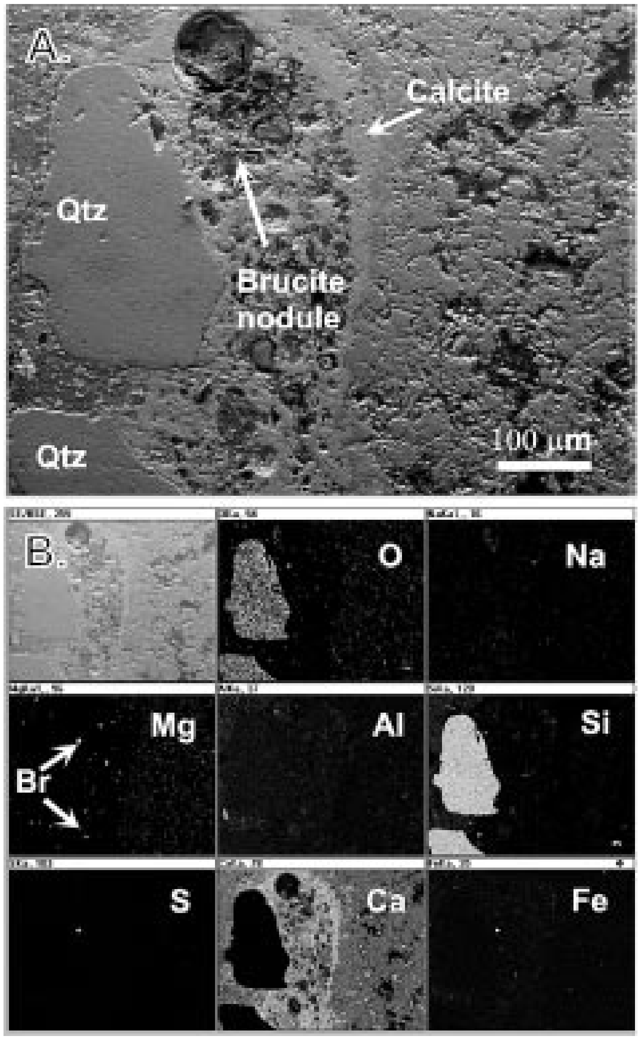

FIGURE 1 Brucite occurrence and calcite enrichment in dolomite coarse aggregate and cement paste. A. SEM micrograph. B. EDAX area mapping of same area. Note the calcite enrichment in the outer rim of the coarse aggregate fragment in the Ca SEM view and the Ca map, and the brucite (Br) occurence as shown by small white areas and dots in the Mg map. Qtz = quartz. Dotzler quarry aggregate.

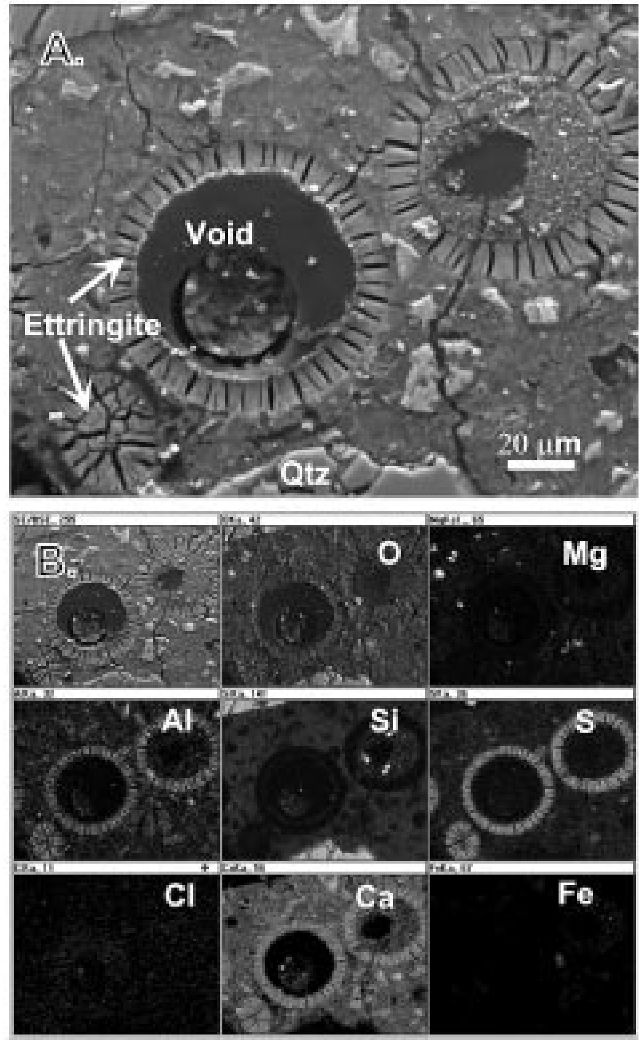

FIGURE 2 Void-rim and void-fill ettringite. A. SEM micrograph. B. EDAX map of same area. Note the radial cracks associated with the ettringite, and that some of the cracks extend outward into the concrete paste. Brucite crystals can be detected in the upper left and in other areas of the Mg EDAX map. Smith quarry aggregate.

in which the mineral completely fills air-entrainment voids that are usually less than about 100 µm in diameter. Abundant cracks, which are irregular and very disruptive, occur in the ettringite fills. The second type is void-rim ettringite that occurs as rims of ettringite lining the margin of voids. This type usually formed in air-entrainment voids of diameter greater than about 100 µm. Large radially-oriented cracks are prominent throughout the ettringite rims. Some of the cracks in both void-fill and void-rim ettringite continue into the cement paste, but ettringite does not occur in them documenting that ettringite had formed before cracking. Minor amounts of ettringite also fill microscopic interstitial pores in the cement paste and are visible in high-magnification back-scattered SEM images. Very rarely, ettringite occurs in cracks formed along the boundary between quartz fine aggregate particles and cement paste.

Relationship of Ettringite to Pyrite Inclusions

Sulfate is a necessary component for the formation of ettringite in the cement paste, so that oxidation of sulfide minerals in concrete coarse and fine aggregate may promote delayed ettringite formation (*5*). Ettringite typically occurs in the cement paste near dolomite aggregates that contain pyrite inclusions, and ettringite abundance is closely associated with the amount of pyrite oxidation as evidenced by the quantities of goethite and/or ferrihydrite associated with the pyrite. Pyrite inclusions in reactive dolomite aggregate are more oxidized than those in non-reactive aggregate because of greater microporosity and finer dolomite crystal sizes in the reactive material that allow oxidizing solutions to react with pyrite.

Part Two: Experimental Observations

Experimental Methods

Small 3cm x 1.5cm x 1.5cm blocks weighing between 7g and 11g were cut from the seven highway cores examined in the first section of this report. Two blocks from each core were immersed in 100 ml of solution and sealed in cleaned polymethylpentene containers that were stored for 132 hours at 58°C in a constant temperature chamber. The solutions used were 0.75 M $CaCl_2 \cdot 2H_2O$, $MgCl_2 \cdot 6H_2O$, NaCl, CMA based on a molar ratio of 3:7, i.e. $3[Ca(CH_3COO)_2 \cdot H_2O]:7[Mg(CH_3COO)_2 \cdot 4H_2O]$, Na_2SO_4, $Ca(CH_3COO)_2 \cdot H_2O$, $Mg(CH_3COO)_2 \cdot 4H_2O$, and distilled water. All solutions contained 0.01% sodium azide to control bacterial growth. Two types of experiments were performed. Wet/Dry Cycling: After being immersed in 58°C solutions for 132 hours, blocks were removed from the solutions, dried 58°C (ª135°F) for 24 hours, air cooled to 25°C, returned to their immersion solutions at 25°C, and again stored at 58°C for 132 hours. Freeze/thaw cycling: Samples removed from the 58°C solutions after 132 hours were air cooled to 25°C and stored for 24 hours in a freezer at -4°C (25°F). The blocks were air warmed to 25°C, returned to their respective solutions at 25°C, and stored at 58°C for 132 hours. Both types of experiments were continued until the blocks exhibited cracking or crumbling, at which time they were rinsed, dried, and prepared for petrographic and SEM analysis. These experiments were similar to those reported by Cody et al. (1996) but the current experiments used less concentrated solutions in order to more closely simulate road conditions where deicers may be applied, and expanded the previous study to include acetates and Na sulfate.

Relative Aggressiveness of Salt Solutions

General conclusions about the effects of these experiments are as follows.

Acetates. Calcium magnesium acetate solutions were the most damaging of all solutions tested. Wet/dry and freeze/thaw cycling in CaMg-acetate produced widespread and severe damage with cracking from replacement of calcium silicate hydrate with non-cementitious magnesium silicate hydrate. Brucite formation was extremely copious, and it was disseminated throughout the cement paste and in voids. It also occurred at the paste-fine aggregate interface where it caused debonding of fine aggregate. Mg-acetate produced similar but slightly less damage, and Ca-acetate solutions produced much less alteration. CaMg-acetate dissolved the cement paste and altered quartz fine aggregate but it is still not clear why it is more deleterious than Mg-chloride or Mg-acetate. We should point out that we mixed our own CaMg-acetate and did not use a commercial variety; commercial formulations of CMA may not have the same aggressiveness to concrete as our mixture of calcium acetate and magnesium acetate.

Sulfates. Sodium sulfate solutions were next to CaMg-acetate and Mg-acetate in aggressiveness. Both wet/dry and freeze/thaw cycling in these solutions produced severe expansion cracking, with wet/dry conditions being worse. Sulfate solutions applied to Sundheim concrete that previously did not contain ettringite produced abundant ettringite disseminated throughout the paste and in voids, and cracking resulted. Deterioration by ettringite expansion is clearly evidenced by these experiments.

Chlorides. Magnesium chloride produced significant concrete crumbling because of widespread replacement of CSH by non-cementitious MSH. Our research results show that calcium chloride deicing salts caused characteristic deterioration in concretes with reactive dolomite aggregates by enhancing dedolomitization that releases magnesium to form destructive brucite and MSH. Sodium chloride solutions did not cause significant changes except that chloride causes the formation of chloroaluminate, probably trichloroaluminate produced from pre-existing ettringite.

Magnesium. In our experiments, the magnesium component of deicer salts proved to be the most deleterious. Magnesium promoted replacement of CSH by non-cementitious MSH with resultant paste shrinkage and cracking. The growth of abundant, potentially-expansive brucite especially in the paste-fine aggregate interface furthered debonding of fine aggregate.

CONCLUSIONS

Our studies of Iowa highway concrete deterioration strongly support the contention that expansive mineral growth is at least partly responsible for premature deterioration. The evidence concerning brucite expansion is, however, not conclusive. In spite of numerous studies that have concluded brucite growth is expansive (*6*), we found no evidence for microcracking directly associated with brucite crystal occurrences. The only supporting data is that analysis of brucite precipitation reactions (*6*) shows that there is a theoretical expansion when cement and dolomite components react to produce brucite. It is also observed that brucite is most common in poorly-performing concretes that exhibit expansion phenomena such as D-cracking.

Our evidence that ettringite creates expansion cracking is stronger than that for brucite. The exact mechanism of cracking is uncertain, but we conclude that abundance of delayed ettringite growth in poorly-performing concretes and the close association with microcracks is good evidence for ettringite growth induced deterioration. This conclusion is supported by experiments showing that abundant cracks accompanied newly-formed ettringite.

Experiments with different potential deicers and Na-sulfate document that all of these chemicals cause concrete deterioration. Magnesium solutions are especially damaging, and consequently support our conclusion that magnesium released from dolomite coarse aggregate decreases service life of concretes. The enhanced damage by calcium magnesium acetate was unexpected and suggests that the effects of CMA on highway concretes should be closely monitored over a prolonged period to determine if similar effects develop over long-term use.

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Evaluation of Mix Time on Concrete Consistency and Consolidation

JAMES K. CABLE

In an effort to look at ways to improve the quality of the finished concrete pavement product, the Iowa Department of Transportation and ISU conducted research on two existing highway projects in 1997. The objectives of that research included evaluation of two alternative types of central mixers, the effect of the type of hauling equipment, the mix design, and the mixing time employed to produce the mix on the consistency, consolidation and air matrix in the concrete pavement. The concrete was tested at the plant site and at the grade by standard methods employed in the testing of ready mixed concrete delivery. The results of that research have provided some insight into the consistency and consolidation of the concrete produced and placed. In addition, core analysis of the hardened concrete by the Scanning Electron Microscope (SEM) have given more information relative to the impact of the paving operation on the air matrix on the finished product. This paper will report some of those results and indicate how they may be shaping the future mix design and construction of concrete pavements.
Key words: mix design, concrete, pavements, mix time.

INTRODUCTION

In an effort to evaluate ways to continuously improve the paving products delivered to the public, the Iowa Department of Transportation (DOT) in conjunction with the Iowa Highway Research Board (IHRB) and the Civil and Construction Engineering (CCE) Department of Iowa State University entered into Project HR-1066. "Effect of Mix times on Portland Cement Concrete (PCC) Properties" was an effort to look at the concrete quality from mixing to consolidation at the paving site.

RESEARCH OBJECTIVES

The research effort was directed at collection and evaluation of data relating mixing time to:

a. Hardened air content and distribution
b. Potential segregation in the hauling units
c. Concrete consolidation quality at the paving site
d. Workability of the concrete at the paving site.

The long term goal of the Iowa DOT in this work was the development of a performance based specification for Portland cement concrete pavement construction that measures quality, consistency, hardened air content and pavement strength at the construction site.

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DATA COLLECTION METHODS

Data was collected at two separate construction sites (Carlisle and Carroll, Iowa) under contract to the same contractor by the Iowa DOT. At the Carlisle site, a conventional central drum mix plant was employed to produce the concrete. A horizontal drum mixer with blades that moved within the drum was employed at the Carroll site. The research staff chose to employ the plastic concrete tests outlined in ASTM, C-94 specification that pertains to measuring consistency and quality of concrete in a ready mix truck or agitor. In the case of the mix design, the contractor was also allowed to use a second mix design of their choosing in separate tests at the Carlisle site. All testing was done at the field concrete plant site with the cooperation of the contractor and the Iowa DOT staff. Samples were obtained from the concrete hauling units, selected at random, as they left the plant site and at the paver when the same load was consolidated.

Mixing time was measured visually at the plant site as the time elapsed between the introduction of all the materials into the mixer and the initial mix delivery into the hauling unit. Nominal mixing times chosen for each site were as follows:

Carroll (Iowa DOT mix) - 30 and 45 seconds
Carlisle (Iowa DOT mix) - 45, 60 and 90 seconds
Carlisle (Contractor mix) - 45, 60 and 90 seconds

Tests were conducted on the plastic concrete samples in accordance with ASTM C-94 specification, on samples collected at the plant site from the center and side of the hauling unit load and in the field directly in front of the paver. Concrete parameters tested included:

a. Slump, as per ASTM standard C143
b. Unit Weight as per ASTM standard C138
c. Air content as per ASTM standard C231
d. Retained Course Aggregate as per ASTM standard C94.

Compressive cylinders were constructed from samples from each of the truck load used in the plastic concrete tests. Cores were also extracted from the hardened concrete in the areas where the test truck loads of concrete were placed and compressive strengths were determined in accordance with ASTM C42 test procedures.

ANALYSIS RESULTS

The field data was summarized in terms of the information gathered from each of the hauling unit loads selected in the sample. This means that at each of the construction sites, test results were collected for randomly selected truck loads of concrete at the plant site. Material in these units were tested at the plant site and at the paver when the material was deposited on the grade. The results of physical tests at both sites, cylinders constructed at the plant and cores extracted from the pavement in the area where the load was placed, were compared statistically relative to mix type and time of mixing. The detailed results of that work are contained in the project report for HR-1066.

A summary of the statistical analysis of variance (ANOVA) for the physical tests of slump, unit weight, air content and retained coarse aggregate yielded the following information:

1. The 30-45 second mixing times for the Carroll DOT mix and alternative mixer indicate no significant differences in slump, unit weight, air content and retained coarse aggregate.
2. The selected mixing times for the Carlisle DOT mix and the conventional drum mixer indicate that increasing the mixing time from 45 to 60, 45 to 90 and 60 to 90 seconds lead to a significant increase in the air content retained in the final product. There were no significant changes noted in any of the other physical test results.
3. The Carlisle contractor mix results indicate a significant increase in unit weight and a reduction in air content when the mixing time was increased from 45 to 60 seconds. However, increasing the mixing time from 60 to 90 seconds indicated a significant difference in the concrete unit weight and retained coarse aggregate test results.

In terms of sampling location the ANOVA tests provided the following answers:

1. For all mix types and mixing times, sampling at the center or side of the truck provided no significant difference in the dependent variables.
2. The Carroll DOT mix did show significant differences in the retained coarse aggregate between side and center of the truck and the grade samples. The same result was also shown when the test for the Carlisle contractor design mix was evaluated.
3. The tests indicate that longer mixing times led to significantly different air contents in the samples taken from the center and side of the truck and at the grade.

ANOVA analysis of the cylinders and cores from each of the test sampling areas at the plant and behind the paver resulted in the following information:

1. The effect of mixing time for the Carroll Iowa DOT mix and the Carlisle contractor designed mix do indicate that longer mixing time did create significantly increase compressive strengths for in both the cylinders and cores.
2. The Carlisle Iowa DOT mix cylinders and core compressive strengths decreased as the mixing time increased from 45 to 60 and 60 to 90 seconds.

An ANOVA in the results of the SEM analysis for the cores and cylinders identified the following results:

1. The Carlisle Iowa DOT mix and the Carroll Iowa DOT mix show no significant difference in average air content for the mixing times compared.
2. The Carlisle contractor designed mix does indicate a significant difference in the average air content for all mixing times compared. The average air content across the test specimen increases as the time of mixing increases from 45 to 90 and 60 to 90 seconds, but decreases when the mixing time is changed from 45 to 60 seconds.

Field visual evaluation of the mix at both the plant site and the grade did yield some information that is difficult to identify in the test results at the Carroll construction site. Mixing at the 30 and 45 seconds yielded visible sand seams (uncoated sand particles) in the discharged materials. Some of this phenomenon was visually present in the truck at the plant testing site. These sand seams were still present in the truck when the material was discharged into the paver. The concrete produced under this set of mixing conditions was difficult to place and finish.

CONCLUSIONS

This research was directed at evaluating the effect of mixing time on the physical characteristics of the finished Portland cement concrete pavement. It considered three mix designs, two difference concrete mixers and four mixing times. The results of the research, compared to the research objectives, indicate the following conclusions:

1. Potential segregation in the hauling units: Dump truck type hauling units do not significantly change or decrease the quality of the material being delivered to the paver and should continue to be allowed in addition to agitor type hauling vehicles for transport of Portland cement concrete paving materials.
2. Concrete consolidation and workability quality at the paving site: The results of the ANOVA indicate that mixing times of 60 seconds or greater do have a positive influence on the physical characteristics of the concrete product. It is recommended that the 60 second minimum mixing time be retained for all mixer types at this time.
3. Hardened air content and distribution: The data from this set of tests indicates that for Iowa DOT designed mixes the mixing time did not effect the physical attributes of the concrete significantly. The results did show a conflicting ideas for the contractor designed mix. We suggest that this is the result of both a different matrix of coarse and fine aggregate in the contractor mix as related to previous Iowa DOT mixes. It does open a new set of parameters for mix approval when coupled with the impact of mix admixtures being used or considered. It is recommended that contractor mix designs be thoroughly laboratory tested prior to construction to determine the impact of admixtures and the differences in aggregate/cement matrix on the desired physical performance factors desired by the agency.
4. Concrete mixer type and mixing times: Visual and physical test data indicate that reduced mixing times for alternative type mixers should only be allowed when steps have been taken to change the mixing process to eliminate any particles of aggregate that are not coated upon discharge into the hauling unit.

ACKNOWLEDGMENTS

The research staff appreciates the help and assistance provided by the staff of the Cedar Valley Construction Co. Inc. at both the construction sites and the cooperation provided by the Iowa DOT Des Moines Construction and Denison Construction Residencies in this effort. It would not have been possible without their support and cooperation.

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Evaluation of Glass Fiber Reinforced Plastic Dowels as Load Transfer Devices in Highway Pavement Slabs

DUSTIN DAVIS AND MAX L. PORTER

The use of dowel bars, fabricated from glass fiber reinforced plastic (GFRP), as load transfer devices in highway pavement slabs is a possible solution to the corrosion problems related to the current use of steel dowels. The material properties of GFRP are considerably different from those of steel. Therefore, to keep material stresses within permissible limits, the diameter and spacing used for steel dowels is no longer valid for GFRP dowels. This paper presents a design procedure for determining the required diameter and spacing for GFRP dowels based on the load transferred through the critical dowel. Essentially, the diameter and spacing requirements for GFRP dowels is based on an equivalent deflection for a joint containing steel dowels spaced at the standard 300 mm (12 in.). GFRP dowels appear to be a feasible solution as long as the diameter of the dowel is increased, spacing decreased, or a combination of both. Key words: concrete pavements, glass fiber reinforced plastics, dowels, doweled joints.

INTRODUCTION

A considerable amount of the nation's transportation infrastructure is in need of repair or replacement because of deterioration resulting from pavement reinforcement corrosion. New construction methods and new materials are needed to protect the infrastructure in order to avoid this type of deterioration. An obvious method of controlling this deterioration is to use a material that is naturally resistant to corrosive environments such as glass fiber reinforced plastic (GFRP).

Load transfer devices are structural members placed at locations of transverse joints in highway pavements that act to transfer shear across the joint. Since these devices are placed along the length of the joint, they are susceptible to de-icing salts. The steel dowels which are currently used as load transfer devices corrode when exposed to these salts and bind or lock the joint, resulting in undesirable stresses. Therefore, the non-corrosive properties of GFRP make it an ideal material for use as a load transfer device in concrete highway pavement slabs.

Since the material properties of GFRP are different from those of steel, the diameter and spacing commonly used for steel dowels is no longer valid for GFRP dowels. This paper presents a design procedure for determining the required diameter and spacing for GFRP dowels based on the load transferred through the critical dowel.

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DESIGN PROCEDURE

The diameter and spacing required for GFRP dowels can be determined by equating the relative deflection of a joint doweled with steel dowels to that of a joint containing GFRP dowels. For a specific diameter and spacing, the relative deflection between adjacent slabs can be determined. The dowel bar diameter and spacing which results in a deflection equivalent to that for a joint containing steel dowels spaced at the standard 300 mm (12 in.) is the desired diameter and spacing for GFRP dowels.

As shown in Figure 1, the relative deflection between adjacent pavement slabs, Δ, consists of two components: the deflection of the dowel bar within the pavement at the face of the joint, y_o, and the shear deflection of the dowel bar across the joint, δ (*1*). The relative joint deflection is given in Equation (1):

$$\Delta = 2y_o + \delta$$

The deflection of the dowel relative to the concrete, at the face of the joint, was developed by Friberg (*2*) for design purposes and is given in Equation (2):

$$y_o = \frac{P_t}{4\beta^3 EI}(2 + \beta z)$$

where:

$$b = \sqrt[4]{\frac{Kb}{4EI}}$$

K = modulus of dowel support
b = dowel bar diameter
E = modulus of elasticity of the dowel bar
I = moment of inertia of the dowel bar
P_t = load transferred by critical dowel
z = joint width

Friberg's equation is based upon the theoretical model developed by Timoshenko and Lessells (*3*) for the analysis of beams on elastic foundations. Friberg's equation was derived assuming a dowel bar of semi-infinite length. Dowel bars used in practice are of finite length, therefore, this equation would not apply. However, Albertson (*4*) has shown that this equation can be applied to dowel bars with a βL value greater than or equal to 2 with little or

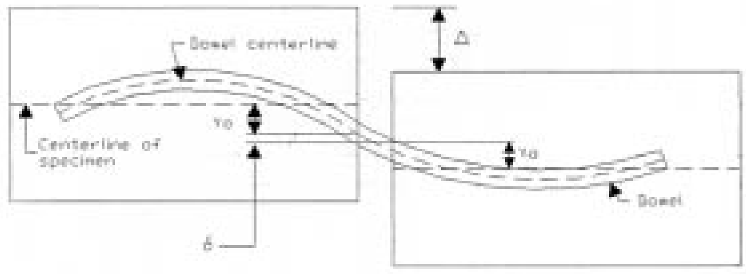

FIGURE 1 Relative deflection between pavement slabs.

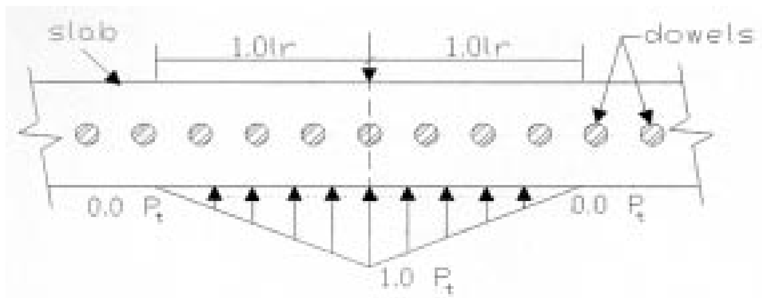

FIGURE 2 Load transfer distribution.

no error. The length of the dowel bar embedded in one side of the slab is denoted as L.

The shear deflection of the dowel across the joint is given in Equation (3):

$$\delta = \frac{\lambda P_t z}{AG}$$

where:
λ= form factor, equal to 10/9 for solid circular section
A = cross-sectional area of the dowel bar
G = shear modulus
P_t = load transferred by critical dowel
z = joint width

Particular attention should be paid to how the value of the shear modulus is obtained for GFRP dowels. Since FRP materials are anisotropic, the shear modulus for a GFRP dowel must be determined by composite materials theory. The procedure for determining the shear modulus is quite involved, however, this value can usually be obtained from the manufacturer.

Modulus of Dowel Support

The modulus of dowel support, K, is the reaction per unit area causing a unit deflection. The value of K must be determined empirically due to the difficulty in establishing a value theoretically. Due to the lack of experimentally determined values of K, a value of 407 Gpa/m (1,500,000 pci) was adopted by the authors as suggested by Yoder and Witczak (*5*).

Load Transferred by Critical Dowel

When a load is applied to the edge of a slab, a portion of that load is transferred to the adjacent slab through the dowels by shear. Tabatabie et al. (*6*) suggested that only the dowels contained within a distance of $1.0\ell_r$ from the load are active in transferring the load where ℓ_r is the radius of relative stiffness, defined by Westergaard (*7*) as follows:

$$\ell_r = \sqrt[4]{\frac{E_c h^3}{12(1-\mu^2)k}}$$

where:
E_c = modulus of elasticity of the pavement concrete
h = pavement thickness
μ = Poisson's ratio for the pavement concrete
k = modulus of subgrade reaction

Tabatabie also proposed a linear distribution of the load transferred across the joint as shown in Figure 2.

If 100 percent efficiency is achieved in load transfer by the dowel bars, 50 percent of the wheel load would be transferred to the subgrade while the other 50 percent would be transferred through the dowels to the adjacent slab. Repetitive loading of the joint

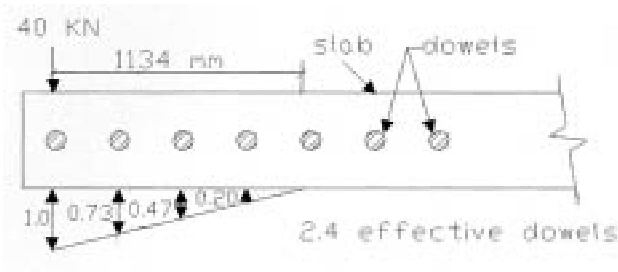

FIGURE 3 Effective dowels for 300 mm dowel bar spacing.

results in the creation of a void directly beneath the dowel at the face of the joint. According to Yoder and Witczak (*5*), a 5 to 10 percent reduction in load transfer occurs upon formation of this void; therefore, a design load transfer of 45 percent of the applied wheel load is recommended.

In their book, *Principles of Pavement Design*, Yoder and Witczak present a method for determining the load transferred by the critical dowel. In determining the load transferred by the critical dowel, Yoder and Witczak assumed that the deflection under a corner load would be greater than the deflection of the interior slab due to the same applied load. Thus, the corner dowel would be the critical dowel for edge loads (*5*). The load transferred by the critical dowel is given in Equation (5):

$$P_t = \frac{\text{Design Load Transfer}}{\text{Number of Effective Dowels}}$$

DESIGN EXAMPLE

Consider a 40 KN (9000 lbf) wheel load applied to a 250 mm (10 in.) thick concrete pavement slab with a compressive strength of 48 MPa (7000 psi). The pavement slab rests on a subbase having a modulus of subgrade reaction equal to 27 MPa/m (100 pci). Assuming a joint width of 6 mm (0.25 in.), determine the required spacing and diameter for GFRP dowels with the following properties:

modulus of elasticity = 41 GPa (6 x 10^6 psi)
shear modulus = 3.3 GPa (476,000 psi)
dowel bar length = 460 mm (18 in.)

Solution

1. Determine the Load Transferred by the Critical Dowel

$$\ell_r = \sqrt[4]{\frac{E_c h^3}{12(1-\mu^2)k}} = \sqrt[4]{\frac{(32.9)0.250^3}{12(1-0.2^2)0.027}}$$

= 1.134 m (45 in.)

The load transferred by the critical dowel is maximum when the wheel load is positioned directly over the dowel. For this loading condition and with the standard 300 mm (12 in.) spacing of dowels, 2.4 dowels are effectively active in transferring the load as shown in Figure 3. (The number of effective dowels was arrived at using U.S. customary units.)

Design load transfer = 0.45(40) = 18 KN (4045 lbf)

$$P_t = \frac{\text{Design Load Transfer}}{\text{Number of Effective Dowels}} = \frac{18}{2.4}$$

= 7.5 KN (1685 lbf)

2. Determine the Relative Deflection for a Joint Containing Steel Dowels

Assume the following properties for the steel dowels:
modulus of elasticity = 200 GPa (29 x 10^6 psi)
shear modulus = 77.5 GPa (11.24 x 10^6 psi)
dowel bar length = 460 mm (18 in.)

The recommended dowel bar diameter should be equal to 1/8 the slab thickness (*8*).

$$b = \frac{250}{8} = 31.25 \text{ mm}$$

Use a value of 31.75 mm for b since a 1.25 in. diameter bar will be used in the field.

$$I = \frac{\pi b^4}{64} = \frac{\pi(0.03175)^4}{64} = 5.0 \times 10^{-8}\ \text{m}^4\ (0.12\ \text{in}^4)$$

$$\beta = \sqrt[4]{\frac{Kb}{4EI}} = \sqrt[4]{\frac{(407)0.03175}{4(200)5.0\text{x}10^{-8}}} = 24\ \text{m}^{-1}\ (0.61\ \text{in}^{-1})$$

$$\beta L = \frac{24(0.46-0.006)}{2} = 5.45$$

Since βL is greater than 2, Equation (2) can be used to determine the deflection of the dowel relative to the concrete.

$$y_o = \frac{P_t}{4\beta^3 EI}(2+\beta z) =$$

$$\frac{0.0000075}{4(24)^3(200)5.0 \text{ x } 10^{-8}}\left(2+(24)0.006\right) = 2.91 \text{ x}$$

10^{-5} m (0.0011 in.)

$$\delta = \frac{\lambda P_t z}{AG} = \frac{(10/9)0.0000075(0.006)}{7.92 \text{ x } 10^{-4}(77.5)} =$$

8.15 x 10^{-7} m (3.21 x 10^{-5} in.)

$\Delta = 2y_o + \delta$ = 2(0.0291) + 0.000815 = 0.06 mm (0.0024 in)

3. Determine the Relative Deflection for a Joint Containing GFRP Dowels

b = 31.75 mm (1.25 in.)

$$I = \frac{\pi b^4}{64} = \frac{\pi(0.03175)^4}{64} = 5.0 \text{ x } 10^{-8} \text{ m}^4 \text{ (0.12 in}^4\text{)}$$

$$\beta = \sqrt[4]{\frac{Kb}{4EI}} = \sqrt[4]{\frac{(407)0.03175}{4(41)5.0\text{x}10^{-8}}} = 35 \text{ m}^{-1} \text{ (0.90 in}^{-1}\text{)}$$

$$\beta L = \frac{35(0.46 - 0.006)}{2} = 7.95$$

Since βL is greater than 2, Equation (2) can be used to determine the deflection of the dowel relative to the concrete.

$$y_o = \frac{P_t}{4\beta^3 EI}(2 + \beta z) =$$

$$\frac{0.0000075}{4(35)^3(41)5.0 \text{ x } 10^{-8}}(2 + (35)0.006) =$$

$4.71 \text{ x } 10^{-5}$ m (0.0018 in.)

$$\delta = \frac{\lambda P_t z}{AG} = \frac{(10/9)0.0000075(0.006)}{7.92 \text{ x } 10^{-4}(3.3)} =$$

$1.91 \text{ x } 10^{-5}$ m (0.0008 in.)

$\Delta = 2y_o + \delta = 2(0.0471) + 0.0191 = 0.1133$ mm (0.0044 in)

4. Determine the Required Diameter and Spacing for GFRP Dowels

Steps 1 and 3 were repeated for various diameters and spacings. The results of these calculations are presented in Table 1. The correct diameter and spacing for GFRP dowels is that which results in a relative deflection of approximately 0.06 mm (0.0025 in.). Table 2 lists the various combinations of diameter and spacing that work for this particular problem. Although this procedure typically results in material stresses within permissible limits, the authors recommend that the bearing stress in the concrete along with the flexural and shear stresses within the critical dowel be checked. The authors refer the reader to Friberg's paper, "Design of Dowels in Transverse Joints of Concrete Pavements," for equations to determine maximum bearing stress in the concrete and maximum shear and moment within the dowel (*2*).

CONCLUSIONS AND RECOMMENDATIONS

Typically, bearing stress in the concrete is the controlling factor in the design of doweled joints. If current design procedures were followed in determining the diameter and spacing for GFRP dowels, the resulting deflection of the dowel relative to the concrete would have been approximately 1.6 times that of an equivalent joint containing steel dowels. Since the bearing stress in the concrete is directly proportional to the deflection of the dowel relative to the concrete (Equation [6]), the concrete bearing stress also would have increased by a factor of 1.6:

$\delta_b = Ky_o$

TABLE 1 Relative Joint Deflections (mm)

	Dowel Bar Spacing (mm)			
Dowel Bar Diameter (mm)	300	250	200	150
31.75	0.11	0.10	0.08	0.06
38.10	0.09	0.07	0.06	0.05
44.45	0.06	0.05	0.04	0.03

* 1 in. = 25.4 mm

TABLE 2 Solution to Design Example

Dowel Bar Diameter (mm)	Dowel Bar Spacing (mm)
31.75	150
38.10	200
44.45	300

* 1 in. = 25.4 mm

In order to keep the bearing stresses in the concrete comparable to those for a steel doweled joint, the dowel bar diameter must be increased, spacing decreased, or a combination of both. Additional experimental evidence is needed to verify these predictions. Currently, Iowa State University is undergoing experimental investigations in the laboratory and field to verify these somewhat theoretical postulations mentioned in this paper.

GFRP dowel bars appear to be a feasible solution to the deterioration of the transverse joints of highway pavement slabs as long as the diameter of the dowel is increased, spacing decreased, or a combination of both. This adjustment is necessary in order to keep the deflection of the joint and thus the bearing stresses in the concrete equivalent to those experienced by their steel counterparts.

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INTRODUCTION

A considerable amount of the nation's transportation infrastructure is in need of repair or replacement because of deterioration resulting from pavement reinforcement corrosion. New construction methods and new materials are needed to protect the infrastructure in order to avoid this type of deterioration. An obvious method of controlling this deterioration is to use a material that is naturally resistant to corrosive environments such as glass fiber reinforced plastic (GFRP).

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$$\Delta = 2y_o + \delta$$

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$$y_o = \frac{P_t}{4\beta^3 EI}\left(2 + \beta z\right)$$

where:

$$b = \sqrt[4]{\frac{Kb}{4EI}}$$

K = modulus of dowel support
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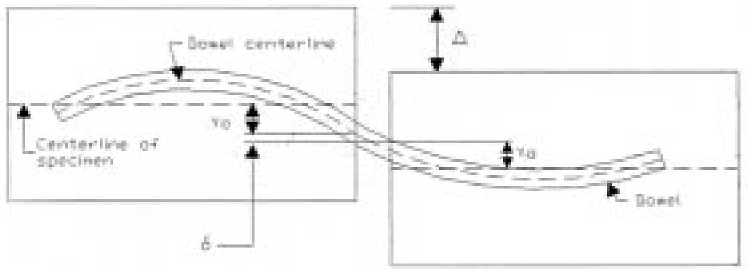

FIGURE 1 Relative deflection between pavement slabs.

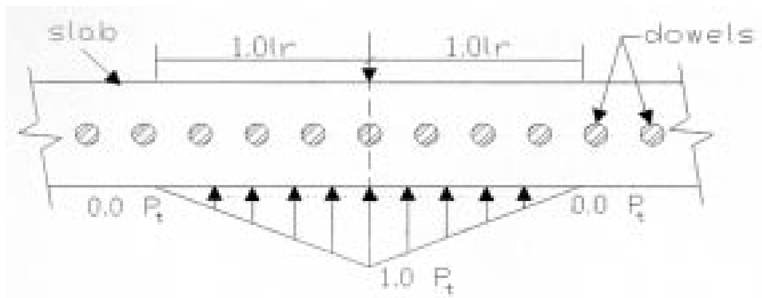

FIGURE 2 Load transfer distribution.

no error. The length of the dowel bar embedded in one side of the slab is denoted as L.

The shear deflection of the dowel across the joint is given in Equation (3):

$$\delta = \frac{\lambda P_t z}{AG}$$

where:
λ= form factor, equal to 10/9 for solid circular section
A = cross-sectional area of the dowel bar
G = shear modulus
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Particular attention should be paid to how the value of the shear modulus is obtained for GFRP dowels. Since FRP materials are anisotropic, the shear modulus for a GFRP dowel must be determined by composite materials theory. The procedure for determining the shear modulus is quite involved, however, this value can usually be obtained from the manufacturer.

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$$\ell_r = \sqrt[4]{\frac{E_c h^3}{12(1-\mu^2)k}}$$

where:
E_c = modulus of elasticity of the pavement concrete
h = pavement thickness
μ = Poisson's ratio for the pavement concrete
k = modulus of subgrade reaction

Tabatabie also proposed a linear distribution of the load transferred across the joint as shown in Figure 2.

If 100 percent efficiency is achieved in load transfer by the dowel bars, 50 percent of the wheel load would be transferred to the subgrade while the other 50 percent would be transferred through the dowels to the adjacent slab. Repetitive loading of the joint

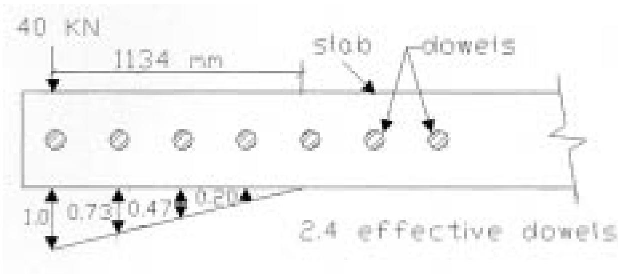

FIGURE 3 Effective dowels for 300 mm dowel bar spacing.

results in the creation of a void directly beneath the dowel at the face of the joint. According to Yoder and Witczak (*5*), a 5 to 10 percent reduction in load transfer occurs upon formation of this void; therefore, a design load transfer of 45 percent of the applied wheel load is recommended.

In their book, *Principles of Pavement Design*, Yoder and Witczak present a method for determining the load transferred by the critical dowel. In determining the load transferred by the critical dowel, Yoder and Witczak assumed that the deflection under a corner load would be greater than the deflection of the interior slab due to the same applied load. Thus, the corner dowel would be the critical dowel for edge loads (*5*). The load transferred by the critical dowel is given in Equation (5):

$$P_t = \frac{\text{Design Load Transfer}}{\text{Number of Effective Dowels}}$$

DESIGN EXAMPLE

Consider a 40 KN (9000 lbf) wheel load applied to a 250 mm (10 in.) thick concrete pavement slab with a compressive strength of 48 MPa (7000 psi). The pavement slab rests on a subbase having a modulus of subgrade reaction equal to 27 MPa/m (100 pci). Assuming a joint width of 6 mm (0.25 in.), determine the required spacing and diameter for GFRP dowels with the following properties:

modulus of elasticity = 41 GPa (6 x 10^6 psi)
shear modulus = 3.3 GPa (476,000 psi)
dowel bar length = 460 mm (18 in.)

Solution

1. Determine the Load Transferred by the Critical Dowel

$$\ell_r = \sqrt[4]{\frac{E_c h^3}{12(1-\mu^2)k}} = \sqrt[4]{\frac{(32.9)0.250^3}{12(1-0.2^2)0.027}}$$

= 1.134 m (45 in.)

The load transferred by the critical dowel is maximum when the wheel load is positioned directly over the dowel. For this loading condition and with the standard 300 mm (12 in.) spacing of dowels, 2.4 dowels are effectively active in transferring the load as shown in Figure 3. (The number of effective dowels was arrived at using U.S. customary units.)

Design load transfer = 0.45(40) = 18 KN (4045 lbf)

$$P_t = \frac{\text{Design Load Transfer}}{\text{Number of Effective Dowels}} = \frac{18}{2.4}$$

= 7.5 KN (1685 lbf)

2. Determine the Relative Deflection for a Joint Containing Steel Dowels

Assume the following properties for the steel dowels:

modulus of elasticity = 200 GPa (29 x 10^6 psi)
shear modulus = 77.5 GPa (11.24 x 10^6 psi)
dowel bar length = 460 mm (18 in.)

The recommended dowel bar diameter should be equal to 1/8 the slab thickness (*8*).

$$b = \frac{250}{8} = 31.25 \text{ mm}$$

Use a value of 31.75 mm for b since a 1.25 in. diameter bar will be used in the field.

$$I = \frac{\pi b^4}{64} = \frac{\pi(0.03175)^4}{64} = 5.0 \times 10^{-8} \text{ m}^4 \ (0.12 \text{ in}^4)$$

$$\beta = \sqrt[4]{\frac{Kb}{4EI}} = \sqrt[4]{\frac{(407)0.03175}{4(200)5.0\text{x}10^{-8}}} = 24 \text{ m}^{-1} \ (0.61 \text{ in}^{-1})$$

$$\beta L = \frac{24(0.46-0.006)}{2} = 5.45$$

Since βL is greater than 2, Equation (2) can be used to determine the deflection of the dowel relative to the concrete.

$$y_o = \frac{P_t}{4\beta^3 EI}(2+\beta z) =$$

$$\frac{0.0000075}{4(24)^3(200)5.0 \times 10^{-8}}\left(2+(24)0.006\right) = 2.91 \times$$

10^{-5} m (0.0011 in.)

$$\delta = \frac{\lambda P_t z}{AG} = \frac{(10/9)0.0000075(0.006)}{7.92 \times 10^{-4}(77.5)} =$$

8.15 x 10^{-7} m (3.21 x 10^{-5} in.)

$\Delta = 2y_o + \delta = 2(0.0291) + 0.000815 = 0.06$ mm (0.0024 in)

3. Determine the Relative Deflection for a Joint Containing GFRP Dowels

b = 31.75 mm (1.25 in.)

$$I = \frac{\pi b^4}{64} = \frac{\pi(0.03175)^4}{64} = 5.0 \text{ x } 10^{-8} \text{ m}^4 \text{ } (0.12 \text{ in}^4)$$

$$\beta = \sqrt[4]{\frac{Kb}{4EI}} = \sqrt[4]{\frac{(407)0.03175}{4(41)5.0\text{x}10^{-8}}} = 35 \text{ m}^{-1} \text{ } (0.90 \text{ in}^{-1})$$

$$\beta L = \frac{35(0.46 - 0.006)}{2} = 7.95$$

Since βL is greater than 2, Equation (2) can be used to determine the deflection of the dowel relative to the concrete.

$$y_o = \frac{P_t}{4\beta^3 EI}\left(2 + \beta z\right) =$$

$$\frac{0.0000075}{4(35)^3(41)5.0 \text{ x } 10^{-8}}\left(2 + (35)0.006\right) =$$

$4.71 \text{ x } 10^{-5}$ m (0.0018 in.)

$$\delta = \frac{\lambda P_t z}{AG} = \frac{(10/9)0.0000075(0.006)}{7.92 \text{ x } 10^{-4}(3.3)} =$$

$1.91 \text{ x } 10^{-5}$ m (0.0008 in.)

$\Delta = 2y_o + \delta = 2(0.0471) + 0.0191 = 0.1133$ mm (0.0044 in)

4. Determine the Required Diameter and Spacing for GFRP Dowels

Steps 1 and 3 were repeated for various diameters and spacings. The results of these calculations are presented in Table 1. The correct diameter and spacing for GFRP dowels is that which results in a relative deflection of approximately 0.06 mm (0.0025 in.). Table 2 lists the various combinations of diameter and spacing that work for this particular problem. Although this procedure typically results in material stresses within permissible limits, the authors recommend that the bearing stress in the concrete along with the flexural and shear stresses within the critical dowel be checked. The authors refer the reader to Friberg's paper, "Design of Dowels in Transverse Joints of Concrete Pavements," for equations to determine maximum bearing stress in the concrete and maximum shear and moment within the dowel (*2*).

CONCLUSIONS AND RECOMMENDATIONS

Typically, bearing stress in the concrete is the controlling factor in the design of doweled joints. If current design procedures were followed in determining the diameter and spacing for GFRP dowels, the resulting deflection of the dowel relative to the concrete would have been approximately 1.6 times that of an equivalent joint containing steel dowels. Since the bearing stress in the concrete is directly proportional to the deflection of the dowel relative to the concrete (Equation [6]), the concrete bearing stress also would have increased by a factor of 1.6:

$\delta_b = Ky_o$

TABLE 1 Relative Joint Deflections (mm)

	Dowel Bar Spacing (mm)			
Dowel Bar Diameter (mm)	300	250	200	150
31.75	0.11	0.10	0.08	0.06
38.10	0.09	0.07	0.06	0.05
44.45	0.06	0.05	0.04	0.03

* 1 in. = 25.4 mm

TABLE 2 Solution to Design Example

Dowel Bar Diameter (mm)	Dowel Bar Spacing (mm)
31.75	150
38.10	200
44.45	300

* 1 in. = 25.4 mm

In order to keep the bearing stresses in the concrete comparable to those for a steel doweled joint, the dowel bar diameter must be increased, spacing decreased, or a combination of both. Additional experimental evidence is needed to verify these predictions. Currently, Iowa State University is undergoing experimental investigations in the laboratory and field to verify these somewhat theoretical postulations mentioned in this paper.

GFRP dowel bars appear to be a feasible solution to the deterioration of the transverse joints of highway pavement slabs as long as the diameter of the dowel is increased, spacing decreased, or a combination of both. This adjustment is necessary in order to keep the deflection of the joint and thus the bearing stresses in the concrete equivalent to those experienced by their steel counterparts.

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Evaluation of Bonding Between Ultra-Thin Portland Cement Concrete and Asphaltic Concrete

JAMES K. CABLE AND JOHN M. HART

Ultra-thin whitetopping (UTW) has evolved as a viable rehabilitation technique for deteriorated asphalt cement concrete (ACC) pavements. Numerous UTW projects have enabled researchers to identify key elements responsible for the successful performance of UTW. They include foundation support, interface bonding condition, portland cement concrete (PCC) thickness, fiber reinforcement usage, and joint spacing. Interface bonding condition is the most important of these elements because it enables the pavement to act as a composite structure; thus, reducing interface stresses and allowing an ultra-thin PCC overlay to perform adequately. The impact that external variables on the elements and the interaction between elements in UTW performance has not been thoroughly investigated. The objective of HR-559 by the Iowa DOT and ISU/CCE was to investigate the interface bonding condition between an ultra-thin PCC overlay and an ACC base over time, considering ACC surface preparation, PCC thickness, fiber reinforcement usage, and joint spacing variables. The goal of identifying potential debonding between the layers of pavement was researched in laboratory testing using instrumented flexural test beams and mechanical loading machines. In full scale field-testing interface strains at the PCC/ACC interface were measured four time each year and the falling weight deflectometer deflection responses were measured at some 35 locations annually along a 7.2 mile Iowa Department of Transportation UTW project HR-559. This paper reflects on the results of the field deflection testing portion of that work. Key words: ultra-thin, overlays, rehabilitation, whitetopping.

INTRODUCTION

The use of the ultra-thin portland cement concrete pavement overlay for rehabilitation of asphaltic concrete pavements has grown rapidly since the first test was conducted in Kentucky at the entrance to a landfill. Questions still remain regarding the development of a design procedure and guidelines on when to apply such a treatment. The Iowa Highway 21 project in Iowa County, constructed in 1994, was a way of evaluating the variables of base preparation, overlay thickness, joint spacing and the use of synthetic fiber reinforcement on an in service pavement site. Base preparations included milling, brooming, and cold in place recycling. Depths of concrete included 2, 4, and 6 inches with joint spacings of 2x2, 4x4, 6x6, 12x12, and 12x15 feet.

PROJECT GOALS

A key to the successful performance of the ultra-thin pavement overlay is the development and retention of bond between the portland cement concrete and asphaltic concrete. Much of the literature makes estimates of the bond strength required at the interface, but it has not been successfully measured in the field. In this project the research sought to determine the bond strength to maintain a composite section through three methods. The methods included the use of direct shear laboratory testing of cores obtained from the finished pavement, monitoring of static strains at the interface over time and annual deflection measurements at selected locations.

DATA COLLECTION

A total of 53 cores were extracted from eight separate test section locations in Spring 1997 by the representatives of the research staff and the Iowa DOT. Each of the sections used in this effort were those of the two inch design depth. A total of three cores were extracted in the interior corner of slabs in the outer wheel path and three more were taken in the center of slabs in the same wheel path area. In two sections additional cores were extracted in cracked or miss aligned joint areas. Care was taken not to break the bond at the interface due to the twisting action of the drill during extraction. This is a limitation in this type of testing, but it was relatively successful in this case. In the cases were the core separated in the asphalt, it occurred at a point some 1-3 inches below the interface. This may have represented the asphalt layer construction interface. It did provide a thick enough layer of asphaltic concrete on the portland cement concrete to be tested in the Iowa DOT Shear Testing Device.

The second method of bond strength detection has been the monitoring of strain gages mounted vertically 1 inch into the overlay on a metal post anchored in the asphalt and in the corner of a overlay slab. Two such posts were located in the corner of some 35 separate slabs. In each case the post is located to allow bending in the longitudinal slab direction and the other post to measure transverse slab movement. The gages were monitored multiple times per day for the first two weeks after construction and then quarterly to date. This part of the work is ongoing.

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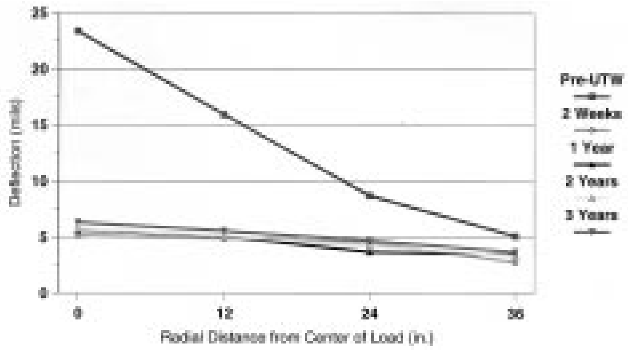

FIGURE 1 Deflection basins for station 2374+50.

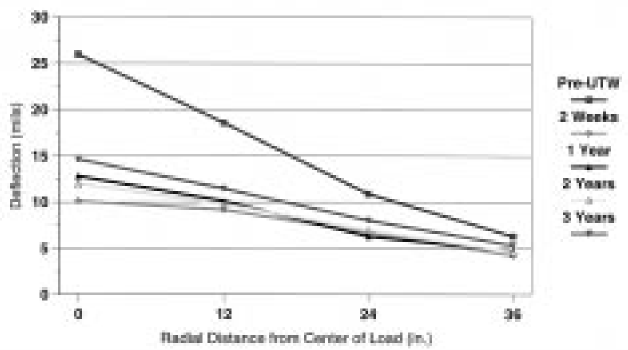

FIGURE 2 Deflection basins for station 2391+50.

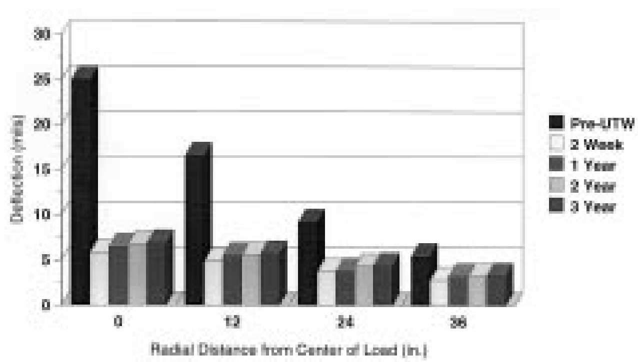

FIGURE 3 Combined average deflections for each testing. period.

The third method of detecting relative bonding of the layers employed the use of the FWD. In this case the unit owned by ERES Consultants of Champaign, Illinois has been employed on an annual basis to test at the midslab and transverse joint locations in the outer wheel path at each of the strain gage locations. Testing is accomplished during August of each year, early in the day to reduce the chance of joint lockup during testing. In this test a load of known magnitude representing a 9,000 pound wheel load is dropped on a bearing pad imparting a load into the slab. The response or deflection is measured at a point under and known distances in front of the loading pad. Testing began with measurement of the existing asphaltic concrete surface prior to the overlay construction and has continued to date.

ANALYSIS

In the case of the direct shear test, the core is loaded into a ring to maintain its shape. It is mounted in a vertical position such that the interface of the two paving materials is at the bottom of the ring. A plate is moved horizontally against the asphalt layer to shear it from the concrete surface. The amount of force required to accomplish the asphalt removal is recorded as the resisting force or shear strength at the interface.

In the case of the FWD deflection data, ERES Consultants Inc. staff used a computer program was used to back calculate the modulus of elasticity for the portland cement and asphaltic concrete layers, the cement treated base, aggregate treated subbase and the subgrade. The deflection data gathered by the FWD each year formed the data set for analysis. Their software can only assume a fully bonded or unbonded interface condition. Modulus seed values and acceptable modulus ranges are established for each material layer based on previous material knowledge. Modulus values obtained from assuming the bond, no bond condition were compared for each of the layers at each test location. Deflection basins showing a difference of greater than 5% were considered suspects for debonding. The results were then compared to visual distress survey information collected on a quarterly basis by the research staff.

In a separate analysis of the raw deflection data provided by ERES, the ISU research staff reviewed the raw deflection basin values over time and for various depths of overlay. Two of the typical plots are shown in Figure 1 for the three inch overlay depth and in Figure 2 for a seven inch overlay depths. These depths represent the depth at one edge of the pavement after overlay construction and exceed the design depths. The design depths were established at the highest point in the asphalt cross section prior to overlay. The pre-UTW line in each graph represents the deflection basin for the asphalt surface prior to overlay. Each of the other lines on the graph represents a deflection basin at this particular station at a given time after construction. Note that the lines for the 7.3 inch pavement basin measured after overlay construction remain relatively constant over time and flat in shape. The overall strength of the concrete is making this section act at as a concrete pavement with an asphalt base only. In the case of the 3.0 inch pavement, the lines have a steeper slope and the deflections at all locations away from the load are increasing over time. If this trend continues, we should be able to predict when it will approach the line representing the asphalt deflections prior to overlay. This should represent a time when the pavement is flexible or the slabs have

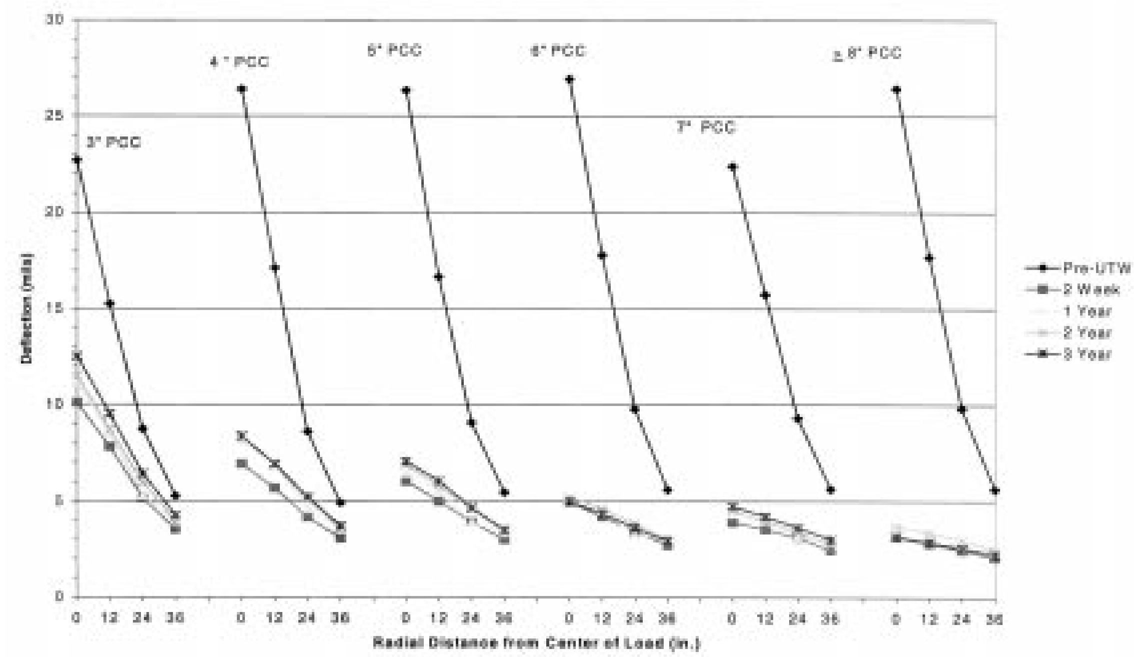

FIGURE 4 Combined average deflection basins of various PCC thicknesses.

lost their interlock and/or bond with the underlying surface of asphalt.

Figure 3 represents the average deflections for the entire population of deflection basins for all depths of overlay. In this graph, the preoverlay deflections represent the asphalt surface response to loading prior to the overlay. Note that there is a small increase in deflections over time in all overlay depths which is considered normal due to the surface wear due to traffic loadings and the effect of the environment.

RESULTS

The direct shear tests provided a wide range of shear values from 35 to 162 psi. This has been typical of values obtained from this type of testing on other Iowa DOT and national whitetopping projects of these depths. The values tend to indicate that the highest average shear values were obtained in the milled areas of base preparation with much lower values being obtained in the patch and cold in place recycle base preparations for the two inch overlay.

Summaries of the average deflection basins for the various overlay depths are shown in Figure 4. The nominal or edge thickness of the overlay is identified above each of the graphs. The deflection basins from all sections exhibiting that edge depth were averaged to provide the data for the after overlay graphs. Note that the graphs for depths of greater than five inches little or no increase in deflection over time or distance from the load location. On the other hand, the depths of three to five inches show increases in deflection over time under the load and a sharper slope between the sensor points. If this trend continues over time, the maximum deflection at any of the sensor points and the slope of the basin should approximate that of the asphalt and represent a loss of bond and free movement of the concrete overlay slabs.

Similar relationship between deflection basin information and time can be seen in Figure 5. In this case a specific three inch depth section and a four inch section have been graphed. An additional basin has been added in each case that represents a special deflection taken in an area of the test section that was suspected of being debonded. Potential debonding was identified by the use of a sounding bar on the surface of the pavement. In the first case a small number of sections have lost bond adjacent to the deflection test location. The deflection basin falls in the range of other bonded test slabs in the test section and may be partially bonded.

In the case of the four inch depth section large numbers of corner cracks are indicating some loss of support in this test section. Here the suspect deflection basin falls very close to the asphalt deflection basin. This may account for the large number of interior corner cracks. Debonding may be occurring at this location and not in the outer two thirds of the slabs in the row along the edge of the pavement. Loss of section in the two inch case only happened after the outer edge row of slabs lost bond.

CONCLUSIONS

Further analysis of the direct shear test results is under way to determine if any construction variables may have effected the magnitude and variability of the values. Those results may be an early indicator of the level of bond achieved at construction and could be used to measure the magnitude of the loss over time and the location of the weakest point in the composite section depth.

The use of the measured pavement depths and the deflection basin changes over time may provide an answer on the expected

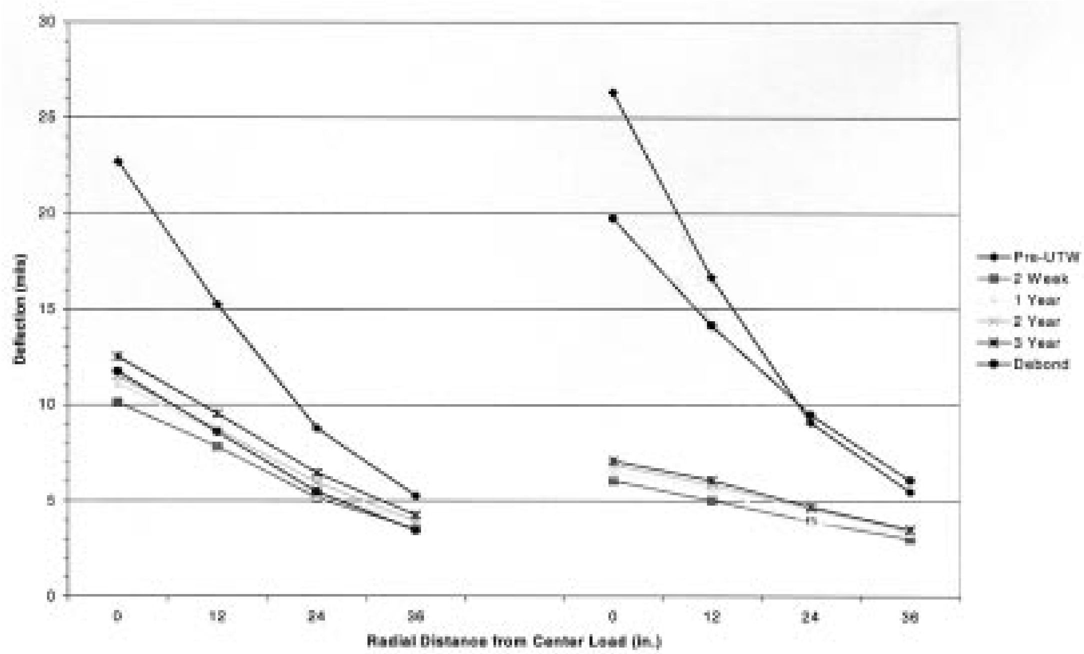

FIGURE 5 Suspected debonding deflection basins compared to combined average deflection basins of similar PCC thickness.

life of the UTW. It may also provide an indication of the effect of various surface preparation treatments on the performance of the UTW.

The use of the deflection basin information and the back calculation of layer moduli by various means as an indication of the level of bonding is still in an experimental stage. There are indications that this type of analysis will direct the engineer to suspect locations. On this project, this method has not provide that level of detail.

ACKNOWLEDGMENTS

The researchers want to acknowledge the assistance provided by the staff from ERES Consultants Inc. and Dr. Brian Coree of ISU/CCE in the evaluation of deflection analysis procedures and results.

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Friction as a Tool for Winter Maintenance

WILFRID A. NIXON

Keeping roads clear of snow and ice during the winter season requires a considerable expenditure of resources. Studies suggest that upwards of $2 Billion is spent on direct winter maintenance activities annually, and indirect costs could be a factor of ten greater. Accordingly, it is important that winter maintenance activities be conducted as efficiently as possible. An important step toward efficiency is the development of quantitative measures of the state of the road surface. Such measures would allow determination of the level of activity required to bring the road surface to a safe condition. One such measure is road surface temperature. Measurements of road surface temperatures have been conducted for at least twenty years by means of Road Weather Information Systems (RWIS). More recently, truck mounted sensors have been tested to determine their effectiveness. While these must still be considered experimental, their use has been enthusiastically greeted by some winter maintenance personnel, as providing real time on-the-spot information that can be of considerable use to operators. Unfortunately, temperature alone does not tell the whole story of the condition of a road surface. A key component is the road surface friction. However, before friction measurements can be used as an objective, quantitative measure of road surface condition, a number of issues have to be addressed. The purpose of this paper is to raise and address some of these critical issues.

INTRODUCTION

Winter weather poses a significant hazard to road transportation for many parts of the United States. Keeping roads clear of snow and ice is a significant challenge that has been estimated to cost more than $2 Billion per year. While this cost is significant, the service provided is critical from several aspects (*1*). Roads must be cleared of ice and snow rapidly and efficiently both to provide a safe road surface for the driving public, and to ensure timely delivery of goods that are carried by road. This latter point is growing in importance as "just-in-time" business practice becomes the norm (*2*).

Indeed, far from a desire to reduce levels of service to obtain savings, the trend in recent years from road users has been to demand higher levels of service. Thus any reductions in maintenance costs must come from more efficient operations. One way in which such savings might be obtained is through using new technology to allow maintenance crews to "work smarter." A number of new technologies have grown in prominence in the U.S. winter maintenance community since the Strategic Highway Research Program was conducted, including anti-icing, RWIS, and improved snow fence design. Other new technologies are being considered but are not yet in operational use. One such technology is the use of friction measuring devices as a real time measure of road surface condition.

Friction measuring devices have been used for a number of years to measure the conditions of pavements under wet and dry conditions. There are a variety of different devices, and the readings given by these devices can be related by means of the PIARC standard curve (*3*). However, no such standard exists for friction devices used on snow or ice covered pavements. Indeed, there are relatively few data available to indicate what friction levels cause problems for drivers under winter conditions, and how friction levels change as a storm progresses, and as the road is subsequently treated. Obviously such information must be developed prior to friction measuring devices can be used operationally. However, discussion of such research uses of friction measuring devices is beyond the scope of this paper (though see [*4*], for a discussion of some of these issues).

The purpose of this paper is to consider how friction measuring devices might be used operationally, and to define the conditions under which such use would be beneficial. The paper will first consider how the devices might be used, and will then discuss means by which the costs and benefits of these techniques can be assessed.

OPERATIONAL USES OF FRICTION MEASURING DEVICES

In order to be an effective tool for winter maintenance activities, a friction measuring device must provide information that allows decisions about winter maintenance activities to be made. This means that such tools must bring about a change in how winter maintenance is currently performed. If such a change does not occur then the tool cannot bring any new benefits to the process of winter maintenance. This is an important factor in determining how effective friction devices can be.

Since friction devices are not in full operational deployment at present in any winter maintenance operation at any location worldwide, it is difficult to say exactly how friction measuring devices will be used. However, present discussions seem to indicate they will be used in one of three ways: As a measure of quality; as a source of road user information; and as a means of controlling chemical application.

Friction Devices As A Measure Of Quality

The first envisaged use of friction measuring devices would be to measure road surface friction as a measure of quality. Friction devices would be mounted on a few vehicles (for example, supervisors' trucks). This usage may become especially prevalent in Finland (Anttalainen, *Personal Communication*, 1998) as a method of

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ensuring that contractors have performed an adequate job of winter maintenance over a given stretch of highway. This has the benefit of providing a direct and immediate measure of road surface condition, that can then be compared with desired conditions, and appropriate action taken if required standards are not met.

For this sort of deployment, direct measurement of savings will be hard to obtain, but a consistent standard of quality will be attained with such devices. Additionally, no other method of measuring quality of service seems likely (at this time) to offer such a consistent and repeatable measure. However, it should be noted that at present there is little knowledge about the relationship between friction levels and safe driving conditions (*4*). For this usage to become effective, considerable research is required to link road friction measurements with perceived and actual road conditions.

Friction Devices As A Source Of Road User Information

The second potential use of friction devices is as a means to provide information and/or warnings to road users about road sections in which friction is particularly low. This use could be achieved by mounting friction measuring devices on a few trucks within a fleet, and having these trucks continuously monitoring the road network for areas of low friction. If such areas are found, the information can be relayed (probably by automatic, electronic means) to a central location, from which appropriate warnings can be issued. The warnings may be issued through variable message signs, or via a graphical representation of the road system (perhaps on the web or at rest areas). Most likely, the information would be included with issued weather information from RWIS sites.

One concern with this use of friction devices is ensuring the timeliness of the information. Typically trucks take three to four hours (or more) to make one circuit of a route. Thus, if all roads within an area were to be covered then information would be up to four hours old, if all trucks were equipped with the measuring devices. If only a few trucks or supervisors vehicles have the devices, then data may be even more dated. Most likely, such a usage would be confined to limited sections of road, for which low friction is perceived as a major problem.

Again, for this type of device usage, direct analysis of benefits and costs is difficult to perform. This form of usage results in no direct savings, and considerable additional costs (extra vehicles out during a storm, and new equipment needed for those vehicles, along with other infrastructure such as variable message signs). There may however be significant indirect savings especially in regard to reduced accidents. Use of such a system would require that sites be chosen for which accident rates in low friction conditions are high. It should be stressed that at present no such systems are envisaged for deployment, and while all the parts necessary to make such a system work exist, the concept has not been tested. Such field testing is clearly necessary before a full analysis of benefits can be made.

Friction Devices As A Means Of Controlling Chemical Application

The third potential use of friction devices is as a controlling input for chemical delivery systems on board snow plows. In essence, the friction device would measure the road surface friction and based on the value found (and other inputs) would determine how much de-icing chemical should be applied to the road surface to bring about a suitable friction level in a desired time frame.

Preliminary measurements of friction on the road surface under winter conditions (e.g. [*5*]) indicate that friction can vary considerably over short distances. In one run that they present, measured under conditions of slush and wet pavement, friction values ranged between 0.9 as a high and below 0.2 as a low, over a distance of 20 km (12.5 miles). This variability may well be due to variable road conditions, and highlights the possible usefulness of such information. The authors suggest that the use of friction limits to determine salt application could limit the quantity of salt applied. In this case, using their suggested standard (heavy salt for values below 0.4, light salt for values below 0.6 and above 0.4) heavy salting would have been required for 8 km, light salting for a further 6 km, and no salting for 6 km (note: these are calculated from [*5*], by the author of this study. The authors of [*5*] did not report such calculations). Current practice would require the whole 20 km segment to receive heavy salting. Assuming heavy salting to be 110 kg/lane kilometer (400 lb/lane mile), and light salting to be 55 kg/lane kilometer (200 lb/lane mile), a saving of about 990 kg of salt (for each lane) would have resulted from the current standard of 2200 kg per lane through the segment. This represents a 45% reduction in salt use, which is a considerable saving. However, it is not clear that such high levels of savings would always be attainable. Nor, as the authors note, are the levels of friction for different chemical treatments based on any valid data - they are merely suggested as seeming suitable by the authors.

Nonetheless, such preliminary studies indicate what might well be the most promising use of such devices. Reducing salt usage is of benefit not only for reducing cost, but also for reducing damage to pavement (due to corrosion of re-bar and subsequent spalling of concrete). However, for such an approach to be maximally effective, it would have eventually to be applied fleet wide, rather than to just one or two trucks. Further, there is considerable work required to determine what friction levels require what amount of salt under which conditions. Finally, the usefulness of such an approach is limited (but not totally negated) if an anti-icing rather than de-icing approach is used.

As indicated above, the value of such uses of friction devices is hard to quantify at present, because such devices are not yet fully deployed. Nonetheless, a simplified cost benefit analysis may be performed that gives a preliminary indication of what such devices should cost to break even. Such a first level analysis is presented below.

PRELIMINARY COST BENEFIT ANALYSIS

A simple cost benefit analysis of the use of friction devices to control chemical applications is given in this section, but it should be noted that this analysis makes a number of assumptions that are not currently justifiable. These assumptions are identified explicitly in the following:

Assume that the use of friction devices results in a reduction in the use of chemical de-icers of R%. This reduction is from a base level of de-icer usage (i.e. the amount of de-icer used now, without friction devices) of D tons per year, at a cost of \$C per ton. Thus

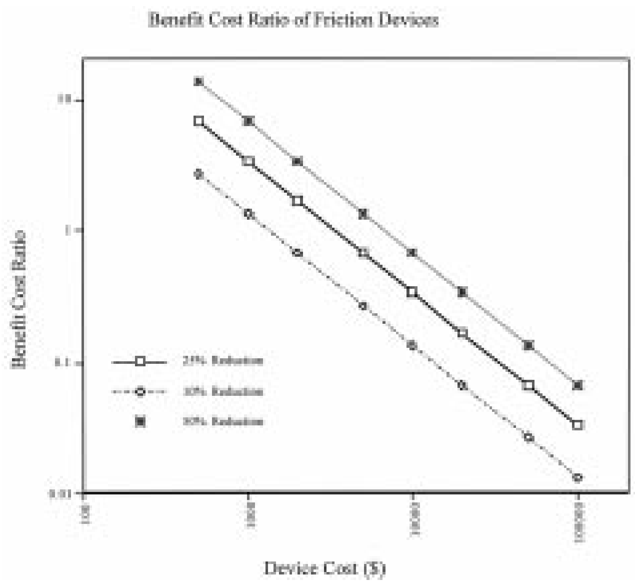

FIGURE 1 Benefit-cost analysis for friction devices.

the potential savings of friction devices ($S per year) can be expressed as:

$$S=\left(\frac{R}{100}\right)DC$$

Thus if 70,000 tons of salt per year are used now, at a cost of $25 per ton, and a full fleet of friction devices in use results in a reduction of salt usage of 25%, then annual savings are $437,500. Note that the assumption of a reduction in salt usage of 25% is *highly* speculative. However, this represents the direct material benefits of using friction devices. There may also be labor cost savings, and there will likely be indirect savings, due to less salt damage of the pavement, and fewer accidents and delays (because of a higher level of service). At present, these additional savings are not considered.

Of course, the friction devices come with a cost. If each device costs $M to purchase and install, and there are F vehicles in the fleet, then the total cost of installing devices in the fleet ($P) is given simply as $MF. Typically, however, such costs are annualized over the lifetime (n years) of the device, by assuming a percentage cost of money of i% per year. This is a standard equation from economic analysis, and it gives the annual cost ($A) as:

$$A=P\left(\frac{i(1+i)^n}{(1+i)^n-1}\right)$$

Thus, if a ten year life is assumed for the equipment, with a percentage cost of money of 5%, a fleet size of 1,000 vehicles and a device cost of $1,000 per device, the annualized cost (A) is $129,500. The benefit-cost ratio (B) of the installation is then calculated as:

$$B=\frac{S}{A}$$

The worked example above gives a ratio of 3.38. That is, every dollar spent on friction devices would result in somewhat more than three dollars in savings. However, note that costs assumed a unit cost of $1,000 per device installed, a very low figure given current costs, and that no account was taken of training costs for use of the new equipment. Nonetheless, the example does show a simple methodology for considering the benefits of such a system. It also indicates the sensitivity of such analyses to a variety of different factors.

This sensitivity is made more explicit in Figure 1. This shows the benefit-cost ratio as a function of the initial device cost for three different levels of salt reduction (10%, 25% and 50%). The plot is linear in log-log space, and shows that if a 25% reduction in salt usage is achieved, then break-even (a benefit-cost ratio of one) requires an initial cost of around $3,400. As indicated above, there are many assumptions in this analysis, and before any great faith can be placed in such an analysis, considerable research is required to clarify some of the assumptions.

CONCLUSIONS

The following conclusions can be drawn from this study.

1. Friction devices are being considered for at least three different modes of use in winter maintenance: as a measure of quality, as a source of road user information, and as a means of controlling chemical application.
2. All three of these uses are not yet operational, and considerable information is required before their success or otherwise in such uses can be evaluated.
3. A preliminary cost benefit analysis for friction devices as controls for chemical application has been conducted, but too much uncertainty exists at present with regard to potential savings for the results to be of any more than academic interest. Nonetheless, a methodology for conducting such studies in the light of better data has been established.

ACKNOWLEDGMENTS

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The assistance of Mr. Lee Smithson throughout this project in providing both guidance and wisdom is gratefully acknowledged. Mr. Tapio Raukola of the Finnish Road Administration was most helpful in providing information about friction measurements in Scandinavian Countries.

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

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A New Paradigm for Winter Maintenance Decisions

DUANE E. SMITH

A universal challenge facing highway agencies and state departments of transportation (DOTs) today is simultaneously increasing productivity, quality, and environmental sensitivity while maintaining a constant or improved level of service on roads. These challenges are of major importance to three-quarters of the states' DOTs and local governments who must face the perils of winter as they strive to provide uninterrupted mobility to the road users. Snow and ice control procedures could benefit greatly from improvements in technology applications and equipment operator efficiency. Sensors and automated attachment controls could improve decision strategies for these agencies. Utilizing sensors that record roadway surface temperatures, computers may determine optimal timing and application rates of chemicals and abrasives. Automatic vehicle location systems can track the progress of maintenance vehicles and record winter maintenance operations. The paper will show a sample of the data plots that are transmitted from the vehicles. Initially, the data is referenced to GPS bearing or to the time from the beginning of the maintenance run. CTRE has adjusted the plots so they are referenced to milepost. This paper will focus on the critical factor in the winter maintenance decision process, pavement temperature, and discuss how accurate data presentation can alter the existing process. A critical ingredient to be recognized, when existing procedures or practices are evaluated for change, is the acceptance by the workers. This paper will provide the results of phone interviews with the vehicle operators to ascertain their evaluation of equipment performance and reaction to the advanced technologies. Key words: anti-icing, GPS, pavement, RWIS, temperature.

INTRODUCTION

A universal challenge facing highway agencies today and state departments of transportation (DOTs) is simultaneously increasing productivity, quality, and environmental sensitivity, while maintaining a constant or improved level of service on roads. These challenges are of major importance to three-quarters of the states' DOTs, who must face the perils of winter as they strive to provide uninterrupted mobility to the road user. Snow and ice control during winter storms includes highly complex tasks and long, stress-filled hours both for equipment operators and for their supervisors. Continued cutbacks in DOT staffs dictates that one equipment operator must now be able to drive a snow plow truck and manage all of its ancillary equipment. These staff reductions come at a time when road users require greater mobility and an increased level of service for winter driving. To address these issues, the concept highway maintenance vehicle project was undertaken by a consortium of three "snowbelt" state DOTs, Iowa, Michigan, and Minnesota, who have reputations for embracing innovation in highway maintenance management, maintenance operations practices, and research. The Center for Transportation Research and Education (CTRE), an Iowa State University center, provided support staff to the consortium. A key element of this project was the inclusion of private sector partners who brought many assets to the project, including staff with specialized expertise, business connections, manufacturing facilities, and the potential to participate in the funding and production of the vehicles.

Snow and ice control operations can benefit greatly from improvements in state-of-the-art on-board computer applications, enhanced safety systems, and improved equipment operator efficiency. Roadway surface temperatures may determine optimal timing and application rates of chemicals and abrasives. Automatic vehicle location systems can track the progress of single vehicles and fleets.

This paper will explore pavement temperature sensing devices that are used in conjunction with global positioning systems (GPS). Both of these technologies are included on the concept vehicles. The paper then presents the reactions of the equipment operators who were exposed to the advanced technologies during winter storm conditions.

PAVEMENT TEMPERATURE

According to the Transportation Research Board, "Demands on highway agencies for fast and effective deicing, however, sometimes results in indiscriminate salting. However, new developments in winter maintenance including deicer application techniques (e.g. salt prewetting), plowing and spreading equipment, and weather and roadway monitoring (e.g., pavement sensors) are making these priorities less confusing" (*1*).

Pavement temperature is the controlling item in the treatment of highways during winter storms (*2*). Using this fact, pavement temperature data may be used to customize the rates of material application and the type of material utilized to match road conditions. CTRE research recommends selecting a salt application rate using a curve adapted from "Smart Salting: A Winter Maintenance Strategy" provided by the Vermont Agency of Transportation (*3*). During the winter of 1993-1994, the Vermont Agency of Transporta-

Center for Transportation Research and Education, 2625 N. Loop Drive, Suite 2100, Ames, Iowa 50010-8615.

TABLE 1 Melting Capacity of Salt (*3*)

Temperature (°F)	Pounds of Ice Melted Per Pound of Salt
30	46.3
25	14.4
20	8.6
15	6.3
10	4.9
5	4.1
0	3.7

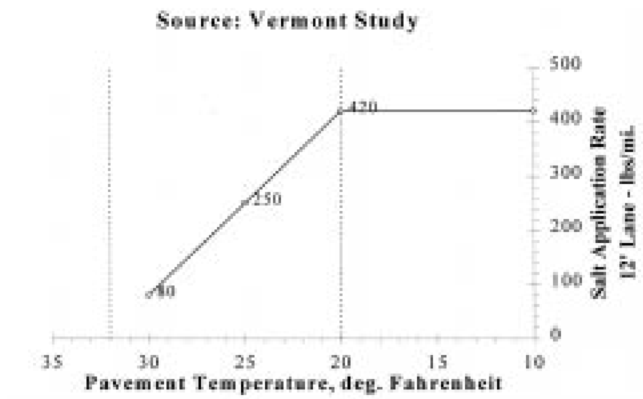

FIGURE 1 Vermont study recommended application rates (*3*).

FIGURE 2 The Iowa vehicle with temperature gauge.

tion (VAT) conducted a study and coordinated pavement temperature information with winter highway maintenance activities, resulting in an anti-icing and deicing strategy. Anti-icing is the application of liquid chemicals and materials early in the storm, or during plowing operations, that prevent the bonding of snow/ice to the road surface. By preventing the bond between snow/ice and the road surfaces the task of removing snow and ice is much easier. Estimates in Iowa indicate 50–60% reduction in the snow/ice removal effort when anti-icing procedures are utilized. De-icing is defined as the removal of snow/ice after the bond has formed. It is the procedure typically used in the past.

The Vermont study called for winter maintenance crews to do two things. First, determine pavement temperature before and during a storm, and second, determine salt application rates based upon the relationship between pavement temperature, melting capacity of salt, and the thickness of ice or snow on the pavement.

The Vermont Study generated a graph correlating recommended salt application rates with pavement temperatures. The Vermont Study identified an "economic salting range" which extends from 30°F to 20°F. The Iowa DOT estimates that 75 to 80 percent of their winter storms occur within this range.

Because the temperature of the pavement is such a critical variable in winter maintenance strategies, CTRE correlated the temperature values from the concept vehicles to another data source. All prototype vehicles were equipped with the same pavement and air temperature sensors. They have a road surface temperature range of -40° to 200° F and an air temperature range of -40° to 120° F. The sensors are accurate to within ±1% of full-scale or 1° F, whichever is greater. The response time is 1/10 second. The system is a passive infrared temperature indicator that uses infrared technology to read road surface energy and converts it to a temperature reading. The pavement sensor is mounted on the outside of the vehicle (typically on the driver side mirror) and reads the pavement temperature directly below the sensor.

To perform validity checks on temperature data, the following data was captured from the concept vehicle and from the Iowa Roadway Weather Information System (RWIS):

- Air temperature from the vehicle, stamped with time and GPS location
- Air temperature from nearby RWIS location stamped with time
- Pavement temperature from the vehicle, stamped with time and GPS location
- Pavement temperature from nearby RWIS location stamped with time.

The vehicle recorded temperature data nearly regularly and stored this data on the Rockwell PlowMaster. The data was transferred to CTRE and converted to a d-Base format for analysis in Microsoft Excel. CTRE then generated charts of pavement and air temperature readings. The initial data is referenced by GPS heading or by time from when the maintenance run began. Figure 3 shows the temperature plot versus time and later on we will see the time reference converted to milepost.

The data from the vehicle temperature sensors needed to be correlated to a known base data. It seemed that the most accurate and easiest way to do this was to compare the concept vehicle temperature data to data taken from RWIS located adjacent to the roadway.

The nearest RWIS site in Iowa was on I-80/I-35, on the north side of Des Moines, near mile marker 135. The location of this site is approximately two miles south of the Iowa concept vehicle's route, on I-35. It was ideal for comparing RWIS temperature data to the concept vehicle's.

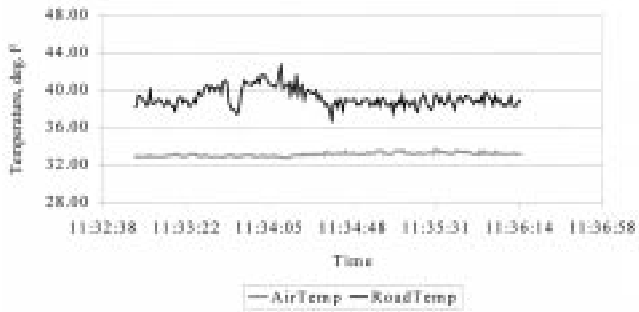

FIGURE 3 Temperature plot versus time.

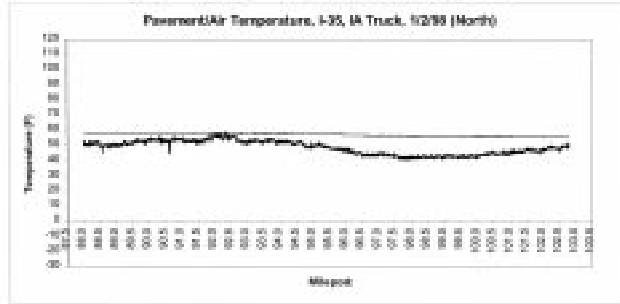

FIGURE 4 Temperature plot versus milepost.

TABLE 2 Temperature Sensor Comparison to RWIS

Date	Differences (PM:RWIS) Deg Diff Air	% Error Air	Deg Diff Pavement	% Error Pavement
1/12/98	1.10	4.99%	-16.63	51.96%
1/14/98	0.56	3.71%	-26.29	87.62%
	-1.94	9.70%	-17.22	57.40%
1/21/98	-3.42	2.15%	-11.72	37.82%
1/22/98	18.44	55.87%	-17.11	55.19%
3/2/98	0.547	1.89%	-10.538	27.02%
	0.006	0.02%	-11.050	28.33%
	-0.890	3.18%	-12.706	33.44%
	0.542	2.01%	-10.945	28.80%
	-0.584	1.95%	-9.258	24.36%
3/7/98	1.43	4.46%	-1.15	2.95%
	-0.67	2.03%	-8.14	21.42%
	-0.31	0.93%	-7.95	20.91%
3/8/98	-0.61	1.85%	-8.20	21.57%
	-1.18	3.58%	-8.55	22.49%
	-0.90	2.73%	-9.15	24.08%
	0.93	3.00%	-7.41	20.02%
3/9/98	9.28	71.38%	-18.26	52.18%
	2.39	17.08%	-16.89	49.67%

The temperature data was collected during the winter of 1997-98 and then compared to the RWIS data for the same dates and times.

After comparing the two sets of data, the air temperature was accurate with a percent error of 10.13%. There were two runs, however, that had extremely high percent errors of 55.87% on 1/22/98 and 71.38% on 3/9/98. Taking these two readings out, the average percent error for the remaining runs was 3.84%. The other run on 3/9/98 also had a high degree of error of 17.08%. So far, nothing of significance has pointed to the reason behind the high percent error on any of those runs.

The pavement temperature, however, had an average percent error of 35.12% over all runs recorded and compared. The pavement temperature depends on the pavement material, surface conditions, treatment material applied, and sun conditions. Because of this, the RWIS site was not the most representative base data to use for comparison because it represents a point location on the roadway and the concept vehicle provides a linear temperature measure.

GLOBAL POSITIONING SYSTEM (GPS)

GPS uses a constellation of 24 satellites which orbit the Earth every 12 hours at an altitude of around 12,000 miles, arranged into six circular orbits inclined 55 degrees with respect to the Earth's equator. Their positions and orbits are always accurately known. Each satellite continuously transmits via a one-way radio communication channel the exact time. GPS antennas or receivers on the Earth use triangulation, with at least three GPS satellites, to establish a position. Each GPS receiver "listens" for the radio signal and calculates the elapsed time between radio signal transmission and reception. The GPS receiver then calculates the distance between the GPS satellite and receiver. More advanced GPS receivers can calculate vehicle speed using the difference in distance and elapsed time between two positions.

Since location data will be used for various functions of the concept vehicle, including pavement temperature plots, location of particularly icy spots on the road, location of material applications, etc, there is a need to compare GPS coordinates with baseline coordinate data supplied by the DOT. The concept vehicle established GPS locations, along I-35 in Iowa, from milepost 88 to 102. This was accomplished by stopping the vehicle at each milepost marker and recorded GPS coordinates. These coordinates were then compared to the officially published Iowa DOT milepost coordinates. CTRE corrected the concept vehicle coordinates to the Iowa DOT coordinates. This now allows the data coming from the concept vehicle to be reported by milepost. The following is an example plot of pavement temperature by milepost.

So far this paper has discussed the value of pre-treatment during winter storms and has presented salt application rates that are most economical. The paper has described how the maintenance concept vehicle can record the pavement temperatures and can locate these temperatures by milepost. But what do the people who used this technology think? The following section illustrates the positive response from the equipment operators.

RESPONSES FROM EQUIPMENT OPERATORS

The winter of 1997-1998 was an important evaluation period for the prototype vehicles, including their performance and identification of malfunctions while performing normal winter maintenance assignments. Each of the three prototype vehicles maintained and treated roads in Iowa, Minnesota, and Michigan. The prototype vehicle operators and mechanics had first-hand experience of vehicles performance and feedback from them was key in evaluating performance. They were an active part of the research team and participated in meetings and conference calls throughout the project.

Questionnaires and equipment performance log sheets were used to capture the reaction of the users to advanced technology applications. Interviews were conducted to determine if advanced technology has made the equipment operator's workload any easier, or if it has added to the job. The questions that were asked and a summary of the responses is provided.

1. "What element of the new technology worked the best?" The operators appreciated the user-friendliness of the PlowMaster computer. Equipment operators commented positively on the operation of the variable speed material applicators. With these tools, the equipment operators can set a prescribed amount at a given speed, and the material applicator compensates material application for changes in speed. One equipment operator termed the material applicator user variable and friendly. Although the material applicator is found on some other winter maintenance trucks, the equipment operators still appreciated the inclusion of the material applicator on the advanced technology vehicle.
2. "What element of the new technology worked the worst? Did this relatively poor performance have any negative impact on the operation of the other vehicle components?" Equipment operators faced continuous challenges with both the temperature sensors and the friction meter. At one point, the Iowa DOT reported the pavement temperature sensor as being off by as much as 30° F, prompting replacement of the sensor with a better functioning one. The Iowa and Minnesota DOTs reported problems with broken belts on the friction meter, in addition to problems associated with corrosion of the friction meter's parts. When equipment malfunctioned or failed, that particular piece of equipment was usually rendered out of service until the vehicle returned to its garage. However, even when the equipment malfunctioned, the drivers reported that they were still able to operate the truck at or above the same level of service with which they operated conventional snowplows. This fact is important and shows the advanced technology vehicle can still complete the basic assignment even when the technology is temporarily not available.
3. "Was the PlowMaster display easy to read while you were driving?" The PlowMaster screens required some learning but the operators admitted they experienced similar situations whenever they received a new piece of equipment. Equipment operators reported the screen dimness and brightness feature of the Rockwell PlowMaster display was relatively easy to read. During the day the operators would brighten the screen, and during the evening the operators would dim the screen. The only reported problem of reading the PlowMaster display was in direct sunlight (from the Minnesota DOT). The screens were designed to be logical and easy-to-follow. Equipment operators reported being able to quickly call up information reported by the PlowMaster computer.
4. "How did the added technology on the prototype vehicle affect your comfort and attention to the road, as compared with conventional maintenance trucks? Was the added technology a detriment or enhancement to the attention you could give the road?" The equipment operators reported the advanced technology helped them focus more of their attention on the road, especially when the equipment was functioning properly. The technology took tasks out of the hands of the equipment operators and allowed them to focus their attention where it was needed. Of key importance was the statement made from equipment operators at all three state DOTs concerning the periods when the equipment malfunctioned. The operators reported that during these periods they were able to operate the truck without a loss in productivity when comparing to conventional DOT snowplows. This fact states that the prototype trucks can function the same as conventional snow plows if there is a failure in the advanced technologies. After the initial time used to become familiar with the new technology, equipment operators were able to use the technology with relative ease, and efficiency higher than that with conventional snowplows.
5. "Any other problems you had with the truck while driving it?" Equipment operators from Iowa reported the present location of the material applicator requires them to stop the vehicle whenever they change the material applicator's settings.
6. "What suggestions for improvement do you have?" Iowa equipment operators suggested changing the placement of the material applicator controls and allow the operator to change the settings while the truck was moving.

All of the responses to these questions were positive and supportive. They also indicate that the equipment operators are looking into the future and presenting input for modifications and refinements to be made.

CONCLUSION

The acceptance of new technology applications by equipment operators, and others whose jobs are related to the highway maintenance vehicle, is critical to its success. The equipment operators embraced the new technologies and the basic reason was due to being involved in the development of the requirements and being involved throughout the development and implementation of the technologies. As a result of their acceptance, the concept vehicle can measure pavement temperature, locate the vehicle position by GPS, and provide reports by milepost.

ACKNOWLEDGMENT

The author wishes to thank the people at the Iowa, Michigan, and Minnesota DOTs who worked so hard to make this project a success. Without their full support the concept maintenance vehicle project would not be as successful as we see it today. The private sector partners were invaluable for providing the technologies, assembling them on the vehicles and then providing support during the initial stages of the project.

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The Measurement and Theory of Tire Friction on Contaminated Surfaces

JAMES C. WAMBOLD AND ARILD ANDRESEN

In the past five years there has been an International Experiment to Harmonize Friction Measurement by the World Road Association (PIARC) and within the past three years there have been at least four separate studies on winter friction, a five year joint winter runway program between NASA, FAA, Transport Canada (TC), the Canadian National Research Council (NRC), the Norwegian Civil Aviation (NCAA) and the French Civil Aviation Administration; a study by the Norwegian Road Administration with Norsemeter; a study by Minnesota DOT and the Concept Highway Maintenance Vehicle Study by the Iowa Center for Transportation. In addition to these studies there are standards under development: an International Friction Index (for wet pavements) and an International Runway Friction Index (for winter operation). This paper summarizes the results of these various studies. In the case of wet pavements we now know that the tire first determines the friction slip characteristics until the peak is reached and then beyond the peak the pavement's ability to drain the water determines the speed gradient. When the tire makes contact with the pavement the tire is the sacrificed part of the friction pair; however, on ice and snow the opposite is true and the ice or snow is the sacrificed part of the friction pair. Thus the peak friction that is developed depends on the shear strength of the sacrificed part. With these studies completed, the highway and aviation communities will be better able to measure friction on contaminated pavements.

INTRODUCTION

In search for a better understanding of braking friction processes, mathematical and graphical models that can describe and visualise the processes are useful. Engineered models are usually limited and often inadequate in their capabilities to capture the true, real world processes. We never accept the lesser models that simplify the real world, when they yield plausible results in the area of focus or application.

This paper looks at some existing models for longitudinal friction in the tire-pavement interaction and tries to incorporate parameters of influence found on winter surfaces. The area of interest encompasses all surface types and conditions, which are considered operational for aircraft ground movements. The models are developed in a context of defined surface classifications. Just as pavement friction models reflect the application to pavement as a base surface, we will look at friction models for ice based and snow based surfaces.

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Models Modification Requirements

One possible use of the model modifiers is to adjust an actual measurement to a standard condition with any modifier developed. For example, if a reference is standardized to represent values of friction at -10 degrees Celsius, the actual measurement can be adjusted from the actual temperature during a measurement to the reference temperature using a temperature modifier.

A tire configuration term is used to group the signatures in families per tire configuration. A tire configuration comprises make and type of tire (footprint, rubber compound, longitudinal stiffness), the inflation pressure used and the normal load used during braking operations. The brake actuator control technique is also considered part of the tire-configuration.

Modifications of the pavement friction models are studied as the new parameters and variables are introduced to cater to the sacrificial surface mechanism (hardness/ultimate shear strength), surface temperature, friction enhancing abrasives (sand, grit) and mechanisms such as rolling resistance, fluid planing and fluid drag. Contaminant compression is imbedded in the shear strength term and is considered for inclusion in a rolling resistance term, pending further investigation. These empirical modifications are good starting points for experimental analysis.

GENERAL TIRE-SURFACE FRICTION MODELS FOR PAVEMENT

The following is a brief review of some available tire-surface friction models using ground speed and slip speed as the independent variable. When the models are applied on experimental data, especially those obtained under the Joint Winter Runway Friction Measurement Program, actual tire configurations will be reflected in reference curves for the calibration and harmonisation of friction measurement devices. A single device or combination of several devices (called a virtual device) may be chosen as a Master Device or Prime Calibration Reference Device.

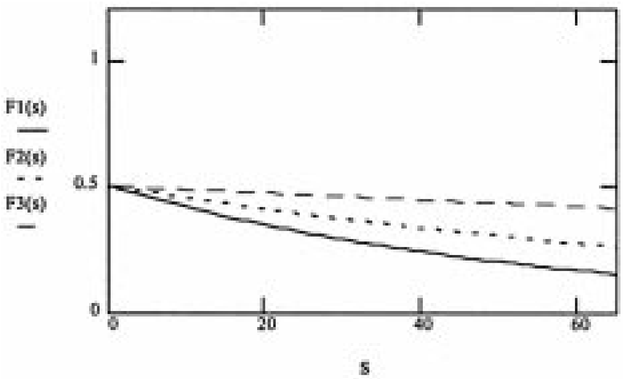

FIGURE 1 Effect of high and low speed constants S_{p1}=55, S_{p2}=100, S_{p3}=340. Lower curve is S_p=55.

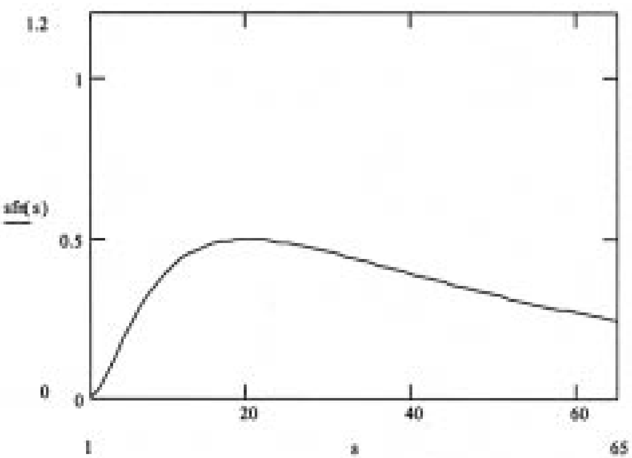

FIGURE 2 A sample friction curve generated with the Rado Model.

The Penn State Model

The Penn State Model is an exponential function with slip speed as independent variable. The model has been used for wet pavements to monitor macrotexture. The model is the basis for the PIARC Model used with the International Friction Index. The model is used here with a zero intercept constant, F_0, and a constant slip speed constant, S_p. Equation (1) is given below and shown graphically in Figure 1 with the following set of parameters:

s = 1 through 65 km/h, F_0 = 0.5, S_p = 55, 100, and 340 with Equation (1).

The speed constant governs the slope of the curve. A higher speed constant makes the curve more flat. The speed constant expresses the influence of macrotexture of the pavement. High macrotexture corresponds with a high speed constant. For the International Friction Index, it is derived from a texture measurement and used with the friction value at the harmonization slip speed of 60 km/h.

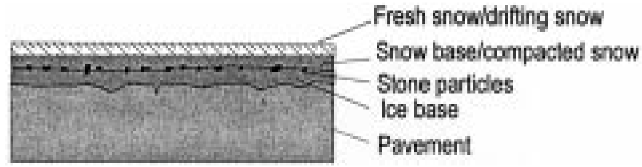

FIGURE 3 A cross section of a winter surface. The pavement can be found with any one or several of the indicated layers. All possibilities of the surface classification are not shown, for instance, wetness.

The Logarithmic Pavement Friction Model

The Logarithmic Pavement Friction Model (also called the Rado Model) is currently used with variable slip friction devices to report three friction variables. The model introduces a logarithmic ratio of slip speed vs. critical slip speed and a shape factor, C (originally designated $\hat{C}$). The critical slip speed (abscissa value), S_c, and peak friction value (ordinate value), P, fixes the location of the maximum friction value on the Cartesian graph of friction vs. slip speed and, therefore, governs the initial climb of the curve. We note that P and S_c fix the position of the maximum friction point.

The set of parameters used in the Rado Model, Equation (2), is s= 1 through 65 km/h, P= 0.5, S_c= 20 km/h, C= 1.4 and is shown in Figure 2.

$$\text{sfn(s)} := P \bullet \exp\left[-\frac{In\left(\frac{s}{S_c}\right)^2}{C^2}\right]$$

Note that the actual resulting curve shape depends on both C and S_c. It has been found that the three Rado Model parameters generally vary with measuring speed. We shall propose speed function for these parameters for winter surfaces in later sections.

COMPOSITE WINTER SURFACES

We introduce the composite surface classification depicted in Figure 3.

The braked wheel can displace all or a large part of a layer of slush, fresh snow or drifting snow that has a fluid powder character. This gives rise to contaminant displacement drag forces on the wheel and varying levels of fluid lift and fluid lubrication as some of the fluid contaminant gets trapped under the tire. Compression may occur with the trapped snow to build a thin layer of new snow base in the track of the wheel. When there is sand applied to the surface (likely before the snow base in Figure 3), it will interact with the tire and raise the friction force experienced. Even in cases without fresh snow/drifting snow, the snow base may be sufficiently soft for the tire to shear off snow crystals during braked wheel rolling and create small amounts of powder to sustain a partial planing condition.

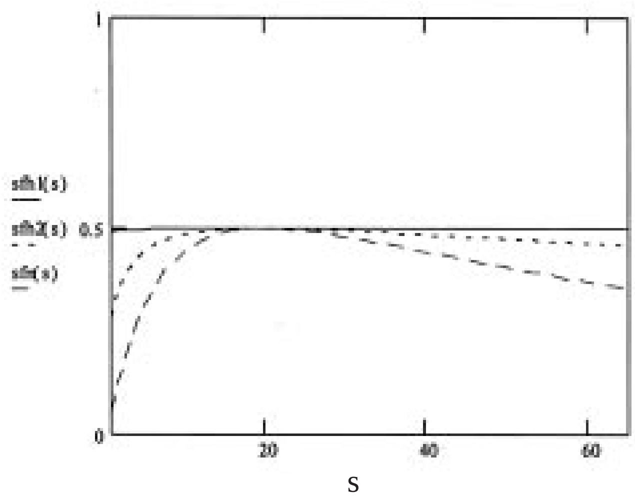

FIGURE 4 A hardness parameter applied to the Rado Model. The upper curve is softest with H=0.1, middle curve is H=0.5 and lowest curve H=1 (hard surface).

Except for clean or damp pavement and clean ice there is always a presence of partial planing. For hydroplaning (liquid water) the fluid dynamic lift part of planing does not contribute significant frictional forces. Only the remaining tire surface contact area is yields braking friction.

For snow planing the shear forces of a laminar or turbulent flow of snow powder at high speeds generates significant shear forces in the planing contact area.

THE MODIFIERS PROPOSED

Surface Shear Strength and Compressive Strength of Snow

For a snow base we need a parameter to express the sacrifice of the surface, as opposed to the pavement, where the sacrifice part of the friction pair is the tire. The onset of such a sacrifice, shearing off or crunching the snow, occurs when the demand for shear force exceeds the shear strength of the snow. The normal load may crunch the snow at local stress concentration points. Automotive type tires and friction tester tires have stress concentrations along the sidewalls. The crushing occurs when the compressive strength of the contaminant material is exceeded.

In the Rado Model C is related to Sp when 1.7<C<6 for wet pavements. In that range C expresses macrotexture influence for rigid surfaces. For winter contaminants the surface may behave as a hard, non-sacrificial surface, or as a loose, sacrificial surface.

The sacrificial surface should not be expected to show any strong relationship to macrotexture, when the ultimate shear strength has been exceeded and some material has been torn loose. Experiments then show friction forces versus speed as a flat curve even though there is no macrotexture. The interpretation of macrotexture with the Penn State Model is not valid for the sacrificial base surface like snow. The surface shear effect masks the macrotexture.

A parameter to reflect loss of coherence is therefore introduced. In the following, pavement friction models are amended on an empirical basis with a parameter for surface hardness, H. A rigid surface base has a value of H = 1' a soft surface base less than 1. C of the Rado Model is associated with winter-contaminated surface and is interpreted as $\left(\frac{C}{H}\right)^2$. The parameters for Figure 4 are P = 0.5, S_c = 20 km/h, C = 2, H = 0.1 and H = 0.5.

Equation (3):

$$sfh(s) := P \bullet \exp\left[-\frac{\left(In\left(\frac{s}{S_c}\right)\right)^2}{C^2 \bullet \frac{1}{H^2}}\right]$$

Contaminant Displacement Drag

The equation (Equation [4]) for contaminant displacement drag by the frontal area of the tire is generally:

$$F_{DRAG} = 0.5 \cdot C_D \cdot \rho \cdot A \cdot v^2$$

Since we will be tabulating parameters per surface class and tire configuration, we can simplify the equation (Equation [5])to

$$F_{DRAG} = k_{drag} \cdot v^2$$

Where $k_{drag} = 0.5 \cdot C_D \cdot \rho \cdot A$

The frontal area, A, on the tire is the product of the tire width and fluid contaminant layer thickness.

Surface Temperature

Surface temperature is observed to raise friction level on winter contaminated surfaces as temperature falls. The mechanisms producing that effect are many. A major effect is believed to be a rise in friction due to the increased shear strength and the absence of free water, which could act as a lubricant.

For convenience we choose the Kelvin temperature scale as basis for temperature modelling. That way we do not have to work with a minus sign below freezing temperatures. The temperature adjustments will be done with two linear equations. The number 1 adjustment will be valid from 268 to 273 Kelvin. Number 2 adjustment will be valid below 268 Kelvin. These ranges will be adjusted after investigating the available field-test database.

The composite temperature function is shown in Figure 5.

TABLE 1 Winter Parameters for One Tire Configuration and One Surface Type and Condition

Winter Parameter per Tire Configuration	Designation	Unit	Range	Source
Shapefactor	C	Dimension less	1.4<C<11	Variable slip measurement
Shapefactor Speed Number	c_v	km/h		Variable slip measurement
Loss of Hardness Factor	H	Dimension less	0.05<H<1	Test program table per surface type and condition
Temperature Factor 1	k_{temp1}	Dimension less	$0.001< k_{temp1}<0.1$	Test program table per surface type and condition
Temperature Factor 2	k_{temp2}	Dimension less	$0.0001< k_{temp2}<0.01$	Test program table per surface type and condition
Planing Factor	k_p	Dimension less	$0<k_p<1$	Friction measurement or table from test program
Drag Factor	k_{drag}	sfn/km²/h²	$1\cdot10^{-5}<k_{drag}<3\cdot10^{-2}$	Friction measurement or table from test program
Rolling Resistance	R	Dimensionless	0.0001<R<0.3	Friction measurement or table from test program
Abrasive Application	A_a	Dimensionless	$0.01<A_a<0.2$	Friction measurement or table from test program
Peak Speed Factor	k_v	km/h	$100<k_v<1000$	Test program table per surface type and condition
Ultimate Friction Value	P_0	sfn	$0<P_0<1.2$	Test program table of averages per surface type and condition
Ultimate Critical Slip Value	S_0	km/h	$0<S_0<30$	Test program table of averages per surface type and condition
Critical Slip Ratio Factor	k_c	km/h	$100<k_c<1000$	Test program table per surface type and condition

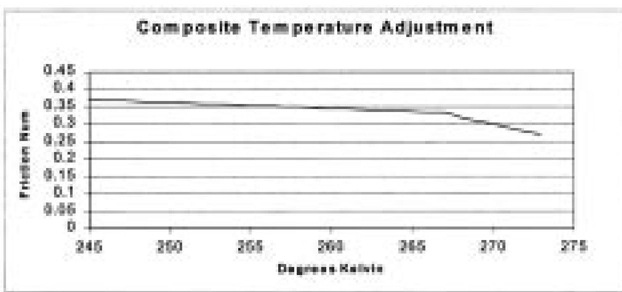

FIGURE 5 Temperature adjustment function consisting of two lines with an intercept at 267 Kelvin.

A temperature dependency term, k_{temp}, is introduced as the gradient of a linear equation, with T being a difference in temperature from a reference.

Rolling Resistance

Rolling resistance has been shown to be a geometrical relationship involving the position, a, of the resultant normal force reaction from ground and the deflected wheel radius, r.

Equation (6):

$$F_{X,R} = \frac{a}{r} \cdot F_{WEIGHT}$$

Compaction Rolling Resistance

For loose snow the tire will compress the loose snow to a higher density, when caught under vertical force, is treated as rolling resistance. The horizontal force is treated as a drag force, F_{XC}.

Equation (7):

$$F_{xc} = k_c \bullet 1/r. \bullet w \bullet d_c \bullet \rho \bullet v$$

Where r is the tire radius, w is the tire width, d_C is the depth of snow layer compacted, ρ is the snow mass density, v is the tire velocity and k_C is a factor of compacting.

For a given tire configuration the r and w are fixed. These parameters can thus be combined into the k_C factor.

Viscous and Dynamic Fluid Lift Planing

The presence of water or water in solutions of de-icer chemicals introduces hydroplaning when the surface is hard and dense enough to support it. Wet, hard ice is an obvious candidate for hydroplaning.

Partial planing may be operational. It can be treated as loss of contact area for solid interaction. As peak friction is very susceptible to the net or real contact area, a planing fraction parameter, $k_{planing}$, for the peak friction value is introduced. The general equation (Equation [8]) for dynamic fluid lift (planing) is

$$F_L = 0.5 \cdot C_L \cdot \rho \cdot A_L \cdot v^2$$

Where C_L is a lift coefficient, ρ is the density of the fluid and A_L is the area of the tire being lifted.

For convenience we use the work by Horne of NASA.

Equation (9):

$F_L = k_{pl}\, v / V_C$

Where k_{pl} is a lift coefficient, V_C is the critical planning speed for the tire-surface and v is the speed this lift gives rise to a horizontal slip friction on the remaining not detached tire-surface contact area. Thus we have Equation (10):

$F_{XL} = \mu_{slip}$ Fweight $(1 - v/V_C)$

TABLE 2 Penn State Winter Parameters for One Tire Configuration and One Surface Type and Condition

Winter Parameter per Tire Configuration	Desig-nation	Unit	Range	Source
Speed Constant	V_p	km/h	$10<V_p<1000$	Fixed slip measurement
Loss of Hardness Factor	H	Dimensionless	$0.05<H<1$	Test program table per surface type and condition
Temperature Factor 1	k_{temp1}	Dimensionless	$0.001< k_{temp1}<0.1$	Test program table per surface type and condition
Temperature Factor 2	k_{temp2}	Dimensionless	$0.0001< k_{temp2}<0.01$	Test program table per surface type and condition
Planing Factor	k_p	Dimensionless	$0<k_p<1$	Friction measurement or table from test program
Drag Factor	k_{drag}	$sfn/km^2/h^2$	$1\cdot10^{-5}<k_{drag}<3\cdot10^{-2}$	Friction measurement or table from test program
Rolling Resistance	R	Dimensionless	$0.0001<R<0.3$	Friction measurement or table from test program
Abrasive Application	A_a	Dimensionless	$0.01<A_a<0.2$	Friction measurement or table from test program
Ultimate Friction Value	F_0	sn	$0<F_0<1.2$	Test program table of averages per surface type and condition

SUMMARY OF MODIFIERS

The Logarithmic Model

In summary, we have 13 parameters to determine, slip speed, s, travel speed, v, and temperature difference, T, are variables of the process (Table 1).

Penn State Model

The modifiers are designated in the same manner as for the logarithmic friction model. Field-testing will show if they must be treated as two different sets of modifiers or if we, in practice, may use them with both models. The speed constant is unique for the Penn State Model and is meaningless with the logarithmic model.

The modified Penn State Model comprises 9 modifiers (Table 2).

PRELIMINARY RESULTS FOR NORTH BAY TESTS

From the North Bay tests it has been shown that there is no speed effect. Since they are all run at low fixed slip ratios there is not a very good speed range and thus not very good data to determine speed effect. Devices that had variable fixed slip ranges did show speed effects, but not enough data was collected. This coming year a better test plan to explore speed effect will need to be devised. For some devices temperature effects were determined. It should be noted that better temperature measurement will be needed and all devices will need to be equipped with their own surface temperature measurement. Last, preliminary results show that for the conditions of ice and packed snow that:

- All devices participating, a simple correlation is possible if the data is grouped so that only the same conditions, in the pared groups are correlated.
- That contact pressure is a very strong influence, in fact in the simple correlation there is a very strong influence of contact pressure on the multiplier constant (correlation is R^2=0.82).

There is more work to do, however, preliminary results show that the various modifiers are playing an important roll and help in developing the test plan.

PRELIMINARY RESULTS FOR MINNDOT AND NORWAY TESTS

This preliminary project was successful in establishing better Bayesian values (filtered peak friction value) and showed that the Rado Model constants can be used to differentiate contaminates. The peak friction along with the slip speed at the peak separates the ice and snow from dry or wet. The shape factor then separates loose snow and slush from packed snow and ice. The project showed that friction levels can be monitored in real time and salting control does appear to be feasible either with a go-no-go or perhaps with varying levels of salting.

More sites where to be tested last season to finalize how the three Rado constants can be used to differentiate the contaminate; however, there was not proper conditions and the tests are to be rescheduled for this coming year. It is planned to continue the study in the US with more experiments. MinnDOT, IowaDot and MichiganDOT mounted a unit on a salting truck and will evaluate its use during the coming winter season.

ACKNOWLEDGMENTS

This paper reflects the intellectual contributions of many people The team of people working together in the Joint NASA/TC/FAA//NRC Winter Runway Friction Measurement Program and the teams working on highway friction measurement for winter maintenance are sharing their ideas and research findings openly and frequently as the works progress.

Retrofit Solution for Out-of-Plane Distortion of X-Type Diaphragm Bridges

AYMAN KHALIL, TERRY J. WIPF, LOWELL GREIMANN, DOUGLAS L. WOOD, AND BRUCE BRAKKE

Some of the Iowa Department of Transportation's (Iowa DOT) welded continuous steel plate girder bridges have developed cracks in the negative moment regions at the web gaps with diaphragm connection plates. A method to prevent cracking in bridges with X-type or K-type diaphragms had been suggested by the Iowa DOT, which consists of loosening the bolts in some of the connections between the diaphragm diagonals and the connection plates. The objective of the paper is to investigate the effectiveness of this method in X-type diaphragm bridges. The research was sponsored by and performed in cooperation with the Iowa DOT. The experimental investigation included selecting and testing three bridges: two skew and one non-skew. The finite element method was used to choose the testing locations at which strain gages and displacement transducers were attached. The response at these locations was collected before and after implementing the method. Bridges were subjected to truck loading in different lanes with different speeds. The preliminary results show that the behavior of the web gaps in X-type diaphragm bridges was greatly enhanced by the suggested method as the stress range and out-of-plane distortion were reduced by at least 42% at exterior girders. Based on the results of the study, it is recommended to implement the method in bridges with X-type diaphragms. Key words: out-of-plane distortion, fatigue, plate girder bridge, cracking, diaphragm connections.

INTRODUCTION

A number of localized failures have developed in steel bridge components due to fatigue during the past several decades. Some of these have resulted in brittle fracture. Out-of-plane distortions in a small gap at the diaphragm connection plates are the cause of the largest category of cracking in steel bridges (*1*). The problem has developed in different types of bridges, including suspension bridges, girder floor beam bridges, multiple girder bridges, tied arch bridges, and box girder bridges.

Figure 1 shows a schematic of the out-of-plane distortion at the end of transverse diaphragm connection plates in plate girder bridges. Under typical vehicle loading, differential vertical deflection of adjacent girders causes forces to develop in the diaphragm elements, which cause the out-of-plane loading on the girder web (Detail A). Without the stiffener attachment to the top flange and with the top flange rigidly connected to the bridge deck by shear

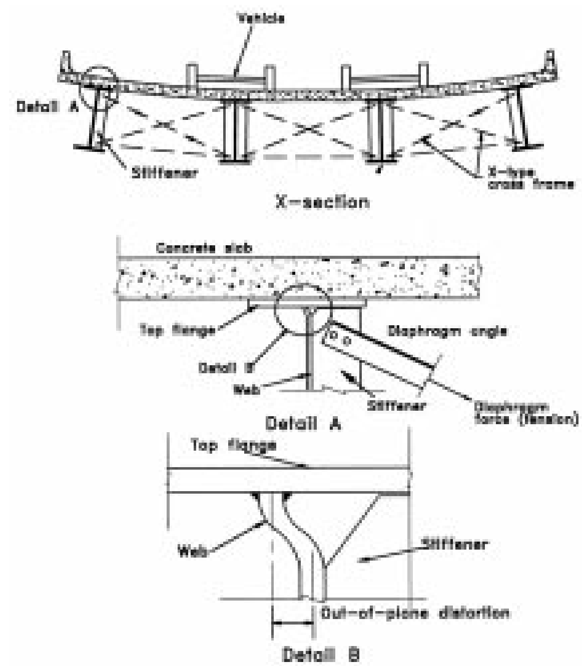

FIGURE 1 Description of out-of-plane girder web distortion in the gap region.

connectors, these forces pass through girder web causing out-of-plane distortion and, hence, bending of the web gap immediately adjacent to the top flange (Detail B). In the negative moment regions, high cyclic stresses due to this distortion cause cracking in the web gap region typically parallel to the longitudinal tensile stresses (*2*).

Current AASHTO Specifications (*3*) require a positive attachment between transverse connection plates for the diaphragms and both girder flanges. However, for many in-service bridges, the connection plates are welded only to the web and the compression flange. This was done because bridge specifications, at the time these bridges were constructed, discouraged welding of connection plates to the tension flange.

The primary objective of this work is to investigate a method suggested by the Iowa DOT to prevent web cracking. This method consists of loosening the bolts in some of the connections between the diaphragm diagonals and the stiffeners, which are welded to

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FIGURE 2 I-80 Bridge No. 7804.8L080.

either the girder bottom, compression flange, and the web or to the web only.

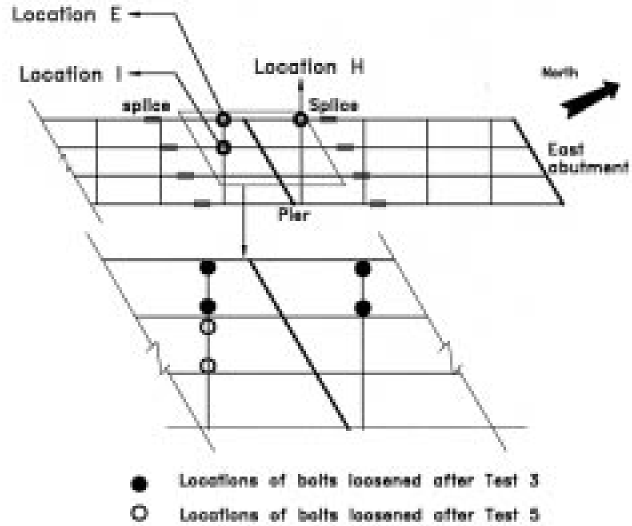

FIGURE 3 Schematic plan showing the instrumentation locations.

DESCRIPTION OF THE BRIDGE

The experimental phase of the study involved testing five bridges (three with X-type diaphragms and two with K-type diaphragms). However, only the results of one bridge with X-type diaphragms will be described in this paper due to space limitations. This bridge is located in Pottawattamie County (Iowa), 6 km west of the junction of Interstate 80 (I-80) with U.S. 6 road. The bridge was constructed in 1965. A photo of the bridge is shown in Figure 2. The superstructure consists of four continuous steel plate girders topped by a reinforced concrete deck slab. The girders have three spans of 27.89, 35.66, and 27.89 m, respectively with a lateral spacing of 2.95 m and are skewed 30 degrees with the bridge substructure. Between the supports, girders are braced using X-type diaphragms at an approximate spacing of 6.71 m. Vertical web stiffeners are closely fitted to the tension flange with a diagonal cope at approximately 50 mm from the web-girder intersection. At locations of intermediate diaphragms, two-bolt connections are used to connect diaphragm elements to the stiffeners.

Fatigue cracks have been suspected or confirmed in the web gap region at nine locations of the bridge. Eight of these locations were in the negative moment region and only one was in the positive moment region. As a retrofit, holes were drilled. However, the cracks extended beyond the drilled holes in some of these locations. Crack extensions were treated by drilling holes at the new crack tip locations.

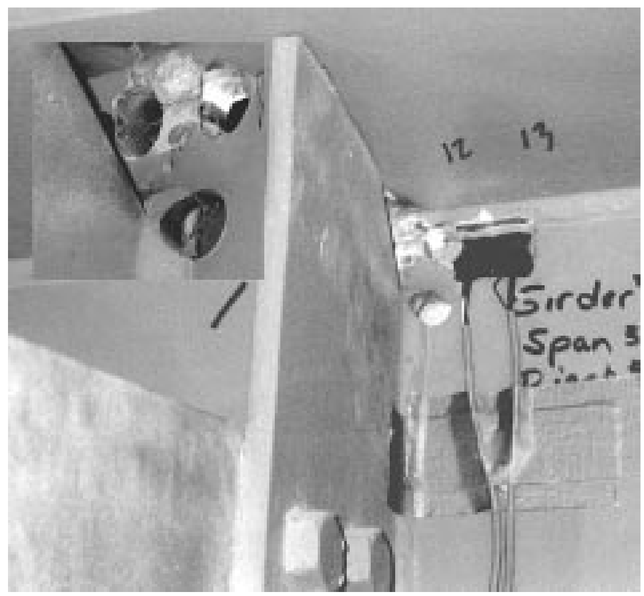

FIGURE 4 Cracks and drilled holes at Location H.

RESEARCH METHOD

Two sets of bolts connect the diaphragm diagonals to the girder webs near the top and bottom flanges. The lower set of bolts is easily accessed in the field and, hence, the method was examined by loosening these bolts. The Iowa DOT provided two rear-tandem-axle loaded test trucks. Each weighed approximately 220 kN. Due to the high traffic volume, traffic was allowed to flow during testing. However, sufficient distance separated the test truck, running at the traffic flow speed, from other vehicles on the bridge to ensure sound interpretation of results.

With the bolts tight, Tests 1-3 (tight condition tests) were conducted with the test truck traveling in the driving lane, passing lane and between lanes. Later, the lower connection bolts attaching both exterior panel diaphragm diagonals to the stiffeners at Locations E and H were completely loosened. Next, Tests 4-5 (partial loose condition tests) were conducted with the truck in the driving and passing lanes, respectively. These tests allowed an investigation of the effects of partial loosening of the diaphragm bolts (by partial loosening it is meant completely loosening the lower connection bolts of the exterior diaphragm panel) on the web gap behavior of

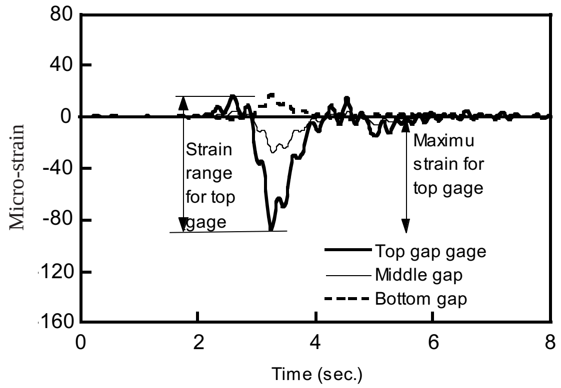

FIGURE 5 Strain data in the web gaps of Locations E, I and H during Test 3.

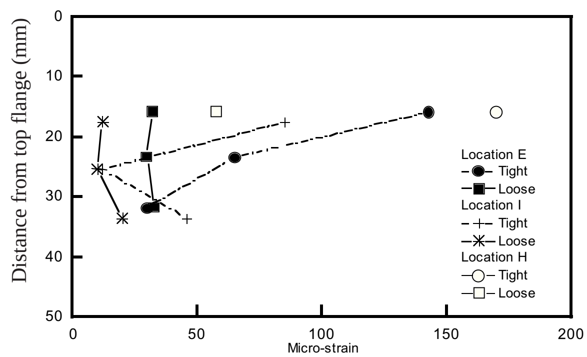

FIGURE 6 Strain range at Locations E, I, and H.

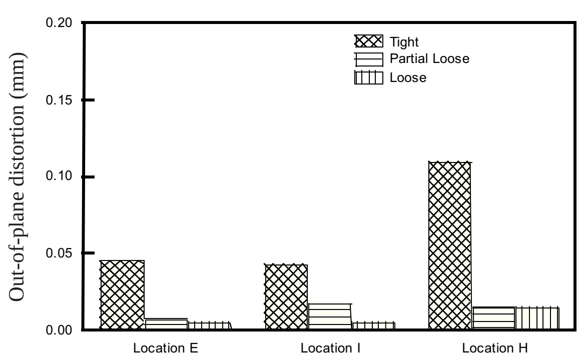

FIGURE 7 Maximum out-of-plane distortion at Locations E, I, and H during tight, partial loose and loose condition tests.

both exterior and interior girders. Later, the lower connection bolts of the interior panel at Location I were loosened and Tests 6-7 (loose condition tests) were conducted with the truck in the driving and passing lane, respectively. During testing, truck speed ranged from 88 to 100 km/h. Although not reported here, additional data were collected with ambient traffic.

Figure 3 shows a schematic half plan of the bridge showing the instrumented locations. Strains and out-of-plane distortion were measured in three web gaps: (1) at an exterior girder with no cracking (Location E), (2) at an interior girder with no cracking (Location I), and at an exterior girder where cracks were detected and holes were drilled (Location H-Figure 4). Diaphragm diagonal strains at location E were also measured. Other locations were instrumented; however, they are outside the scope of this paper.

RESULTS

Web Gap Strains

Figure 5 shows the strain reading of the gages installed in the web gap of Location E during Test 3 (tight condition). This figure further shows how the maximum strain and the strain range are defined. Figure 6 shows the maximum strain ranges computed at Locations E, I, and H in both tight and loose conditions. Obviously, the strain ranges in the web gap at Location H were slightly higher than the corresponding values at Location E implying that drilling holes technique may not prevent crack extension. The maximum computed strain ranges during tight condition tests at Locations E, I, and H were 143, 85, and 170 micro-strain, respectively. In the loose condition tests, there was a significant reduction in the strain ranges, as the maximum strain ranges were 32, 12, and 56 micro-strain, respectively with at least 65% reduction compared to the tight condition tests. It should be noted that the reported values in this paper are those obtained directly from the strain gage readings without extrapolation inside the web gap.

Web Gap Out-of-Plane Distortion

In the tight condition, the maximum out-of-plane distortion at Locations E, I and H (as shown in Figure 7) were 0.046, 0.043, and 0.110 mm, respectively. The out-of-plane distortion at Location H (with cracks and drilled holes) is more than twice those occurred at either Location E or Location I. This conforms to what was reported by Fisher et al. (*4*) where it was mentioned that the amount of the out-of-distortion increased with the existence of cracks. This implies that cracks and drilling holes reduced the lateral stiffness of the web gap significantly. Note, however, that the maximum values of strains were of comparable magnitude at the three locations.

Generally, loosening bolts reduced the web gap out-of-plane distortion at all the three locations. As illustrated in Figure 6, loosening only the exterior diaphragm panel bolts (partial loose condition) produced effects comparable to complete loosening both exterior and interior panel bolts (loose condition) on the web gap at Locations E and H. For Location I, there was a significant reduction in the out-of-plane distortion between the partial loose and loose condition tests.

Diaphragm Diagonal Forces

In the tight condition, the peak forces in the diaphragm diagonal (D1) showed variation with the transverse truck position with a peak magnitude of approximately 6 kN. The forces nearly diminished upon either loosening the exterior panel bolts (partial loose condition) or both exterior and interior panel bolts (loose condition). During the loose condition tests, the maximum force was within 0.5 kN.

SUMMARY

The general out-of-plane deformation behavior of the web gaps was enhanced as the nondestructive retrofit method was applied. For this bridge, the strain range in all tested web gaps was reduced by at least 65%. A similar trend was noticed for the out-of-plane distortion for which maximum values at exterior and interior girders decreased by at least 83% and 88%, respectively. Tests on other X-type diaphragm bridges yielded similar results. Considering the experimental results of this study, it is recommended to adopt the proposed method (loosening diaphragm bolts in the negative moment region) as a retrofit technique to prevent cracking from out-of-plane distortion of web gaps in X-type diaphragm bridges. Nevertheless, the design steps as presented in (*5*) should be performed before implementing the method.

ACKNOWLEDGMENTS

The research and results presented above are parts of the Project HR-393 funded by the Iowa DOT. The project is still in progress, hence, the results and conclusions are preliminary. The authors gratefully acknowledge the interest, help, and cooperation of numerous Iowa DOT personnel.

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1998 TRANSPORTATION CONFERENCE PROCEEDINGS

Effects of Damage on the Behavior of Pretensioned/Prestressed Concrete Beam (PPCB) Bridges

FRANCESCO M. RUSSO, F. WAYNE KLAIBER, TERRY J. WIPF, AND WILLIAM A. LUNDQUIST

In June 1996, a series of field-tests were conducted on twin bridges carrying I-680 over county road L34 in Beebeetown, Iowa. The westbound bridge was damaged by an over-height load. The load fractured a portion of the bottom flange and web of the first two beam lines of the 11-beam structure. A majority of the strands were exposed in the process but no strands were severed. Several strands appear lax. The eastbound bridge is undamaged. The Iowa DOT decided to replace the two damaged beams due to uncertainties regarding their capacity and long term serviceability. Prior to removal of the beams, the westbound and eastbound bridges were load tested. Results from the tests are still being interpreted but the bridges do appear to behave differently. Results indicate that the differences may be only partially due to damage; the other causes being differences in end restraint of the center span and participation of the barrier rail in edge stiffening. Grillage models of an undamaged bridge have been developed and are shown to reasonably agree with the experimental results of both the damaged westbound and undamaged eastbound bridges. The analytical models are more flexible than the experimental results. This is likely due to neglect of the barrier rail stiffness and the treatment of the beams as simply supported. The influence of intermediate diaphragms has been analytically shown to have little effect on the distribution of loads in exterior lanes. Key words: prestressed, bridge, damage, load testing, analysis.

INTRODUCTION

This paper describes the results of a field test conducted in June 1997, on two similar pretensioned/prestressed concrete beam (PPCB) bridges located in Beebeetown, Iowa. The bridges carry I-680 eastbound and westbound over county road L34 in Pottawattamie County. Figures 1 and 2 present the geometry of the two bridges, instrumentation and test vehicle locations, as well as test vehicle geometry and load.

The impetus for this research project was the collision of an unknown vehicle with the north three beams, beams 1W - 3W, of the westbound structure in July 1996. Damage was centered ± 5' (1525 mm) west of the mid-span diaphragm of the center span. Approximately 6' (1830 mm) of the bottom flange spalled from the north fascia stringer, beam 1W, exposing the bottom two layers of 0.5" (12.7 mm) diameter strands. Several of the strands on beam 1W seem to be lax but no strands were severed during this collision. There was a preexisting severed strand from a 1993 collision. Cracking of the bottom flange and web as well as fracturing of the core concrete is present on both beams, but to a lesser extent on beam 2W. Cracking appears to have been arrested by the presence of the cast-in-place concrete diaphragm. The second interior beam, beam 3W, was also damaged, but not as severely, with the damage consisting of the spalling of a patch installed following prior collisions with the bridge.

Prior to the initiation of this research project, the IADOT decided that beams 1W and 2W would be replaced while beam 3W would be patched. The decision to replace beams 1W and 2W was made due to uncertainties concerning the remaining strength, effect of damage on load distribution, and long-term serviceability of the damaged beams. IADOT's decision to support a study on load distribution and remaining strength in damaged PPCB bridges is an attempt to develop a more refined criteria through which the effects of damage on the behavior and strength of such bridges can be more accurately assessed. The project has two objectives: determine the effects of damage on load distribution and assess the remaining strength of a damaged member.

FIELD TESTING OF THE I-680 BEEBEETOWN BRIDGES

The focus of the load testing portion of this project was to track the flow of forces in a damaged and complementary undamaged PPCB bridge. The project endeavored to answer the following questions:

- Are redundant load paths available to assist in load sharing?
- What load is on the damaged beam(s)?
- Has the redistribution of load following damage overloaded any elements that were not originally damaged?

The answer to the first question is evident. It is well known that typical multiple stringer bridges are highly redundant in that "failure" of an individual superstructure element does not constitute collapse of the structure. This is primarily due to the interconnection of adjacent stringers through a common slab and secondarily due to the presence of intermediate diaphragms. However, the answer to the load distribution questions could only be determined through testing and subsequent data analysis of the two bridges.

The static tests conducted on the Beebeetown bridges used loads of known magnitude and configuration. The tests employed two IADOT supplied maintenance vehicles (dump trucks) described in

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TABLE 1 Test Truck Position Matrix

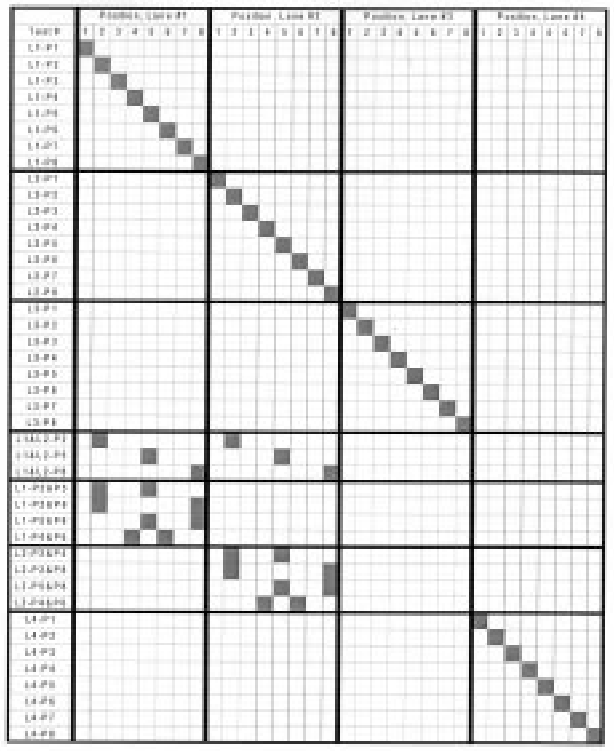

Figure 2. Most of the tests were conducted with a single truck, IA-4280, at various longitudinal and transverse positions. The tests were designed to place the truck at a specified transverse location on the bridge, i.e. a specified "lane" location, and incrementally move the truck while taking readings at predetermined positions. The predetermined truck locations are illustrated in Figure 2 and described by Table 1.

Table 1 presents the test name and indicates by shading the position of the center of the rear tandem of the test vehicle(s). Tests in which only one block is shaded are tests in which only one test vehicle, IA-4280, was employed. In the tests in which multiple trucks were used in the same lane, i.e. L1-P2&P5, truck IA-4280 was the lead vehicle. In tests where trucks were side-by-side in adjacent lanes, i.e., L1&L2-P5, IA-4280 was closest to the flared edge of the structure.

A number of deflection transducers and electrical resistance strain gages were used to instrument the bridges. Celesco PT101 and Unimeasure HX-PA displacement transducers were used at various locations as shown in Figures 1 and 2. Micro Measurements 032UW 'C Feature' gages were used to acquire strains in four of the exposed strands of the westbound bridge. Precision Measurements F-2400-06 foil gages were installed on the center span intermediate diaphragms connecting beams 1 through 4. F-2400-6 gages were also installed at the end regions of beams 1 and 2 near the pier diaphragm. These gages were intended to detect any strains at the ends of the beams which would be indicative of a degree of continuity. Refer to Figure 1 for partial details of the instrumentation layout.

TEST RESULTS

Selected results from the tests on both the eastbound and westbound bridges will be discussed in this section. The results are still being interpreted and compared to analytical predictions at the time of this writing. The intent of the experimental/analytical comparison is to develop computer modeling guidelines for damaged structures that will allow practicing engineers to more accurately assess the effects of localized damage on the overall behavior of damaged PPCB bridges. The following text will selectively comment on the observed behavior of the two bridges tested. Results from only the center span will be presented.

Figure 3 is a plot of several transverse deflected shapes taken at midspan of the damaged center span of the westbound bridge and the complementary location in the eastbound bridge. L1W-P5(A) and (B) are tests performed at the same location of the westbound bridge and illustrate repeatability of the experiment. L1E-P5 is a similar test of the eastbound bridge. The accompanying series entitled STAAD-1 (w/o) and (w) are computer models where the effects of the midspan diaphragm were included (w) or excluded (w/o) from the model. The model presumes that the members are undamaged, there is no edge-stiffening participation from the barrier rail, and that all beam ends are simply supported (i.e. continuity with the tail spans is ignored). It is reasonable to initially assume that there is no continuity because of the lack of pintles in the curved sole plate pier bearings along beam lines 1, 2, 10, and 11, along with the fact that the pier diaphragm does not encase the beam ends.

Figure 3 illustrates several key points. First, there is close agreement between the undamaged analytical models and the behavior of the damaged westbound bridge. This may either indicate that the damage is such that it has no measurable effect on the response of the bridge or that a number of other factors such as barrier rail stiffening or interior beam line continuity result in a deflected shape that appears undamaged. If in fact the analytical model is too "soft," the behavior of a more representative model may tend to more closely match that of the eastbound bridge and one could conclude that the damage was sufficient to alter the behavior of the westbound bridge. Figure 1 also illustrates the negligible change in deflected shape due to the presence of a midspan intermediate diaphragm. This would tend to indicate that the exterior beam line intermediate diaphragms have little effect on overall deflection patterns.

Figure 4 depicts the experimental and analytical transverse response when loads are moved to the second longitudinal lane line. In this figure it is again apparent that the analytical models are more flexible than either of the bridges tested. The westbound bridge appears to be resisting the majority of load in a localized area surrounding the point of application more so than the other scenarios. The presence of intermediate diaphragms has a more pronounced effect on the response than when loads are placed in lane 1.

Figure 5 illustrates a large difference in the experimental and analytical responses of the bridges. In this series of tests, truck IA-4280 was positioned to be 2' (610 mm) from the median rail. The damaged region of the westbound bridge would have little to no effect on this load location and the eastbound bridge is not known to be damaged at all. As expected, the bridges behaved similarly with the westbound bridge being somewhat softer in the transverse direction. However, there is a great disparity in the predicted and measured response. The predicted response on the median side is

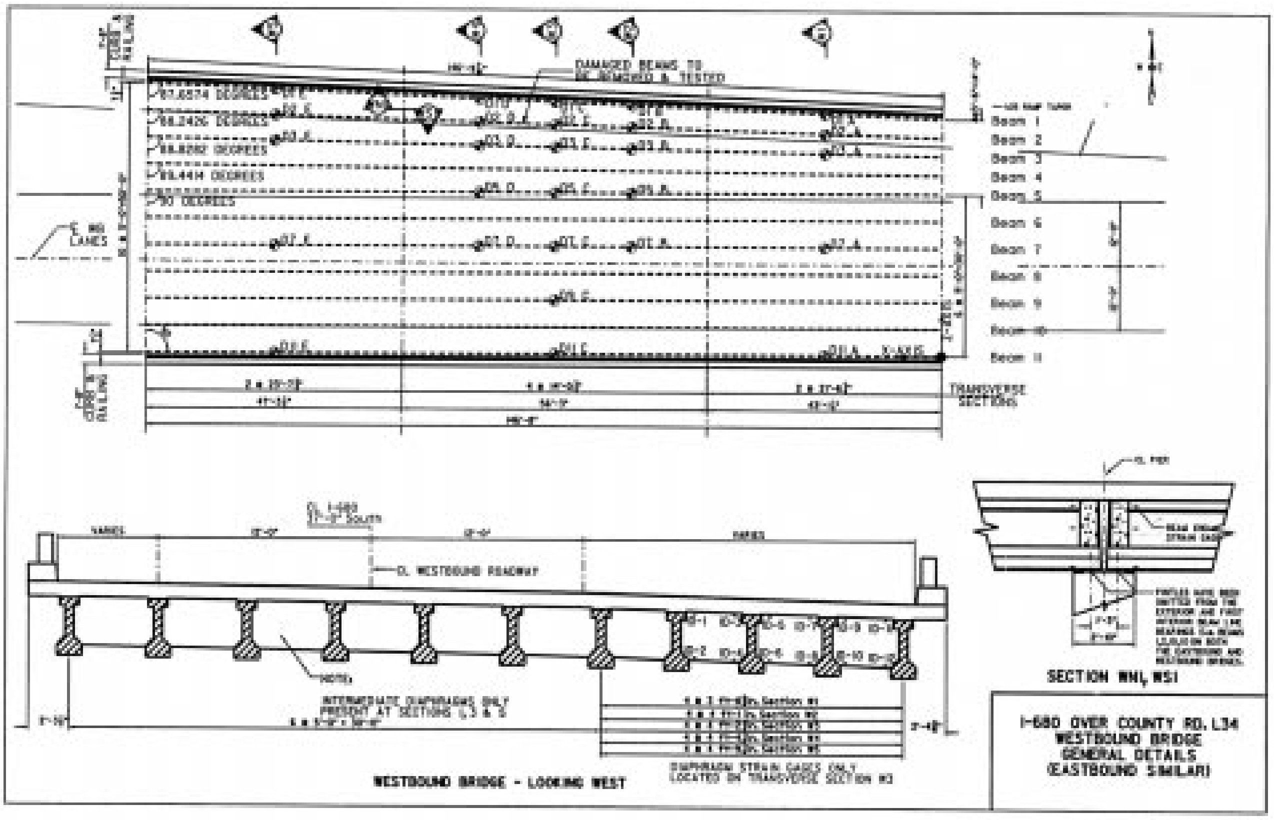

FIGURE 1 General plan and instrumentation layout.

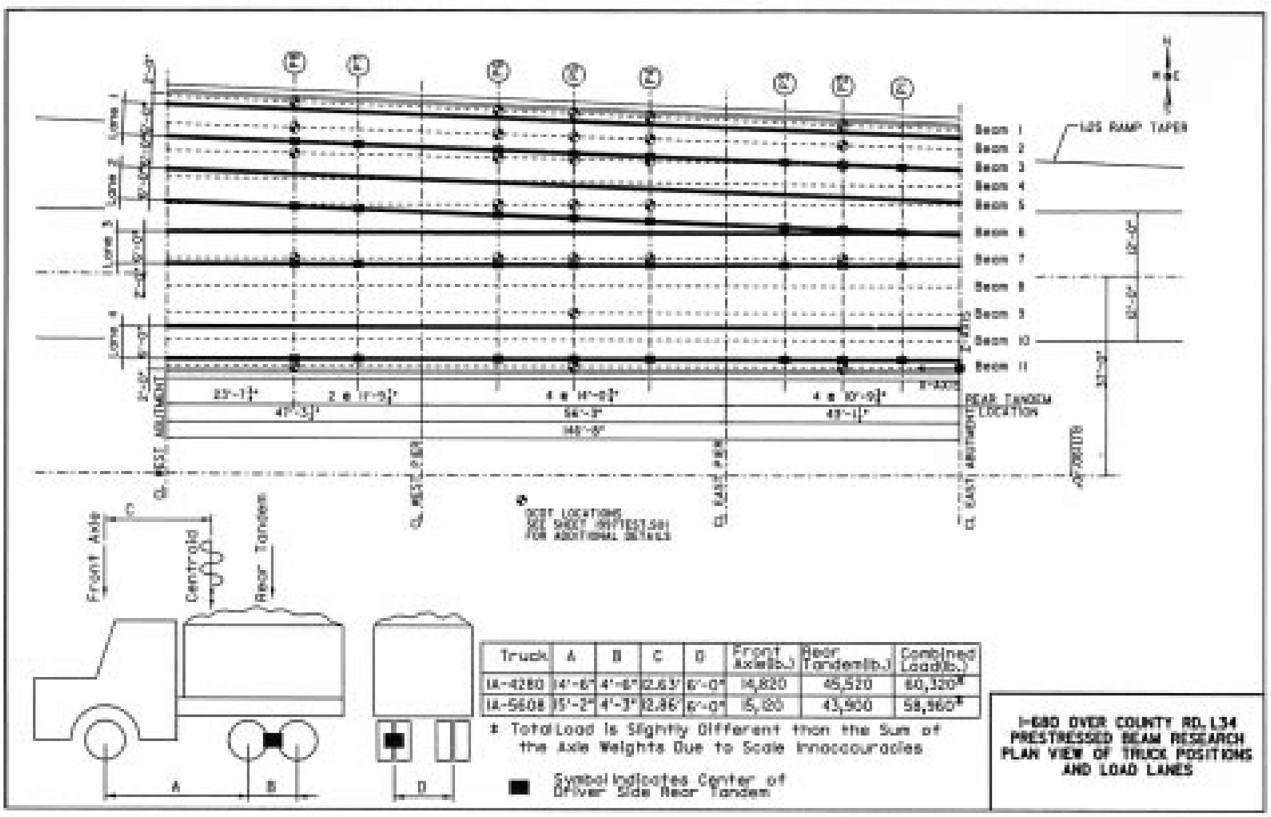

FIGURE 2 Test truck positions, geometry, and axle loads.

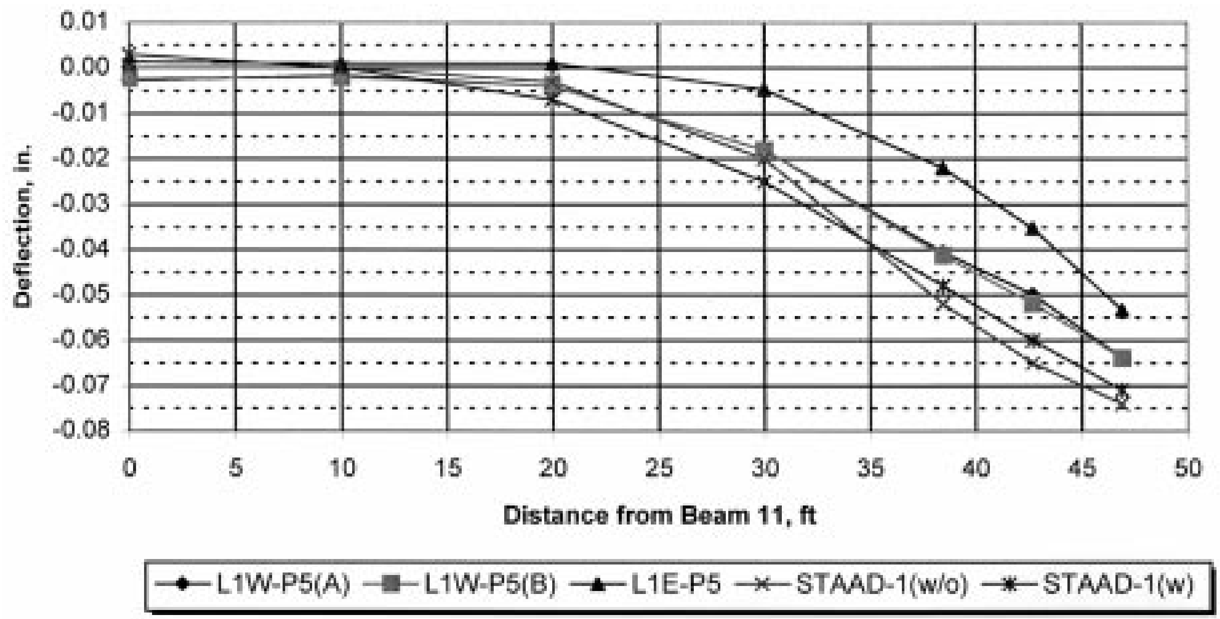

FIGURE 3 Transverse deflection, midspan, center span, lane 1 loaded.

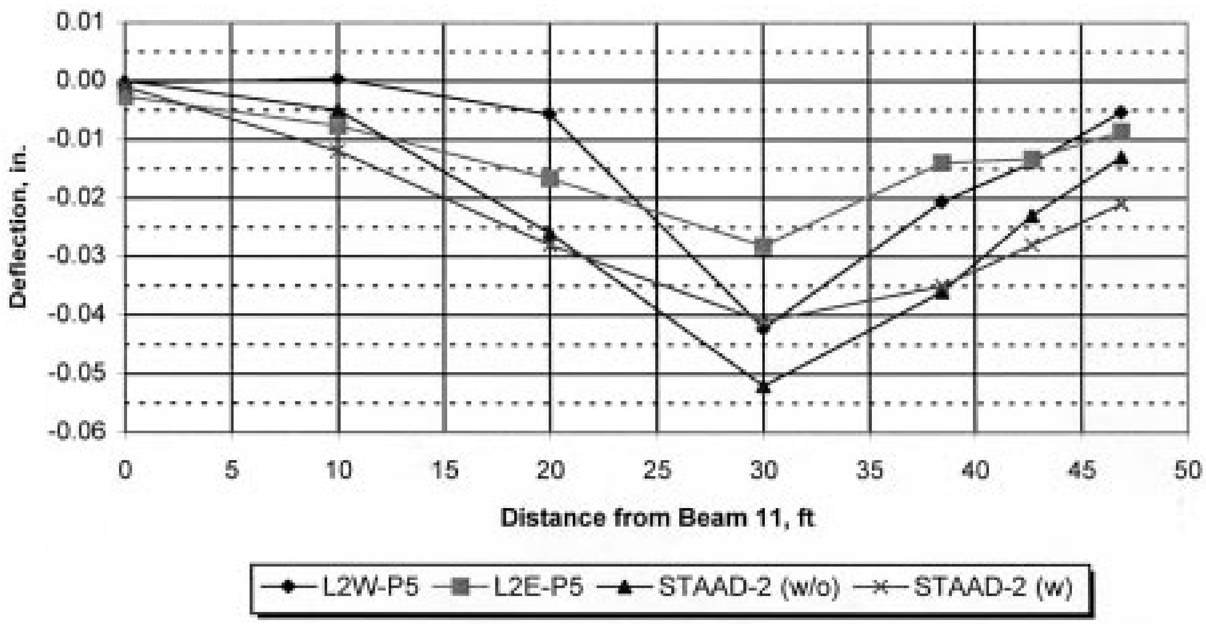

FIGURE 4 Transverse deflection, midspan, center span, lane 2 loaded.

expectedly larger than on the flared side due to the greater beam spacing. However, both the eastbound and westbound bridges deflected less on their median sides than the flared sides. The only explanation for this behavior is a much stiffer structure than analytically modeled. This stiffness is likely due to barrier rail participation and unaccounted for continuity effects.

Figure 6 presents the analytical and experimental results for the case where both loaded test vehicles were placed side-by-side with their rear tandems at midspan. The westbound bridge appears somewhat softer than the eastbound, but the characteristic shape of the transverse response is the same. The analytical models are once again more flexible than the experimental response. It is of interest

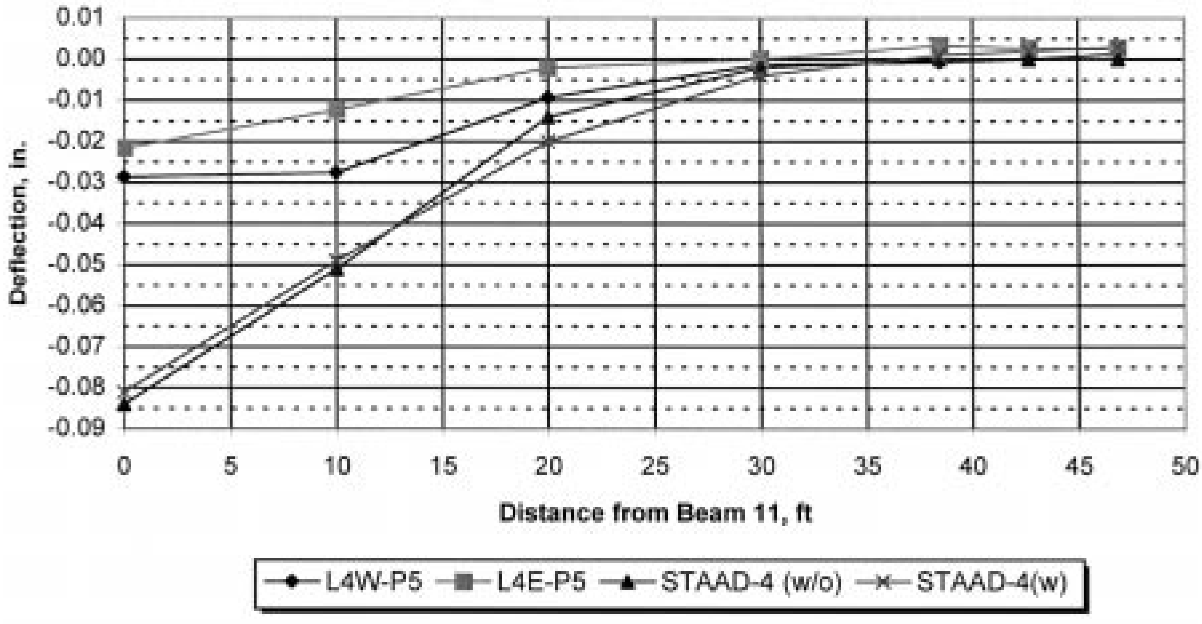

FIGURE 5 Transverse deflection, midspan, center span, lane 4 loaded.

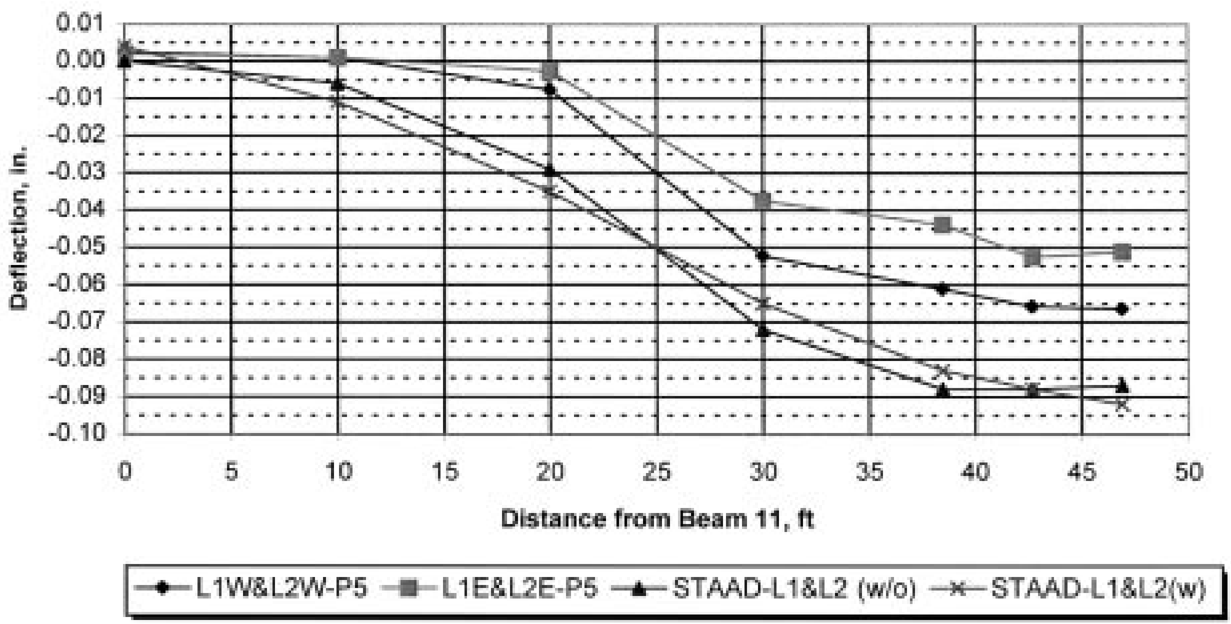

FIGURE 6 Transverse deflection, midspan, center span, lanes 1 and 2 loaded.

to note that the intermediate diaphragms appear to once again have little effect on the transverse deflected shape (load distribution) pattern of the bridge. The effect is only a slight flattening of the transverse response with a correspondingly small increase in deflection and moment to the exterior beam.

Examination of the diaphragm strain gages indicates that strain is consistent with deflection. Tests in which appreciable deflections were recorded in beams 1-5 tended to have higher strain readings in the diaphragms. Examples of these tests are the single and multiple truck tests in lanes 1 and 2. In these tests, diaphragm strain readings as high as ±15 $\mu\varepsilon$ were recorded in the westbound bridge and ±35 $\mu\varepsilon$ in the eastbound bridge. When comparing similar load positions in the two bridges, the eastbound bridge diaphragm strain gages consistently read higher than the westbound bridge gages.

The beam end strain gages gave an intermittent indication of continuity though the indications were stronger and more consistent on the eastbound bridge. The most pronounced response in the westbound bridge was in beam 1 at the bottom flange strain gage on the center span beam. The highest reading under a single truck was with a truck positioned at midspan of the center span. The reading was -31 $\mu\varepsilon$. Side-by-side loads did not increase the strain response in beam 1. In a test specifically designed to check for continuity over the piers, loads were positioned at P5 and P8. In this scenario, the maximum strain recorded was -46 $\mu\varepsilon$. The highest strain readings were with trucks at P4 and P6, a reading of -48 $\mu\varepsilon$.

In contrast to the westbound bridge strain readings are those obtained on beam 2 of the eastbound bridge. In tests under a single truck in lane 1, the bottom flange strain gage indicated compressive strains of -165 $\mu\varepsilon$ with a load at midspan of the center span. The corresponding bottom flange strain gage in the east tail span was not comparable. This is an indication of restraint due to factors other than full longitudinal continuity. In side-by-side tests, this strain increased to -200 $\mu\varepsilon$ and when trucks were placed at P4 and P6 so as to maximize the amount of load in the center span, the bottom flange strain gage read -235 $\mu\varepsilon$. Although no conclusions can be drawn about the amount of restraint present in other uninstrumented beam lines or spans of the eastbound bridge, nor the source of such restraint, the existence of these high compressive strains lends credence to the argument that a certain degree of negative moment capacity exists at the piers of this bridge. It is interesting to note that on both the eastbound and westbound bridges, only the bottom flange gages measured any appreciable strain. The web and top flange gages gave very little strain indication whatsoever.

SUMMARY

This paper has briefly described an ongoing investigation into the behavior of impact damaged ppcb bridges. As of this point, the bridges have been tested in the field and evaluation of that data continues. Tentative conclusions indicate that the eastbound and westbound bridges are behaving differently. This may either be de to damage or other differences in behavior. The deflections measured under known loads were small, the maximum being less than 0.1' (2.5 mm). Strains have been detected in both the intermediate diaphragms and at the beam ends. Strain values were higher in the eastbound bridge with the beam end strains being substantially so.

Further calibration of the analytical models is required to assess the sensitivity to factors such as loss of stiffness in a damaged member, effects of varying degrees of end restraint, and changes in structural behavior due to barrier rail participation. In the summer of 1998, the two damaged beams removed from the westbound bridge will be tested in order to determine the effect of the impact on their stiffness and strength. An additional three beams will be tested that have been damaged in the laboratory. Following these tests and the completion of a number of other analytical tasks, it is the intent of this project to provide a series of recommendations regarding when it is necessary to consider replacing a damaged ppc bridge beam.

Modal Testing for Nondestructive Evaluation of Bridges: Issues

AYMAN KHALIL, LOWELL GREIMANN, TERRY J. WIPF, AND DOUGLAS WOOD

Currently, global bridge inspection relies primarily on visual inspection with its well-known limitations. Among the promising global nondestructive methods is modal testing on which research has been conducted for two decades. The concept of the method is that modal characteristics (frequencies and mode shapes) of the structure are directly related to its stiffness properties which change as the structure deteriorates. At Iowa State University, a research program has been initiated to study modal testing using ambient excitation with the objective to determine the ability of the method to detect, locate, and determine the measurable size of defects in steel plate girder bridges. Effects of several factors on modal signatures are examined either experimentally or theoretically: environmental conditions, excitation trucks, pseudo-structural defects, deck rehabilitation, and structural cracks. The experimental portion consisted of modal tests on a bridge (Boone River Bridge in Hamilton County, Iowa) that was made available for the current investigation by the Iowa DOT. This paper presents the experimental investigation: motive, investigated parameters, and test program. Since the research is still in progress, only some preliminary results are given. Considering these results, the modal testing procedure presented proved to be an efficient method to monitor the health of bridge structures.
Key words: modal testing, nondestructive bridge testing, environmental conditions, excitation methods.

INTRODUCTION

Current bridge evaluation is based almost entirely on the use of visual inspection with its many limitations. Other localized experimental techniques such as ultrasonic methods are used only if a segment of a bridge is suspected to be defective by visual inspection. The inspection cycle time is another facet to the problem. Inspection cycles set by federal regulations vary according to the bridge classification (use and essentiality) with a regular interval of two years. Often, such a period is sufficient for defects to grow and to cause major problems. As such, there is a great need for improving bridge evaluation techniques. Primarily, the desired technique should be (1) global in nature and (2) automated. Modal testing appears to fulfill both requirements. The objective of modal testing is to obtain a signature of the dynamic or vibration behavior of a structure in the form of mode shapes and frequencies. The frequencies of vibration of the structure are directly related to the stiffness and the mass of the structure. If the structure deteriorates, the stiffness will decrease and, hence, so will the frequencies of vibration. Further, changes in mode shapes are also expected to relate to the defect location. The first use of modal testing dates back to the 1950's. Since then, it has been used extensively in testing aerospace and offshore structures.

Several researchers have conducted modal tests on bridges to monitor changes in the modal properties due to changes in the structural condition (repair or induced defects). For example, Salawu and Williams (*1,2*) investigated the effects of deck repair and bearing replacement on the modal characteristics of a voided slab bridge. Aktan et al. (*3*) monitored the modal characteristics of two truss bridges while the bridge was loaded to failure. In other investigations, Aktan et al. (*4,5*) gave a strong argument supporting modal testing and stated that it is the only nondestructive method capable of determining the global condition of civil infrastructures (including bridges). Further, Aktan et al. presented (*6*) a conceptual system, that incorporates modal testing as well as strain monitoring, to accurately assess the structural condition of bridge structures. They commented that adopting similar systems would eliminate the subjectivity encountered in current bridge-management programs.

Recently, Alampalli et al. (*7,8*) addressed an essential question of what the minimum flaw size that can be detected using modal testing. The authors predicted that a 6 cm crack length would be the minimum size detectable at the most critical section of the bridge. However, this was based on tests conducted in the laboratory remote from inevitable environmental conditions encountered in the field. Further, in almost all previous investigations, bridges were excited by an impact hammer and the response was measured at closely spaced points. If a modal test were to be automated which is a basic requirement for a successful bridge evaluation program, the ambient traffic, not an impact hammer, would be the perfect source of excitation.

Research was initiated at Iowa State University with a primary objective to determine the discrimination limits of modal testing by field testing of a typical steel plate girder bridge using ambient or controlled traffic as an excitation source. Sensitivity of the modal signatures to several condition changes of an actual bridge was evaluated both experimentally and theoretically. Experimentally, the effects of four parameters are investigated: (1) environmental conditions (temperature and wind), (2) pseudo change in the structural stiffness (i.e., change in the bridge mass), (3) excitation methods, and (4) deck rehabilitation. This paper presents the research methods for the experimental phase of the research. The tested bridge is briefly described and the instrumentation is presented. The test program is introduced with a description of the modal test procedure. Further, the method utilized to extract the modal data is described. Finally, some preliminary results are presented.

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FIGURE 1 Boone River bridge on IA-17.

BRIDGE DESCRIPTION

The bridge tested in this study is located approximately 2 km south of U.S. 20 on Iowa Highway 17 (IA-17). The bridge was constructed in 1972, and as shown in Figure 1, it is a three-span, steel welded plate girder bridge. A 200 mm thick reinforced concrete deck slab is supported on five plate girders of spans 29.70, 38.10, and 29.70 m, respectively. Composite action is provided between the deck slab and the steel girders using shear studs. At intermediate locations, X-type cross frame diaphragms brace girder webs at approximately 6.70 m intervals. Deck rehabilitation was conducted in the spring of 1997 during which 40 mm of concrete overlay was added to the existing deck.

RESEARCH METHODS

Vibration Measurement

The acceleration dynamic response of the bridge was measured and recorded using a data acquisition system (DAS). The recorded data was transferred, on site, to a notebook computer where it was stored and later transferred to the structural engineering laboratory at Iowa State University for analysis. Accelerometers were connected to the DAS using special coaxial cables, some as long as 100 m. Data were sampled at 100 points per second giving a Nyquist (cut-off) frequency of 50 Hertz. The total sampling time per test was varied as it depended on the response of the bridge to the passing vehicles. At least thirty seconds of data were collected per test with few tests exceeding a sixty-second recording time. Therefore, a frequency resolution of at least 0.033 Hertz was attainable.

Modal Test Procedure

A three-dimensional finite element model was constructed for the Boone River Bridge to help in understanding the modal behavior of the bridge, determining the response frequency range of interest, and choosing the sensor locations. Accelerometer stations were along the bridge shoulders with distances between consecutive stations varying between 3.90 and 7.00 m. The acceleration response was measured at 42 points (21 on each side of the bridge) with four of these points on the pier sections. Steel plates were affixed to the deck using moisture resistant epoxy and the accelerometers (with magnetic bases) were attached to these plates. As the number of DAS channels was limited, it was necessary to acquire data at reference locations to obtain mode shape information. Consequently, two accelerometers were attached to a reference station for all tests to provide a common database for all test results. The modal test started with the six other accelerometers attached to the steel plates (on the deck) at six locations. With the traffic restricted on the bridge, a test truck (excitation source), provided by the Iowa DOT, traveled in the West Lane at a speed of 50 km/h (in several cases the speed was approximately 80 km/h). The traffic was then allowed on the bridge and the accelerometers, other than those at the reference stations, were moved to new locations. The same procedure was repeated until data at all the 40 stations was acquired. This accounted for seven test truck runs for each test.

Test Program

A summary of the tests is presented in Table 1. Tests were categorized in five groups: A, B, C, D, and E. Within each group, two digits designate each test: the one to the right refers to the test order, and the other represents the year in which the test was conducted. Tests with the letter d after the two digits were conducted after artificially changing the pseudo stiffness of the bridge (i.e., adding a vehicle on the bridge deck) to simulate damage. The test duration varied from one test to another but was generally performed well within 90 minutes.

TABLE 1 Summary of Conducted Modal Tests

Group	Test	Parameter investigated	Induced or apparent condition change from that in October 1997
A	71-74,76-78	Environmental effects	No change
B	71d	Simulated-structural defects	232 kN (mid span of Intermediate Span)
	72d		232 kN (mid span of South Span)
	73d		232 kN (quarter point of Intermediate Span)
	74d		14 kN (mid span of Intermediate Span)
	76d		76 kN (mid span of Intermediate Span)
	77d		36 kN (mid span of Intermediate Span)
	78d		147 kn (mid span of Intermediate Span)
C	61,75	Excitation method	Ambient traffic for excitation
D	61	Deck repair	Before deck rehabilitation, ambient traffic excitation
E	81,82		No apparent change

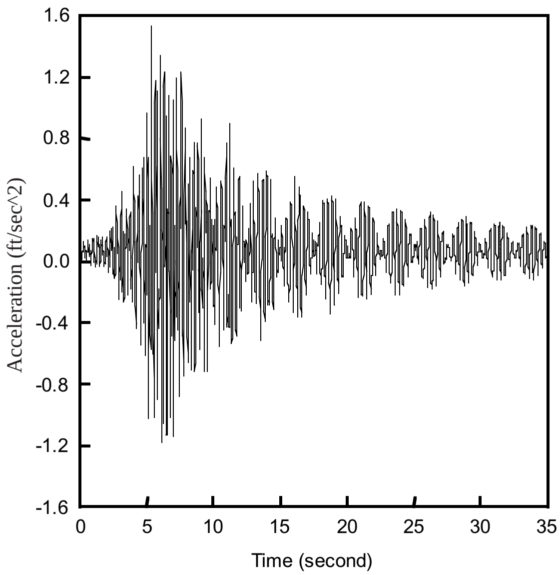

FIGURE 2 Time domain signal for the acceleration at Point RE during Test 72, Run 3.

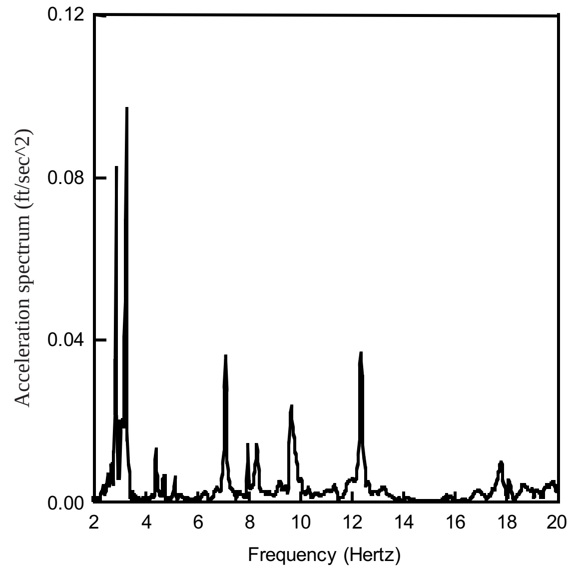

FIGURE 3 The acceleration spectrum at Point RE during Test 72, Run 3.

No change in the structural condition of the bridge was reported by the Iowa DOT or noticed by the test team while conducting Group A tests. Therefore, any changes in the modal parameters during these tests were attributed to environmental conditions and/ or experimental noise. Effects of simulated structural defects were examined by conducting the Group B tests using different-weight vehicles placed on the bridge at different locations in the East Shoulder. This changed the mass distribution of the structure and, therefore, the frequencies and mode shapes as well. The same would have been true of a change in the stiffness were made to the bridge. Obviously, the former approach is simpler than the latter and was chosen for this research. Tests of Groups A and B were conducted in pairs, that is, tests having pseudo-structural defects (added mass-Group B) were conducted concurrently with tests with no condition alteration (Group A) to minimize possible environmental effects on the results.

To determine whether the exciting truck would have an effect on the extracted modal properties of the bridge, the bridge was excited by trucks from ambient traffic in Group C tests. While preparing the test program, there was no intention to study the effects of deck rehabilitation. However, after conducting Group A tests, it was obvious that the modal characteristic of the bridge differed significantly from those during Group D (conducted in 1996). After checking with the Iowa DOT, it was determined that deck rehabilitation had been performed in early 1997 (before Group A). This allowed an additional parameter (i.e. the effect of deck rehabilitation) to be investigated.

ANALYSIS METHOD

In the current investigation, the peak amplitude method (indirect frequency domain) is utilized. In this method, the natural frequencies correspond to amplitude peaks of the response in the frequency domain. The mode shapes are computed from the ratios of the peak amplitudes, taking into consideration the relative phase angle, at various points of the bridge. The method assumes that the modes are real and it provides good results if the modes are well separated. A fast Fourier transform (FFT), with a rectangular window, was applied to each acceleration time history to get the acceleration response in the frequency domain (the acceleration auto-spectrum or the acceleration spectrum). For example, in Figure 3, the acceleration spectrum that corresponds to the acceleration time history in Figure 2 is given. Natural frequencies appear as peaks in the response spectrum. It should be noted that not all peaks correspond to natural frequencies as some peaks in acceleration spectra simply corresponded to noise in the electrical measuring system. After preliminary investigations of all response spectra, it was possible to isolate ten modes of vibration and determine the corresponding natural frequencies. An interactive program written in C++ was used to extract the frequencies and mode shapes. Figure 4 illustrates the shape of the first six detected modes.

PRELIMINARY RESULTS

Utilizing Group A test results, a regression analysis was conducted to determine statistically the impact of ambient temperature and temperature differential between both sides of the deck slab on the measured frequencies. Figure 5 illustrates the results of the correlation analysis between the normalized frequencies and ambient temperature. A high correlation coefficient of approximately 85% was computed. As shown in the figure, the frequencies of vibration tend to decrease with temperature increase. On the other hand, it was found that the effect of temperature differential on measured frequencies was minimal.

The effect of the deck rehabilitation on the modal characteristics can be evaluated by comparing the results of Group D test to

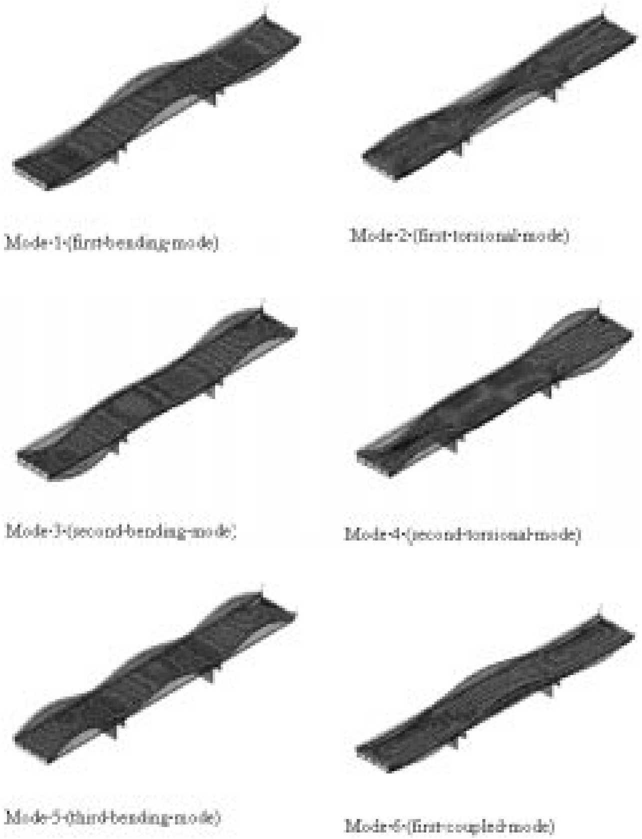

FIGURE 4 Description of the first six modes.

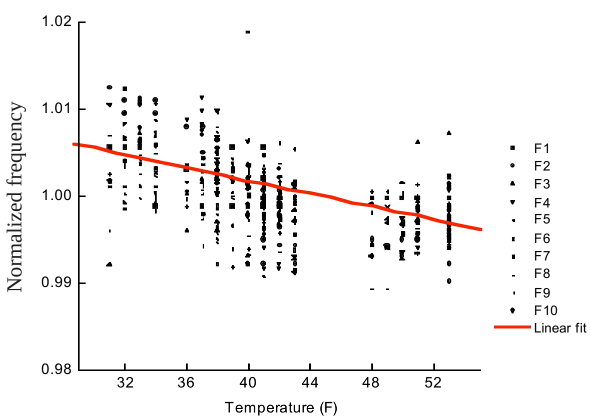

FIGURE 5 Regression results for the effect of ambient temperature on the natural frequencies.

TABLE 2 First Eight Natural Frequencies of Groups A and D Tests

Modal frequencies	Before Rehabilitation	After (Hertz)
F1	2.99	2.90
F2	3.29	3.28
F3	4.58	4.42
F4	4.81	4.76
F5	5.35	5.17
F6	6.72	7.16
F7	7.57	8.00
F8	8.02	8.38

those of Group A (a comparison is presented in Table 2). The added mass of the deck rehabilitation resulted in lowering the longitudinal bending frequencies (F1, F3, F5) significantly; however, it enhanced the transverse rigidity of the bridge resulting in minimal changes in the longitudinal torsional modes (F2, F4) and an increase in coupled longitudinal-transverse bending modes (F6, F7, F8).

ACKNOWLEDGMENT

The authors gratefully acknowledge the help and cooperation of Bruce Brakke, Maintenance bridge engineer at the Iowa DOT, during several stages of this study. Without the efforts and support of numerous other Iowa DOT personnel, this work could not have been accomplished. The project is still in progress; hence, the results and conclusions are preliminary.

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Use of Secondary Data Sources to Determine the Business Vitality Impacts of Access Management Projects in Iowa

DAVID PLAZAK, TOM SANCHEZ, AND KEN STONE

Access management is the process of carefully managing the access by vehicles from major transportation routes to adjacent land development. Access management projects, such as those involving driveway consolidation and installation of raised medians, have proven highly effective in improving traffic safety and traffic operations. However, the impact of retrofit access management projects on the vitality of existing businesses along the improvement corridor is a continuing source of concern for business owners, city officials, chambers of commerce, and transportation professionals. As part of a major research, education, and outreach project conducted for the Iowa Department of Transportation's (Iowa DOT) Access Management Task Force, a variety of secondary data sources and analytic methods were used to assess the impact of completed access management projects on local retail activity and business vitality. Methods developed and used included community-level business market share "pull factors" and business survival rates developed using original source data made available by the Iowa Department of Revenue and Finance (IDRF); detailed "before and after" business profiles along access management project corridors; and detailed retail sales trends for selected businesses along access management project corridors. The results of this research indicate that for the great majority of businesses, access management projects are not detrimental. In fact, access-managed corridors studied in Iowa generally outperformed their larger communities in terms of business losses and retail sales growth.

PROBLEM STATEMENT

Access management has proven to be a very effective technique around the nation for improving both the safety and operations of arterial roadways. Like most other roadways, arterials must serve a dual function: moving through traffic and providing access to adjacent property and land development. Arterials should predominantly serve through traffic movement, however some have developed and been managed in such a way that extensive access to property has been provided, mainly via private driveways. When this happens, the result is often high rates of accidents related to turning movements, high rates of property damage and personal injury accidents, traffic congestion at peak hours, and motorist delays.

Recent case study research in Iowa indicates that applying access management principles to arterial roadways can greatly enhance their safety and operations. Accident rates per million vehicle-miles on a variety of roadways in Iowa declined by an average of 40 percent. This magnitude of decline is consistent with results from other studies on access management conducted elsewhere around the United States.

Opinion surveys of motorists in Iowa indicate that they perceive the positive impact that access management projects can have both in terms of safety and operations. Motorists believe that specific access management projects such as driveway consolidation, turn lane installation, and median construction improve traffic flow, ease of turning, and safety. The great majority of motorists (typically 90 to 95 percent) are also supportive of projects that have been completed.

The problem with access management projects both in Iowa and in the nation as a whole tends to come in terms of lack of support from businesses, in particular retail businesses adjacent to. Businesspersons are considerably less supportive of access management than motorists, who are also their customers. Businesspersons often equate reductions in the number of driveways and other direct access ways with loss of sales. They may actively oppose access management projects, particularly those that involve more restrictive treatments such as raised medians. Local businesspersons are often influential with local public officials. Even though local officials are typically supportive of most access management projects because of the safety and operational benefits, they can be swayed by vocal opposition from businesspersons. It is for this reason that the Iowa Department of Transportation and the Iowa Highway Research Board (IHRB) commissioned research on the actual impacts of access management projects on business vitality.

PREVIOUS RESEARCH

There is very little in the way of literature available on the business vitality impacts of access management projects. Most of the research that has been completed is from Florida and involves the

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impacts of the retrofit installation of raised medians on very high traffic arterials. A study completed in 1991 notes that "the majority of through travelers, residents, customers, and business owners favored the retrofit despite some inconveniences…(t)hese opinions after project development contrast with those set forth at two public hearings conducted by the Florida DOT during project development. These meetings were mainly attended by local merchants and residents who opposed the project" (*1*).

Another Florida study surveyed 180 drivers and 228 businesses along several major projects. The motorists were very supportive of the projects, however about 30 percent of the business owners felt that the projects had at least a small detrimental impact on their businesses (including sales declines, truck delivery difficulties and the like) (*2*).

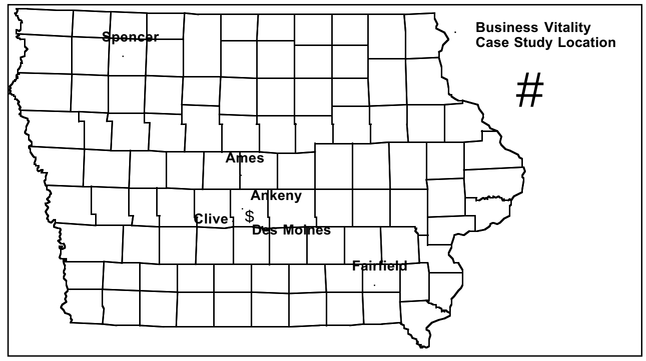

FIGURE 1 Locations of business vitality case studies.

DATA SOURCES AND METHODOLOGY

The Iowa Department of Revenue and Finance maintains an extensive historical database of retail sales tax payments by individual businesses in the state. This allows for quarterly or annual sales trends to be analyzed. IDRF allows these data to be used for research purposes provided that the confidentiality of data can be maintained for individual businesses. In the case of this study, sales tax trends were aggregated to the corridor level (by street address range) and on a quarterly basis to maintain confidentiality.

The sales tax database was used for two purposes. The first purpose involved creating a profile of the entire community's retail trade environment and characteristics. This involved assessing such factors as five-year business survival rates, retail sales growth trends, and retail trade "pull factors." In addition to the IDRF sales tax data, other secondary data that were used to assess impacts of access projects management on business vitality were from R.L. Polk City Directories. These are privately compiled directories that list businesses (and households) by street address. These directories were used to create a before and after project "snapshot" of the corridors and to assess turnover in the corridor.

RESULTS

This analysis examines business trends in five case study communities: Ames, Ankeny, Clive, Fairfield, and Spencer (Figure 1). Data on the change in total number of businesses, business composition, and retail sales activity levels are analyzed for each of the communities as well as the corridor in which access improvements were completed. Each of the case study communities is profiled below.

Ames is a large, regional hub for trade serving an area covering several counties. It has a population of slightly under 50,000 persons. A large percentage of the residents are college students at Iowa State University. Ames has experienced slow but steady growth in retail trade activity since the mid-1980s.

Ankeny is a rapidly growing suburb of Des Moines with nearly 20,000 residents. Along with rapid increases in population, it has experienced strong growth in retail trade since the late 1980s. The number of retail firms in Ankeny has roughly doubled since the mid-1980s with well over 500 currently.

Clive is also a suburban community of Des Moines with a population approaching 10,000. It has experienced very rapid growth in retail trade since the early 1990s and has been one of the fastest-growing communities in Iowa in terms of retail trade. Given current development trends, it is likely that the strong retail growth will continue in Clive over the next few years.

Fairfield is a mid-sized, rural community with about 10,000 persons located in the southeast part of Iowa. The city has experienced relatively stable levels of retail sales over the past two decades. The number of retail firms located in Fairfield has remained nearly constant at 400 to 450 for the past 10 years.

Spencer is another mid-sized community with about 11,000 residents with a geographic trade area that is considerably larger than Fairfield. Spencer is located in a less densely populated region of Iowa with no other, similar regional centers in close proximity. Like Fairfield, it has experienced stable inflation-adjusted retail sales since the mid-1980s.

The five business vitality case study communities represent a range of business activity levels, although all are generally prosperous and have maintained relatively stable business growth. In Iowa, only about 50 percent of businesses with sales tax permits survive over a five-year period. For the five business vitality case study communities, this percentage varied from 41 percent (Fairfield) to 54 percent (Spencer). Clive is a notable exception with a business survival rate of 64 percent. In other words, a business loss rate of 7 to 12 percent per year can be expected in almost any community in Iowa.

All of the case study communities, with the exception of Fairfield, have expanded their retail sales markets. In 1996, each of the case study communities had retail sales pull factors over 1.00. A pull factor of more than 1.00 indicates that a community is serving the retail needs of persons beyond those living in the local community. The pull factors for these communities range from 1.06 (Ankeny) to 1.71 (Clive). Ames, Ankeny, and Clive all experienced significant increases in retail sales pull factors between 1990 and 1996 (see Table 1). In terms of total retail sales activity for this period, all five case study communities experienced increases in total retail sales activity ranging from 5.5 percent (Spencer) to 346.2 percent (Clive). Clive has experienced explosive growth in business activity as a result of rapid development in the western Des Moines metropolitan area. The other four case study communities have had somewhat slower rates of growth, averaging approximately 1 to 10 percent annual growth (adjusted for inflation).

The previous results indicate that the business vitality case study communities have been successful retail markets over the past few

Table 1 Summary of Case Study Community Retail Business Trends

Community	Five Year Business Survival Rate	Five Year Change in Retail Sales	Five Year Change in Number of Retail Firms	Retail Sales Pull Factor 1990	Retail Sales Pull Factor 1996	Pull Factor % Change 1990-1996
Ames	44.6%	8.8%	2.1%	1.00	1.14	+14.0%
Ankeny	44.1%	57.2%	22.7%	0.86	1.06	+23.3%
Clive	63.7%	346.2%	171.0%	0.44	1.71	+388.6%
Fairfield	41.2%	7.0%	10.4%	1.20	1.16	-0.3%
Spencer	54.3%	5.5%	3.4%	1.56	1.57	+0.1%
State of Iowa	49.8%	—-	—-	1.00	1.00	——

Source: Iowa Department of Revenue and Finance.

Table 2 Difference in Current (1996) Business Composition: City Compared to Corridor

	Ames	Ankeny	Clive	Fairfield	Spencer
Utilities/Trans.	1.3%	1.6%	0.0%	2.7%	2.3%
Building Materials	-2.6%	-1.4%	0.0%	-5.5%	0.3%
General Merchandise	2.1%	0.5%	0.0%	-0.4%	1.2%
Food Dealers	-0.1%	-4.9%	0.0%	-5.5%	-2.5%
Motor Vehicle	-19.9%	0.6%	-5.1%	-28.7%	-16.0%
Apparel	4.3%	-2.1%	-7.1%	2.8%	1.9%
Home Furnishings	2.2%	3.8%	5.7%	3.6%	4.6%
Eating and Drinking	-20.1%	-3.2%	-10.0%	1.8%	-5.1%
Specialty Retail	14.6%	4.2%	-4.1%	12.0%	12.2%
Services	10.8%	-13.5%	6.6%	12.3%	-5.7%
Miscellaneous	5.1%	5.5%	7.5%	-1.8%	1.1%
Mobile Home Sales	na	na	na	na	na
Residential	na	na	na	na	na

Source: R.L. Polk Directories and Iowa Department of Revenue and Finance

Table 3 Change in Business Composition by Corridor

	Ames	Ankeny	Clive	Fairfield	Spencer
Utilities/Trans.	na	-100.0%	na	na	na
Building Materials	0.0%	-25.0%	na	200.0%	0.0%
General Merchandise	na	100.0%	na	0.0%	-100.0%
Food Dealers	-50.0%	66.7%	na	0.0%	0.0%
Motor Vehicle	10.0%	50.0%	100.0%	85.7%	-25.0%
Apparel	-100.0%	0.0%	0.0%	na	100.0%
Home Furnishings	0.0%	-100.0%	na	na	-100.0%
Eating and Drinking	40.0%	33.3%	66.7%	0.0%	0.0%
Specialty Retail	100.0%	50.0%	266.7%	400.0%	100.0%
Services	-42.9%	24.1%	11.1%	250.0%	-5.6%
Miscellaneous	-50.0%	100.0%	100.0%	25.0%	-55.6%
Mobile Home Sales	0.0%	na	na	0.0%	na
Residential	na	na	na	na	0.0%
TOTAL	-8.0%	21.3%	68.0%	85.7%	-13.7%

Source: R.L. Polk Directories

years. These trends can then be compared to business activities within each of the case study corridors.

As might be expected, the most frequent types of businesses in each of the corridors studied are services, eating and drinking, miscellaneous, automotive, and specialty retail—typical "commercial strip" businesses. These categories range from 10 to 45 percent of the businesses in each corridor. By comparison, services, specialty retail, and miscellaneous businesses are the most prevalent in each communities as a whole. In general, the corridors differ from their community business composition primarily in the proportion of motor vehicle, eating and drinking, and to a degree, specialty retail and service establishments. Corridors tend to have a greater share of auto-related and restaurant establishments. Corridors also tend to have a smaller share of specialty retail and services compared to their communities (Table 2). Such businesses might instead be located within shopping malls or in downtown areas. Because there are not dramatic differences in overall composition of businesses, changes in business activity for each corridor should be comparable to changes in business activity for the community in which they are located.

In addition to analyzing the changes in total business activities within the case study corridors, the changes in specific categories of businesses are also of interest. Impacts of access modifications can have different affects on different types of businesses as well as on overall business activity. A review of the five case study corridors does not indicate a consistent pattern of business composition changes. This means that there does not appear to be a proportionately larger impact of access management projects on one type of business compared to another in these areas.

There were no particular business categories that consistently decreased in number of locations for the case study corridor areas (Table 3). Home furnishings, services, and miscellaneous were the only business types to decrease in number of establishments in more than one corridor for the periods analyzed (each decreased in two corridors). The loss of the home furnishings, services, and miscellaneous businesses (a total of 18 for all case study corridors combined) did not have a significant impact on total business numbers, with the total number of businesses increasing an average of approximately 20 percent for each of the five corridors. It should be noted here that some of the business categories listed have high thresholds for success. For example, a large, new home furnishings establishment opening in a nearby community could easily disrupt local home furnishing sales, resulting in a nearly 100 percent loss of this type of establishment to the local business mix.

The business changes previously discussed represent net changes in number of businesses. The composition of the current stock of businesses in each corridor is the result of businesses existing before and after access improvements, new businesses (or name changes), and loss of businesses (or name changes). Because the business information was collected from R.L. Polk directories, the difference between name changes and new or lost businesses was not always clear. For this reason, renamed businesses were counted as losses. Ames and Spencer had the highest rates of remaining businesses (67 and 64 percent respectively). Ankeny, Clive, and Fairfield had the highest rates of new business locations (61, 67, and 54 percent respectively). The rates of business losses or turnover ranged from 13 percent for Fairfield to 50 percent for Spencer over a five-year period, with Ames, Ankeny, and Clive all between 20 and 35 percent. This equates to approximately a 2.6 to 10.0 percent annual turnover of businesses. A typical Iowa community will experience anywhere from a 5 to 15 percent annual change (with 10 percent being the statewide average), so these rates can be viewed as indicating stable and typical business environments. In fact, only in Spencer did the loss rate for the corridor (at 50 percent) equal or exceed that for the community as a whole (46 percent).

Table 4 Sales Activity for Corridors and Communities

Year	Corridor Sales	Community Sales	Corridor Index	Community Index	
Ames					
1991	15,068,900	384,328,804	100.0%	100.0%	
1991	13,445,019	388,862,320	89.2%	101.2%	
1992	14,215,046	388,774,727	94.3%	101.2%	
1993	14,393,570	399,426,817	95.5%	103.9%	
1994	13,846,263	402,207,192	91.9%	104.7%	Project completed Fall 1994
1995	14,693,133	418,148,170	97.5%	108.8%	
1996	15,798,306	na	104.8%	na	
Ankeny					
1990	8,211,100	116,564,938	100.0%	100.0%	
1991	10,132,321	128,618,282	123.4%	110.3%	
1992	13,989,674	136,846,533	170.4%	117.4%	
1993	13,456,890	137,075,948	163.9%	117.6%	Project completed Fall 1993
1994	16,492,770	154,852,588	200.9%	132.8%	
1995	17,067,945	183,212,866	207.9%	157.2%	
1996	18,595,836	na	226.5%	na	
Clive					
1990	6,478,100	24,020,089	100.0%	100.0%	
1991	6,745,108	49,518,974	104.1%	206.2%	Project completed Fall 1991
1992	8,800,195	66,399,337	135.8%	276.4%	
1993	9,564,852	74,415,863	147.6%	309.8%	
1994	10,773,843	103,397,595	166.3%	430.5%	
1995	10,890,861	107,170,336	168.1%	446.2%	
1996	11,588,433	na	178.9%	na	
Fairfield					
1990	18,261,350	86,837,886	100.0%	100.0%	
1991	17,354,959	83,458,500	95.0%	96.1%	
1992	17,549,619	83,106,800	96.1%	95.7%	Project completed Fall 1992
1993	17,840,378	87,605,920	97.7%	100.9%	
1994	17,776,399	88,784,797	97.3%	102.2%	
1995	19,211,631	92,891,667	105.2%	107.0%	
1996	20,720,973	na	113.5%	na	
Spencer					
1990	4,583,850	129,098,725	100.0%	100.0%	
1991	4,521,337	126,634,993	98.6%	98.1%	
1992	4,130,907	123,987,367	90.1%	96.0%	Project completed Fall 1992
1993	5,158,390	126,038,810	112.5%	97.6%	
1994	4,976,689	132,828,612	108.6%	102.9%	
1995	5,834,589	136,202,113	127.3%	105.5%	
1996	6,027,244	na	131.5%	na	

Notes: Years are State of Iowa Fiscal Years, 7/1-6/30. Figures adjusted to 1990 dollars. 1996 community sales were not available at the time the research was completed.
Source: Iowa Department of Revenue and Finance.

Further analysis used disaggregate retail sales data for each of the five case study corridors. The Iowa Department of Revenue and Finance was able to report sales tax summaries for the street addresses which fell within a project corridor. (Breakdowns by business type were not possible for any corridor because of data confidentiality rules). From 1990 to 1995 retail sales activity increased within each of the five business vitality case study communities. Inflation adjusted annual increases ranged from 0.9 percent (Spencer) to 57.7 percent (Clive) (see Table 4). At the same time, all of the case study corridors experienced retail sales growth between 1990 and 1996. The annual corridor increases ranged from 0.7 percent (Ames) to 18.1 percent (Ankeny). Overall, the average annual sales growth rate for the five corridors was 7.3 percent compared to an average of 14.1 percent for the communities. However, excluding Clive, the average rate of growth for communities was 3.3 percent. Clive has experienced such phenomenal retail sales growth that its sales figures distort the average of the five communities. With this in mind, the results suggest that on average, the corridors have out-paced the communities in terms of retail sales activity over the past seven years by some 15 to 20 percent (see Figure 2).

To examine the short-term impacts of the access management projects, this analysis looked at changes in sales activity from the year before and the year after access improvements were completed. In each of the five case study corridors, sales activity increased the year after the projects were completed. Retail sales activity increases ranged from 1.6 percent (Fairfield) to 37.0 percent (Ankeny) with a 19.7 percent average rate of increase. The average for the corridors was slightly higher than the 'before and after' average changes for the communities (19.29). The average corridor sales activity increases are even more dramatic if Clive is excluded from the community average, with the comparison then becoming 19.7 percent (corridors) to 6.5 percent (communities).

CONCLUSIONS

The results of this effort with secondary data as well as opinion surveys conducted during the research project suggest that access management projects have a far less detrimental impact on retail businesses than business owners and managers initially believe. Business failure rates in corridors where access management projects have been constructed were almost always lower than those of their surrounding communities or that of the state of Iowa as a whole. Further, retail sales growth in access-managed corridors significantly exceeded that in surrounding communities. It is simply not possible to say that access management projects have a large, detrimental impact on local retail businesses. These findings were confirmed by the Iowa opinion survey results, which indicated that only a small minority (5 percent) of business owners in the case study corridors felt their sales had been negatively impacted by the access management projects.

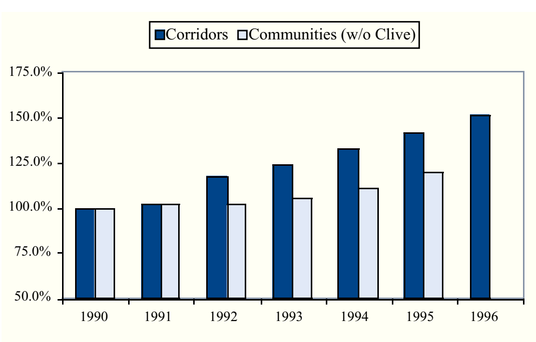

FIGURE 2 Retail sales activity comparisons.

SUGGESTIONS FOR FURTHER RESEARCH

In any experimental design, the ideal situation is to have only one variable. In the case of the Iowa corridor studies, this sort of approach was not followed. Case studies were selected to explore and illustrate the impact of different. A suggestion for further research would be to select several pairs of similar cases where one arterial roadway has had access management improvements made and another has not and then assess business vitality trends. This would create a more rigorous experimental design. A reasonable working hypothesis would be that access management projects do not adversely impact business vitality. One potential problem with this approach, however, is that corridors with the worst access-related safety problems tend to be programmed for improvements. Further, the universe of available corridors in Iowa (both improved and not improved) is rather limited. The total number of corridors originally considered suitable for analysis at the beginning of the Iowa research project was only about 50.

There is also some anecdotal evidence based on several project corridors in Iowa that access management projects, once successful, may lead to land redevelopment along the corridor. This phenomenon is particularly noticeable in the cases located in growing metropolitan areas. It may be worthwhile to track land development, sales trends, and property valuations over a longer period of time for these corridors.

ACKNOWLEDGMENTS

The Iowa Department of Revenue and Finance greatly assisted with this research by providing access to an compiling retail sales tax data for the access management case study corridors. The Iowa Highway Research Board and the Iowa Department of Transportation provided the funding for the project. The Engineering Division of the Iowa Department of Transportation and the Iowa Access Management Task Force provided guidance to the project.

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Bridging the Gap Between Access Management Ideals and Land Use Planning Practice: Suggested Policies and Potential Benefits

CHRISTOPHER P. ALBRECHT AND DAVID PLAZAK

Access management has become an increasingly important and controversial issue for the state of Iowa and other states across the nation. Implementing access management can be beneficial in several ways, including improved traffic safety and operations, higher quality corridor development, and the avoidance of more expensive and damaging methods of capacity improvement. One of the major obstacles to the successful implementation of access management principles is the significant disconnect between the agencies that administer roadways and those responsible for local land use planning and regulation. Bridging this gap is clearly a key to safer and better functioning roadways. Making this connection, though, has proven to be a difficult task. The solution involves, among other things, improved communication between road jurisdictions and local planning agencies. A first step toward improved communication is the identification of current "best practices" as models for localities in Iowa to use. Next, a process for identifying future transportation and access conflicts is necessary. Such locations could then be designated as priority areas for increased interaction and coordinated planning. Finally, designating the best possible treatments to apply on such access projects could be accomplished through a more systematic approach. Bridging the gap between access management and land use can produce significant benefits in the success of long-range land use planning, as well as in the functioning and safety of transportation facilities. These benefits are applicable to areas within Iowa, as well as in other parts of the nation.

ACCESS MANAGEMENT BASICS

One of the most difficult problems in roadway administration and design today is balancing the dual function that many roads have: serving through traffic while providing access to property. Providing inappropriate or excessive access to property on arterial roadways can lead to increases in crashes, delays, and traffic congestion. According to the Federal Highway Administration:

"Access Management is the process that provides access to land development while simultaneously preserving the flow of traffic on the surrounding road system in terms of safety, capacity, and speed. It attempts to balance the need to provide good mobility for through traffic with the requirements for reasonable access to adjacent land uses" (*1*).

Access management involves carefully planning direct access from property to adjacent roadways. By doing this, the number of conflict points that lead to opportunities for accidents are reduced. Therefore, access management projects are designed to serve one main purpose: reducing conflict points. Six general types of treatments or retrofit projects are most commonly used in the state of Iowa. These six major treatments are often used in combination with each other and along with other roadway improvements to help manage access along arterial roadways:

- Driveway consolidation: Designed to limit the number of driveways per mile and provide adequate spacing between driveways.
- Corner clearance: Involves removing or relocating drives away from intersections or to better locations on side streets.
- Continuous two-way left-turn lanes: Adds dedicated turning lanes in the center of a three or five-lane roadway to separate left-turns from through traffic.
- Alternate access ways (frontage or backage roads): Adds alternate roads off the main traveled roadway that function to separate turning and through traffic completely.
- Raised medians at intersections: Provides for limited installation of barriers near intersections that prevent some turning movements near intersections.
- Full raised medians: Add barriers in the center of a major roadway that prevent left-turns and cross traffic and eliminating a considerable number of conflict points.

BENEFITS OF ACCESS MANAGEMENT

Implementing the principal techniques of access management can be beneficial in several ways. First of all, access management can have significant positive impacts in terms of safety, congestion reduction, and lessened delay (*2*). In addition, there are numerous secondary benefits that can be accrued through improved management of access. Among other benefits, the use of access management techniques may ultimately promote corridor redevelopment. These basic access management principles can also be effectively used to avoid more expensive and environmentally damaging methods of capacity improvements. There are also possible indirect benefits resulting from access projects. Such benefits may include

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the overall investment in transportation infrastructure and lessened auto emissions due to delay and congestion.

In the state of Iowa, a number of these benefits have recently been quantified through a joint study conducted by the Iowa Department of Transportation, the Access Management Task Force, and the Center for Transportation Research and Education (CTRE). The Access Management Task Force, made up of private and public sector representatives, worked cooperatively with the other entities to develop a work plan that would guide research, education, and outreach activities related to access management. Phase I of this study, called the Iowa Access Management Awareness Program, resulted in the publication of two reports. First, a literature review covered the relevant access management sources to assist in refining the research agenda (*3*). Secondly, a report was developed on current access management policies and practices in the State of Iowa (*4*). This second report concluded that there is a significant disconnect in Iowa between roadway administration (e.g. driveway permitting and control of access rights) and land use regulation (e.g. master planning and local zoning). The integration of this vital transportation issue and land use planning was one of the major reasons behind the Iowa program. The principal potential impacts of access management were divided into three categories in the literature review: traffic safety, traffic operations, and business vitality. These topics were addressed in a report on Iowa case studies with an emphasis on safety and business vitality, since traffic congestion is not a serious problem in most places in Iowa. Results from the case studies indicate that (*2*):

- A typical access management project in Iowa may reduce annual accident rates between 10 and 70 percent. The average reduction in accidents for all projects is around 40 percent. Both personal injury and property damage only accidents are reduced significantly.
- Access management projects raised the level of traffic service to motorists during the peak hour along a corridor by one level, providing significant benefits in terms of increased operating speed and reduced traffic congestion.
- Access projects are often controversial among local businesses because of a perceived potential for lost business. However, corridors where projects have been completed actually perform better in terms of retail sales than their surrounding communities. This indicates that access management projects generally do not have an adverse effect on the majority of businesses. Some individual businesses may be affected, however the number is small.
- Around 80 percent of businesses reported sales at least as high after the project was in place. Few businesses reported declines associated with the projects. Similarly, about 80 percent of businesses reported no customer complaints regarding access to their businesses after projects. The businesses that report complaints are highly oriented toward drive-by traffic.
- Motorist opinions of the access management projects studied were highly positive. In all cases, 90 to 100 percent of motorists surveyed had a favorable opinion of access improvements made. The vast majority of motorists agreed that the improved roadways are safer, operate better, and are easier to drive on.

The result of the business vitality related analysis points to another important aspect of access management. Access management can have a very positive impact on overall corridor development. As a secondary benefit of improved traffic flow and safety, access management can promote a healthier business climate along an improved corridor. Some evidence from the Iowa projects shows that business re-development, investment, and revitalization begin to occur a few years after access management projects are completed. Another important benefit of access management is the reduced cost of new roadway construction. Today, we are seeing a paradigm shift in how we approach our transportation system. As our system has matured to its current level, we have turned to updating, managing, and improving the current network instead of more new construction. While construction costs are rising, access management projects are much less costly. They also reduce the need for displacement of businesses and homes often necessary with major widening projects. Some of the cost of additional right-of-way may also be avoided, although not entirely. Because of these factors, access management techniques and projects are becoming more attractive to transportation officials.

Access management can also potentially promote less environmental impact and more sustainable development through its positive effect on the movement of many different types of transportation modes. First of all, alternative forms of transportation will also benefit from safety and decreases in delay. Public bus transportation, for example, can directly benefit from reduced travel time, congestion, and delay. For public transportation, reduction in travel time is a key factor in increasing ridership. Bicycles and pedestrians can also benefit in some ways from access projects. In a corridor that has been improved, cyclists face fewer decisions and conflict points as well. With access improvements, bicyclists would have to deal with fewer driveway locations and turning movements and consequently more predictable motorist travel patterns (*5*). In addition, these projects can have a lesser effect on the environment than major new construction or other reconstruction.

Finally, access management can ultimately be an effective tool in protecting the public investment. Access management is a method of promoting a maximum return on the highway system (*6*). Ultimately, all these potential benefits resulting from sound access management techniques can have a significant impact in numerous areas. From the direct traffic and safety benefits to the potential reduction in automobile emissions, the benefits of sound access management are extensive.

CURRENT PROBLEMS

Assuming that access management is a beneficial activity in terms of safety, traffic operations, and other areas, there are certain barriers that remain. These barriers are what keep transportation programming and land use planning apart. The survey of Iowa communities mentioned earlier identified this gap, but at the same time, the survey concluded that city and county officials in Iowa see access management as an increasingly important issue and as a high priority for action (*4*). The survey of Iowa cities and counties also found four fundamental legal aspects of access management in Iowa. First of all, access management is exerted initially at the state level through enabling legislation. Secondly, state's enabling legislation can dictate the level of power given to local jurisdictions. In Iowa there is significant latitude given to counties and cities in terms of access management. Thirdly, there are definite legal implications of managing access. A "taking" may potentially result from insufficient compensation to private property owners for using their land for a public purpose (*4*). Lastly though, land use planning techniques can be used to promote proper access management and provide consistency in legal and regulatory practices.

In most cases in Iowa, larger city and county governments do have ordinances pertaining to some aspect of access management;

however, these ordinances have little direct relationship to an overall transportation plan. Most local ordinances in Iowa do not fully utilize the powers granted by the Code of Iowa for controlling access. There is also no consistent process used by roadway jurisdictions to review local access-related changes to roadways, except at the point when driveway permits are being sought by landowners (*4*). At this point, the involvement is very often too late. These transportation plans must incorporate access management as an integral part of the process. This can only be realized by the involvement of land developers, local planners, and state transportation planners as early in the planning process as possible.

In Iowa, there are also very few examples of a community incorporating access management elements within their comprehensive planning activities, but this is clearly another way in which access ideals can be used at the local level (*4*). Other local planning methods such as land division techniques, subdivision regulations, permitting of access locations, driveway spacing requirements, and overlay zoning, are possible ways to supplement access management strategies. The environment for applying access management principles is similar in other states to that in Iowa. Therefore, coordination of access management policies among local agencies and other agencies involved is the key challenge for most states wanting to administer access management programs.

In order for access management activity to be viable and acceptable to potential developers, they and the public should also be involved at the earliest possible stages. Transportation planners and engineers must work together and consider impacts of access decisions on the surrounding land use and development. Too often in the past, it has been an afterthought or no thought at all. Therefore, it is necessary to integrate all of these areas and develop a system which will use input from several important and often overlooked sources, and include these important players earlier in the planning process. This does make conflicts and legal challenges less likely, though there have been few legal challenges to create case law in the state of Iowa to date.

A NEW APPROACH

The apparent disconnect between transportation ideals and practice is an obvious problem. There are several potential and important ingredients to building a solution to this problem. All of these elements must, most importantly, involve improving communication between all parties involved. This new approach, integrating many ideals learned from studying access projects and the issues that surround them, can be formulated into a simple process involving three main steps. This new integrated transportation planning process would include more well-defined steps and more extensive involvement by all those affected by any access-related decision.

The first, and very crucial, step in approaching access management involves the strengthening of the transportation planning process itself. Once again, improved communication and involvement at every level of the process is vital. In a few places there do exist processes that work well and provide sufficient involvement (*7*). The first action, which is currently underway in the state of Iowa, is the identification of such "best practices" and the subsequent integration of these elements within local land use practice. In other words, these "best practices" may be used as models for other localities in Iowa to use. Through projects such as the Iowa Access Management Research and Awareness Program parties at every level of the process are being educated about the potential benefits and subsequent need for good access management techniques through an outreach effort. Specifically, a new set of educational materials is being designed in Iowa to demonstrate the benefits of coordinated access management to both local land use planners, government officials, transportation engineers and planners and local officials.

The next major step in improving access problems statewide is to identify the major problem areas. These are the roadway sections that are most likely to benefit from the principles of access management. In the past, most major access projects in Iowa have been accomplished through the Transportation Safety Improvement Program of the Iowa Department of Transportation. This safety-driven approach was understandable given the substantial safety benefits of access management. However, this approach downplayed the other potential benefits, and was primarily reactive rather than proactive. A more systematic and logical approach to identifying access problem areas is needed. This new approach must create a typology of practices to match best practices with community attributes. Such a systematic process for identifying such problem areas could be most readily implemented through the use of Geographic Information Systems (GIS) technology. GIS has the ability to analyze several different sets of information spatially to determine the potential relationships between factors such as driveway placement and traffic volumes.

This GIS-based approach would involve the integration of several data sets, as well as the expertise and ideas of those professionals most involved with handling access management issues. A delphi method approach could be utilized to assemble ideas and to create a basis for weighting the importance of the multitude of factors which affect the functioning and safety of roadways. It is first necessary to identify, through interviews and case study analysis, which potential factors and subsequent data elements are most important in locating potential access-related problem locations. In other words, it is necessary to determine which factors contribute the most to access problems, and to what extent. Base record data is available for many of the different factors involved in "creating" an access problem. Data sets to be considered in this step are traffic volumes, roadway classification, speed limit, current land use, future land use, access points per mile, and number of lanes of through traffic.

The next logical action would be to query a variety of these relevant data sets to make an initial identification of potential problem roadways. Data elements that would be queried at this initial stage include roadway classification and traffic volume (AADT). Roadways such as local streets with very low daily traffic volumes and completely access controlled interstate highways would be eliminated at this step. Both of these roadway types fall outside the realm of where roadway function and need for access conflict (*4*). The relationship between traffic movement and property access is shown in Figure 1.

The next action would be to query these most likely problem roadway links through further data elements; such as number of accesses/egresses, roadway section length, number of lanes of traffic, and current and future land use. Combining these first two data elements would provide a rough estimate of access points per mile. With this knowledge and the number of lanes, a rough estimate of overall conflict points could be established. This estimate would provide a useful index of potential access management problems. Further iterations involving specific levels of traffic would give more specific ideas of potential locations of serious conflict prob-

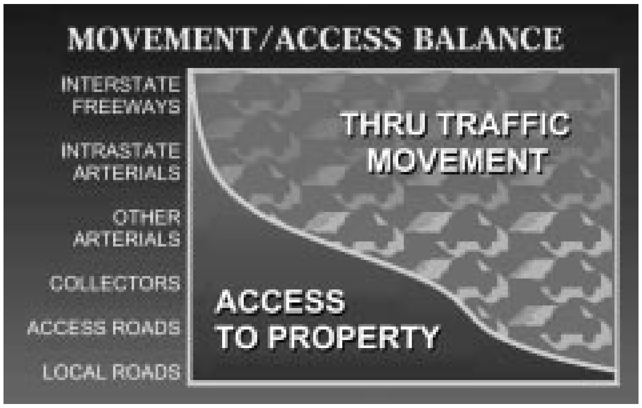

Figure 1 Relationship between traffic movement and access.

lems. Other data elements, such as current and future land use developments and access-related accident data from GIS-ALAS could also potentially be used for the identification of future access problem areas. This approach, using GIS based tools, would provide a more systematic way for a state or other agency to focus on the most serious potential access problems. This method could then be instrumental in identifying where future transportation and land access conflicts have or are most likely to arise statewide, such that a state Department of Transportation could designate them as high priority areas for increased interaction and coordinated planning.

The third major step in the implementation of access management ideals is the actual application of the best possible methods or treatments for managing access at a specific location. Fitting the best techniques to specific situations is an important part of the process. Through research and data collection into successful and unsuccessful uses of access management techniques, general rules of thumb can be identified. One approach to this step would be the implementation of an "expert system" to model a procedure for determining the preferred access management applications at specific locations identified by the previous step.

CONCLUSION

Bridging the gap between access management transportation planning ideals and actual land use practices can produce significant benefits in the success of long-range land use planning as well as the functioning and safety of transportation facilities, in addition to numerous secondary indirect benefits. The process for bridging this gap involves the interdisciplinary technical expertise, improved communication, cooperation of all parties involved in the transportation planning process, and the application of information technology to identify problem areas as early as possible and suggest the best possible solutions.

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The Impact of Factoring Traffic Counts for Daily and Monthly Variation in Reducing Sample Counting Error

SAM GRANATO

Transportation agencies often determine what the annual average daily traffic (AADT) count is on streets and highways by counting traffic for short time periods (usually for 24 hours) and then estimating the AADT based on this count and a numerical factor that takes into account day-of-week and/or seasonal variations in traffic volumes found at a small number of permanent automatic traffic recording stations (ATR's). Considerable research has been devoted to help state departments of transportation (DOTs) and other agencies develop cost-effective programs to develop factoring procedures to ensure reasonably accurate estimates of AADT from short-term counts. The U.S. DOT has also published estimates of sample error as a function of the volume in an *un*factored count. However, no recent research has been found that provides answers or guidance as to how much (sampling) error remains in the estimation of AADT from a *factored* short-term count in urban areas. Such research is necessary to help agencies determine whether changes in counted volume over time represent a significant change in traffic flow or not, for how long a period of time should a count be taken to reach a desired level of confidence in the count, and to help develop a standard for traffic forecasting model performance regarding the minimization of discrepancies between counted and modeled traffic flows. This paper presents an analysis of just how much day of week/month of year factors can reduce the error of prediction of AADT from a short-term traffic count, utilizing data from an ATR station maintained by the Iowa DOT in Cedar Rapids, Iowa. The benefits of factoring are shown to be a one-quarter reduction in error of AADT prediction for a 24-hour count at this station, with minimal added benefit of a (consecutive) multiple-day count. The metropolitan planning agency will utilize these findings in future evaluations of forecasting model performance. Key words: traffic counts, count factoring, traffic model calibration.

OVERVIEW

Traffic and transportation agencies often estimate the annual average daily traffic (AADT) count on streets and highways by counting traffic for short time periods (usually 24-48 hours) and then making adjustments based on numerical factors that takes into account the day of week and/or seasonal variations in traffic volumes found at a small number of permanent automatic traffic recording stations (ATR's).

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Research has been devoted in the recent past to help state departments of transportation (DOTs) and other agencies that prepare traffic counts develop a cost-effective program of stratified permanent count stations that minimize AADT estimation error from short-term counts (*1*,*2*), with findings stressing the importance of length of counting time and proper clustering of ATRs into factor groups. The author has found no recent formal research, however, that determines for the end user of traffic data what the value of the factoring process is in reducing sample traffic count error in urban areas (and the results of older research has been called into question[*3*]). Such research is necessary for at least three reasons:

1. To help agencies determine whether changes in counted traffic volume over time actually constitute a significant change or not;
2. For how long a period of time should a count be taken to reach a desired level of confidence in the count;
3. To help planning agencies develop a standard for traffic forecasting model performance (regarding the minimization of discrepancies between counted and modeled traffic flows).

The U.S. DOT has published estimates of sample error as a function of the volume in an *un*factored 24-hour count (*4*), which has been incorporated into at least two "how-to" traffic forecasting manuals (*5*,*6*). However, it is not known if any planning agencies other than the Cedar Rapids metropolitan planning organization make use of this information in assessing the level of accuracy of their (base-year) traffic forecasts.

Cedar Rapids is a Midwestern city of about 110,000 people (metro area 150,000) with an employment base dominated by agribusiness, avionics, and telecommunications. While it is a significant employment and shopping destination for the surrounding rural area, it is not deemed a significant tourist destination, has no major universities, and does not host many "big-draw" special events (major league sports, festivals, etc.). Therefore, traffic patterns can be considered relatively stable for an urban area.

Traffic counts on city streets are conducted every two years by the Cedar Rapids Traffic Engineering Department (TED) at close to 1000 locations. This is done by leaving a mechanical counter at a street location for (typically) 24 hours during a weekday, than adjusting the count based on day of week/month of year factors supplied by the Iowa Department of Transportation (IDOT). This factoring is done based on a network of 124 ATR stations maintained by IDOT around the state, grouped into six categories based on type of roadway (interstate, primary road, local street) and surrounding environment (municipal or rural) (*7*). The counts are used locally to determine travel trends, calculate accident rates for intersections and "mid-block" street sections, and calibrate traffic simulation and forecasting models. The author has previously reported

TABLE 1 Johnson Avenue Daily Traffic Count Data And Variability

Year	AADT	Number of Daily counts Used	Average deviation from AADT: Unfactored	"Internal" factors*	Published factors**	Percent reduction in error: "Internal" factors*	Published factors**
1991	9,673	72	6.1%	2.0%	6.2%	67	-2
1992	9,728	67	8.1%	2.3%	3.2%	72	60
1993	10,493	86	13.1%	3.2%	11.1%	76	15
1994	10,091	89	8.6%	2.9%	6.4%	66	26
Avg.	9,996	79	9.0%	2.6%	6.7%	71	25

* Factored for count site's "intra-year" day of week/month of year variation.
** Based on published factors for "municipal street composite," based on the previous three year's worth of traffic data from 12 ATR stations in the state of Iowa.

TABLE 2 Johnson Avenue Peak Hour Traffic Count Data And Variability

Year	AAHT	Number of Daily counts Used	Average deviation from AAHT: Unfactored	"Internal" factors*	Published factors**	Percent reduction in error: "Internal" factors*	Published factors**
AM peak hour (7-8 a.m.)							
1993	571	86	32.9%	4.1%	24.9%	88	24
PM peak hour (4-5 p.m.)							
1994	869	86	14.3%	3.7%	9.5%	74	34

* Factored for count site's "intra-year" day of week/month of year variation.
** Based on published factors for "municipal street composite," based on the previous three year's worth of traffic data from 12 ATR stations in the state of Iowa.

that traffic forecasting model error in the metro area is not significantly different from sample counting error for count locations over 15,000 AADT (*8*), but this claim is based on published U.S. DOT research for unfactored counts (*4*). The purpose of this paper is to determine, at least for one urban area, how to adjust standardized estimates of traffic count error based on the factoring done locally. It is *not* the purpose of this paper to question IDOT's current count factoring methodology or suggest alternatives.

ANALYSIS

The case study described in this paper is based on an analysis of four year's worth of daily traffic counts from one ATR station located on a local arterial street (Johnson Avenue) in a residential area in the city of Cedar Rapids. The Johnson Avenue site is one of 12 ATR stations around the state classified as a "municipal street" station used to factor counts taken on any city street. "Municipal street composite" factors are made available annually to local agencies that reflect traffic patterns at these 12 locations during the three previous calendar years. The Johnson Avenue site carries about 10% of the combined volume of this group of stations (*7*). Therefore, the factors can be said to be somewhat, but not completely, independent of the traffic counts collected at Johnson Avenue.

For each of four consecutive years (1991-1994), traffic counts for individual days and their variation from the average count for that year were compared to the same counts factored for the statewide municipal street patterns and the remaining variation in those counts. Days of the year excluded from consideration are days where the local agency is not likely to conduct counts, days where traffic patterns are impacted by holiday travel, and days where IDOT has indicated that a count on a particular day at the site is an "estimate" due to maintenance or mechanical problems with the ATR. TED only conducts counts on weekdays from April to October. Therefore, as shown in Table 1, the average number of days in a year that ATR counts are considered useful for this study is about 80. The AADT at this location from 1991 to 1994, as published by IDOT, is about 10,000 with a coefficient of variation of about 4% (*7*).

As shown in Table 1, the impact of factoring counts varied widely among the four years, but has an overall average of 25%. (This is less than findings from comparable research for rural roads [*9*] which found an error reduction of 32% on Interstates and 37% on other roads.) When one looks at day of week and month of year variation *within* the April-October counting season at the Johnson Avenue site, it is apparent that most of the variation in traffic volume (about 70%) is due quite literally to day of week and monthly patterns. However, most of this potential gain in the accuracy of AADT

TABLE 3 Johnson Avenue Multi-Day Traffic Count Data And Variability

Year	AADT	Number of Multi-day Counts Used 48-hour	72-hour	96-hour	Average deviation from AADT (factored)* 48-hour	72-hour	96-hour
1991	9,673	53	34	15	6.1%	6.2%	6.2%
1992	9,728	65	41	20	2.1%	2.0%	2.0%
1993	10,493	63	40	19	10.9%	10.8%	11.3%
1994	10,091	66	43	21	6.2%	6.2%	6.3%
Avg.	9,996	62	40	19	6.3%	6.3%	6.4%

* Based on published factors for "municipal street composite," based on the previous three year's worth of traffic data from 12 ATR stations in the state of Iowa.

TABLE 4 Sample Count Error Compared to Traffic Forecasting Error

Traffic AADT Range	Number of counts*	Forecasting error Avg.	Std. Dev.	Sample Count Error Unfactored (4)	Factored	Forecast error significantly different? (95% Conf. Int.) Unfactored (4)	Factored
0-5000	340	45.6%	40.6%	19.9%	14.9%	Yes	Yes
5-10,000	236	26.6%	19.6%	15.0%	11.2%	Yes	Yes
10-15,000	84	14.9%	10.6%	12.4%	9.3%	Yes	Yes
15-20,000	42	11.9%	11.0%	10.7%	8.0%	No	Yes
20-30,000	33	11.6%	7.5%	9.1%	6.8%	No	Yes
TOTAL	735	32.5%	32.8%	16.4%	12.3%	Yes	Yes

* On the modeled major street network in the year 1990.

prediction is lost due to the fact that the published factors introduce errors in space (use of other count locations with different traffic patterns) and time (use of the previous three years of traffic patterns rather than the current year). A separate analysis of one year's worth of peak hour counts in 1993 (Table 2) indicates that while the factors have a comparable impact on reducing the error of peak hour traffic estimation, AM peak hour counts have considerably higher variability than PM peak hour counts - which is consistent with previous research findings (*10,11*).

Also significant, as shown in Table 3, is that longer count periods - in the form of 48, 72 or 96 consecutive hour counts during the weekday - were found to make only a 5% improvement in the accuracy of AADT estimation (with factoring). This is consistent with previous research findings in which count days scattered across two or more weeks in a counting season are recommended instead of a focus on any one particular week of the year (*3*). However, this recommendation would prove more costly to agencies conducting such multi-day counts than use of consecutive-day counts.

Finally, the author's previous claim of traffic forecasting model error not being significantly different from sample count error locally is reviewed. As shown in Table 4, while modeling error was within the 95% confidence interval at relatively high-volume count locations (AADT greater than 15,000) based on the U.S. DOT's published estimate of error for unfactored counts (*4*), an assumed 25% reduction in count error due to factoring means that model error is now considered significantly higher than count error at all levels of traffic volume.

CONCLUSIONS

Due to local economic factors and the method used to factor counts locally, it would stand to reason that appreciable gains in count accuracy could be achieved by factoring to account for the day of the week and month of the year, as is the current practice in Iowa. The findings from this paper show that this is not necessarily the case - even if the day of the week and the month of the year can literally explain most variations in traffic counts. This finding is due to spatial (use of factors developed from different locations than the short-term count site) and temporal (year the short-term count was taken versus years of the ATR traffic data used to develop the factors) considerations. This finding may be disappointing to those who expect more from the count factoring process, but important for "end user" agencies to keep in mind when local count programs reach counter-intuitive conclusions and are called into question by policymakers or other recipients of traffic count information.

While it is customary to say that further research is always needed in the topic of discussion, it needs to be stressed that only one count location was utilized in this study. From a research perspective, this and the unexplained year-to-year variability in the impact of factoring the daily counts are the obvious weaknesses of this study. While this location may not have an atypical traffic pattern relative to the rest of the state or the country, other agencies interested in determining how accurate their sample traffic counts are likely to be (even with factoring) should do as much background work as they can in determining any local variations to statewide or regional traffic patterns. The use of only one ATR station in this study is a function of both the limited geographic scope of interest to the author, and the limited number of ATR count sites deemed relevant within this scope. The study method is a base from which others may want to make improvements.

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A Summary of the Economic Analysis Concerning the Application of Intelligent Transportation Systems/Commercial Vehicle Operations (ITS/CVO) to the Mid-Continent Corridor

CHRISTOPHER M. MONSERE AND T. H. MAZE

The existing operational procedures of motor carriers and state enforcement agencies have potentially substantial benefits to gain by integrating Intelligent Transportation System technology into their commercial vehicle operations (ITS/CVO). In particular, multi-state integration of these technologies throughout a transportation corridor will lead to more significant benefits then single state applications. This paper summarizes the results of an economic analysis on the application of three principal ITS/CVO technologies to the mid continent corridor: electronic credentialing, electronic screening and electronic border crossings. The mid-continent corridor is defined as Interstate Highway 35 (I-35) from Duluth, Minnesota to Laredo, Texas. The analysis corridor includes the international border crossing at the Laredo. Current baseline data for weigh station, permitting, credentialing, and international border crossing activities were collected for the corridor states. Additional data were collected on accidents and overweight citations in the vicinity of weigh stations for the Iowa segment of the corridor. A methodology was developed to identify and estimate the potential benefits and costs relating to the implementation of electronic credentialing, electronic screening and electronic border crossing technologies in the corridor states. Benefits and costs were estimated on a corridor wide basis for a ten year analysis period. Total benefits of electronic credentialing ranged from $25 to $50 million and total costs were estimated at $19 to $38 million. Benefits from the implementation of electronic screening were estimated at $49 to $59 million, while costs were estimated at $7 to $23 million. Benefits at the international border were significantly higher at $721 to $1,373 million while costs were estimated at $1.9 to 8.5 million. The analysis indicates that the corridor states and the motor carriers that operate in them may benefit from the implementation of the ITS/CVO technologies. Key words: Intelligent Transportation Systems, Commercial Vehicle Operations, benefit-cost analysis, corridor analysis.

INTRODUCTION

The existing operational procedures of commercial vehicles, mostly motor carriers, and state enforcement agencies have potentially substantial benefits to gain by integrating Commercial Vehicle Operation functions into Intelligent Transportation Systems deployments (ITS/CVO). In particular, multi-state deployment and coordination of ITS technologies throughout a transportation corridor will lead to greater benefits then single state applications. As a case study, the mid-continent corridor, defined as Interstate Highway 35 (I-35) from Duluth, Minnesota to Laredo, Texas (including the international border crossing at the Laredo), was analyzed to quantify the benefits and costs associated with deployment. The mid-continent corridor was designated a high priority corridor in the National Highway Highway System Designation Act of 1995 and is shown in Figure 1.

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ANALYSIS APPROACH

The research first identified the anticipated benefits from the deployment of ITS/CVO in the mid-continent corridor. Three ITS/CVO functions were analyzed: electronic credentialing, electronic screening, and electronic border crossings. A methodology was developed to quantify the benefits for each of the ITS/CVO functions and the data to support the analysis were collected from various state agencies and the literature. To capture the uncertainty in future benefits streams, three estimates (named conservative, aggressive and expected) were developed for the market penetration

FIGURE 1 Mid-continent transportation corridor.

TABLE 1 Summary of Electronic Credentialing

Benefits (million dollars)	Conservative	Expected	Aggressive
Motor Carriers	21.88	29.71	43.74
Jurisdictions	3.62	4.85	7.06
Total Benefits	25.50	34.56	50.79
Costs (million dollars)	Low		High
Motor Carriers	12.84		26.29
Jurisdictions	6.30		12.00
Total Costs	19.14		38.29
Motor Carriers			
B/C, Low Cost	1.70	2.31	3.41
B/C, High Cost	0.83	1.13	1.66
Jurisdictions			
B/C, Low Cost	0.58	0.77	1.12
B/C, High Cost	0.30	0.40	0.59
Total			
B/C, Low Cost	1.33	1.81	2.65
B/C, High Cost	0.67	0.90	1.33

of ITS/CVO technology, increase in truck volumes, increase in credentialing activities, and border crossing movements. Two cost estimates for each technology were also developed, a low and high costs estimate. A more complete description of data collection, methodology, and analysis can be found research report (*1*).

ELECTRONIC CREDENTIALING

Electronic credentialing functions for commercial vehicles will increase the efficiency of the existing process. Rather than the inefficient paper-based system currently in use, electronic credentialing will allow motor carriers or their representatives to request, pay for, and receive any necessary credentials or permits for all jurisdictions through a single, simplified electronic interface.

Benefits

The benefits of electronic credentialing were calculated as the labor savings made available to motor carriers and jurisdictions from the functions of electronic credentialing. As base data, the number of International Registration Plan (IRP) accounts, International Fuel Tax Agreement (IFTA) accounts, and Oversize/Overweight (OS/OW) permits were collected for all states in the corridor. The benefits were calculated for all motor carriers in the corridor states.

The labor savings for motor carriers were calculated as the difference between the current and future activity times. To account for the different cost structures of carriers, cost savings estimates were made for small, medium and large carriers. Activity times for current and future IFTA and IRP credential applications were derived from a recent Western Highway Institute (WHI) field operational test (*2*). The benefits for electronic requests of OS/OW permits were determined by estimating a cost savings per permit.

The labor savings to jurisdictions for IRP renewals and IFTA renewals were estimated using two sources. The current activity times for processing each type of application was taken from the WHI report. The future activity times were derived by reducing current times with factors estimated in a National Governor's Association (NGA) report on ITS/CVO benefits (*3*). The labor savings for oversize and overweight permits were calculated similarly.

A summary of the calculated benefits over the ten year analysis period is presented in Table 1. Clearly, the majority of benefits accrue to the motor carrier industry. The three estimates were base on the relative market penetration and growth in truck traffic in the corridor taken over a ten year period. The conservative estimate assumes 20 percent of the carriers will initially participate in electronic credentialling and penetration will grow by 3.5 percent per year until 50 percent participate, expected assumes a 5 percent per year growth to 60 percent of the carrier participate, and aggressive assume a seven percent per year growth rate to a maximum participation of 80 percent. The benefits to states were difficult to estimate accurately without a detailed knowledge of the agency structure. As a result, the benefits to state agencies might be underestimated. A more detailed approach would allow more accurate estimation of benefits.

Costs

The costs for deployment of electronic credentialing will be shared by state agencies and motor carriers, however, the majority of costs will be borne by the state agencies. The cost for motor carriers include the cost of personal computer based software system, annual communication cost, and maintenance of the system. The costs were also developed for all carriers in the state. The results of the cost estimates are summarized in Table 1.

The cost to state agencies included the modification costs of existing computer systems, interface systems for communication with carriers, annual communications cost, training of employees, annual maintenance costs, software development costs, and new hardware costs. The development of cost estimates for the implementation of electronic credentialing technologies was a difficult task. Current completed operational tests are limited in scope and the extrapolation of their costs to entire state agencies is difficult. The cost of deployment of electronic credentialing largely depends on the current status of credentialing and permitting in each individual state. The cost to state agencies was estimated very generally using the NGA report (*3*). The estimates are presented in Table 1.

Benefit to Cost Ratio

The analysis indicates that except for the most conservative growth and high cost estimate, benefits exceed costs for motor carriers. These ratios do not represent the B/C ratios that would be available to individual carriers, rather they represent the benefits and costs to motor carriers in the corridor states. A report by the American Trucking Association (*4*) found that B/C ratios for individual carriers from the implementation of electronic credentialing functions would range from 1.0 for small carriers to 19.1 for larger carriers.

TABLE 2 Summary of Electronic Screening

Benefits (million dollars)	Conservative	Expected	Aggressive
Motor Carriers	13.47	16.75	22.77
Jurisdictions	36.33	36.33	36.33
Total Benefits	49.80	53.08	59.10
Costs (million dollars)	Low		High
Motor Carriers	4.98		8.38
Jurisdictions	2.73		14.87
Total Costs	7.72		23.24
Motor Carriers			
B/C, Low Cost	2.70	3.36	4.57
B/C, High Cost	1.61	2.00	2.72
Jurisdictions			
B/C, Low Cost	13.29	13.29	13.29
B/C, High Cost	2.44	2.44	2.44
Total			
B/C, Low Cost	9.99	10.65	11.86
B/C, High Cost	5.95	6.34	7.06

For this research, benefits only exceeded costs for the most aggressive growth scenario and the lowest cost estimate. The NGA report (*3*), however, found B/C ratios ranging from 2.0 to 7.0 for state agencies. Benefits were also not included for such agency savings as paper costs, mailing savings, and reduced auditing costs. These benefits, however, are not likely to be significant enough to influence the B/C ratios.

ELECTRONIC SCREENING

Electronic screening will take place at static weigh/inspection facilities and possibly portable inspection sites. Vehicles known to be in compliance or having a low risk are allowed to bypass the weigh station, thereby providing travel time savings to the motor carrier and freeing agency personnel for other activities. Increased enforcement initiated by electronic screening will likely force a reduction in overweight trucks.

Benefits

As base data, each agency that operates the 19 static scales along I-35 was contacted to obtain the hours of operation, the number of trucks weighed, and the number of safety inspections performed (Level I, II, III). Except for Kansas, the corridor states have low levels of enforcement, weighing between 2 and 10 percent traffic (Minnesota does not presently operate fixed scales on I-35). In order to estimate reduced pavement damage and safety benefits, data were collected in Iowa to be used as an estimation for the rest the corridor. A sample of overweight citations not issued at fixed facilities was collected from the three Iowa counties with scales on I-35. In addition, accident data were collected within one mile of the weigh station location in the same counties. This data revealed no significant accidents related to weigh station activities.

Benefits to motor carriers included both time and fuel savings for bypassing the weigh station. Time savings was the difference between bypass time and through time, defined as the deceleration time off the mainline, time spent in queue, time at the scale, and time spent in acceleration onto the mainline. In addition, the time savings from bypassing safety inspections was also included. Fuel savings, 1/3 gallon per bypass per truck, were estimated using the results from a fuel consumption test from the Advantage I-75 project (*5*).

Benefits to jurisdictions included two parts: labor savings and reduced pavement damage caused by overweight trucks. The salary savings were calculated using existing operations as the base. It was assumed that ITS/CVO technologies would reduce the required personnel at each operating station. These benefits were minor. An estimate was made of the reduced amount of pavement damage for the corridor. To develop the estimate, overweight citation data were analyzed to determine a distribution of weight and axle configuration for overweight trucks. These base data were applied to the assumed amount of overweight trucks, estimated to be 5 percent of the total truck traffic, when the fixed scales were not operating. Because of the increased enforcement allowed through ITS deployment, the number of overweight trucks operating was expected to decline over the ten year period. The miles of overweight equivalent single axle loads (ESAL) saved were converted to a dollar amount.

A summary of the benefits of electronic screening is shown in Table 2. The benefits, as well as costs, are estimated over a ten year period. It is assume that the level of enforcement will grow at the same rate as truck traffic growth and participating carriers will bypass weigh stations. The conservative estimate is based on 1.5 percent per year growth in truck traffic, expected is based on 2.5 percent per year growth in truck traffic, and aggressive is based on a 3.5 percent per year growth in traffic. Under the conservative estimate, it is assumed that initially 20 percent of trucks will participate in the electronic screening program and participation will grow by 3.5 percent per year to a maximum of 50 percent, the expected estimate assume that participation will grow by 5 percent per year till 60 percent participation, and aggressive assumes a 7 percent per year growth in participation until 80 percent penetration is reached.

Compared to other analyses of electronic screening to motor carriers, these benefits are relatively low. This is related to the relatively low level of enforcement in the corridor states. Also the pavement damage savings were calculated using the overweight citation data from Iowa as representative of the entire corridor which may not accurately represent the entire corridor. Further, pavement and truck configurations could vary significantly along the corridor, which was not accounted for in this analysis. However, this simple analysis reveals that pavement damage savings could be very significant.

Costs

The costs for electronic screening will be shared by motor carriers and state agencies, but again the majority of costs will be borne by state agencies. The final estimates are shown in Table 2. The costs to motor carriers include the cost of transponders.

TABLE 3 Summary of Electronic Border Crossing

Benefits (million dollars)	Conservative	Expected	Aggressive
Motor Carriers	721.92	947.53	1373.05
Jurisdictions	na	na	na
Total Benefits	721.92	947.53	1373.05
Costs (million dollars)	Low		High
Motor Carriers	0.47		5.46
Jurisdictions	1.44		3.05
Total Costs	1.91		8.51
Motor Carriers			
B/C, Low Cost	1,532.39	2,011.28	2,914.49
B/C, High Cost	132.21	173.53	251.46
Total			
B/C, Low Cost	377.69	495.72	718.34
B/C, High Cost	84.80	111.31	161.29

The costs to state agencies include automatic vehicle identification (AVI) readers, weigh-in-motion (WIM) scales and equipment, computer workstations, communication costs, annual operation and maintenance costs. The costs for the necessary infrastructure were developed using estimates in the NGA report (*3*). Two cost scenarios were developed for deploying ITS at weigh stations.

Benefit to Cost Ratio

The ratios in Table 2 reflect that for electronic screening, analyzed for the entire corridor, has benefits that exceed cost for all scenarios. Unexpectedly, even with the low enforcement levels and truck volumes, electronic screening proves cost effective. The relatively low cost of implementation and the significant benefits yield these results.

ELECTRONIC BORDER SCREENING

Electronic processes at the international border will use some of the same functions as domestic electronic credentialing and screening, but will be more complex. All of the necessary paperwork and information to expedite transborder shipments will be transmitted in advance to the customs facility. When motor carriers arrive at the border, they will be screened by border crossing officials with minimal delay.

Benefits

Base data were collected from a variety of sources. The most recent data were available from the United States Customs Service. The Border Trade Institute at Texas A&M International University supplied northbound and southbound traffic from 1990-1995 for different bridge locations in Laredo.

Benefits to motor carriers were calculated as a function of time savings at the two border bridges accepting commercial vehicles, Laredo and Columbia, for northbound and southbound traffic. Benefits to shippers were calculated as reduction of in-transit inventory costs due to reduced transborder shipping time. Total benefits for the analysis period are presented in Table 3 (Shipper benefits included in motor carrier benefits). The conservative estimates assume 8 percent per year grow in transborder traffic, the expected estimate assumes a 10 percent per year growth rate, and the aggressive assumes a 12 percent growth rate. All three growth assumptions are based on market penetration rates for international electronic screen equivalent to the growth rates for domestic electronic screening.

The method used to calculate reduced inventory costs was extremely conservative. The addition of the inventory savings in this methodology is insignificant compared to the time savings to motor carriers. In practice, however, inventory savings could be very relevant. If total transborder time was reduced by one day, benefits could be a minimum of 1 million dollars annually for northbound traffic at the Laredo bridge alone. This methodology also did not attempt to analyze the reduction in warehouse inventory which would be possible with more reliable transborder shipping time. This benefit is likely to be very significant.

Because the traffic situation at the border is dynamic, the estimation of these benefits should be viewed carefully. The methodology assumes that the delays at bridges will remain constant over the analysis period. If no improvements were made, infrastructure or otherwise, delay at the bridges would most likely increase. While truck traffic is increased annually, the method does not account for the shifting of traffic from one bridge to another, rather, it assumes a constant growth. However, even the current situation, with no growth, produces annual benefits of 26 million dollars. Also, this analysis does not address the future plans for the fourth international bridge planned at Laredo.

Benefits to jurisdictions are likely to result from an increased efficiency in customs enforcement. Staff reductions at the border are unlikely, but the reduced delays will allow customs to better direct their efforts towards drug enforcement and other activities. In addition, Texas will be able to perform more effective weight and safety inspections, thereby decreasing the number of overweight and unsafe vehicles on Texas roadways. These benefits, however, were not calculated because of data constraints.

Costs

The costs to motor carriers for the installation of electronic border crossing include the cost of transponder and annual communication costs. Cost for motor carrier participation in electronic border crossing technology was much the same as the cost to participate in electronic screening. A summary of costs are shown in Table 3.

The costs to jurisdictions will include AVI readers, WIM scales and equipment, computer workstations, communication costs, and annual operation and maintenance costs. Total present worth of the implementation of the system is presented in Table 3. The costs for electronic border crossing have not included the development cost of the software that allows shippers, carriers and drivers to file the necessary documents to participate in the technology.

Benefit to Cost Ratios

A summary of benefit to cost ratios is shown in Table 3. Because delays at the border are so significant and the cost to motor carriers of participating in the international electronic screening program is so small, large benefit to costs ratios result. Since jurisdictional benefits were not quantified, and although the jurisdictional benefits are significant, they were not included in the analysis. This simple analysis of the border shows that implementation of ITS/CVO functions at the border would have significant benefits. In fact the benefits are so significant that it would be improper not to implement these functions at the border. (Author's note: This technology has since been deployed at the Laredo border).

CONCLUSIONS

The analysis of electronic credentialing functions for the entire corridor was a complicated task. Many assumptions were made in performing the analysis. For example, the methodology assumes that time savings for electronic credentialing functions are identical in each state, which is likely not the case. A majority of the time savings approximations were based on WHI's operational test of electronic credentialing. This test had a limited sample size and the savings identified in the report may not necessarily apply to the motor carrier industry and state agencies in the mid-continent corridor. Perhaps the part of the methodology requiring the most refinement is the analysis of OS/OW permits. Because of data limitations, average values of time and dollar savings had to be used to estimate the benefits of electronic credentialing functions for OS/OW permit applications. The structure of motor carriers who apply for permits may make an average value of the savings inappropriate. Final conclusions about electronic credentialing functions can be drawn, however, that the implementation of electronic credentialing will be beneficial to both motor carriers and jurisdictions.

The analysis of electronic screening showed that the application of these functions had benefits greater than costs. The methodology used to calculate benefits for motor carriers could be improved by collecting more than one year of weigh station data. This would eliminate any concerns about the sample not being representative. The cost analysis for jurisdictions could be improved by collecting more specific information on each individual weigh stations and determining the exact nature of improvements necessary to allow electronic screening. A more detailed analysis of overweight data and pavement structures in other states would improve the accuracy of the pavement damage savings. Final conclusions about electronic screening functions can be drawn, however, that the implementation of electronic screening will be beneficial to both motor carriers and jurisdictions.

The analysis of electronic border crossing found that benefits to motor carriers far exceeded any costs. Analysis of the Laredo border was dominated by the existing delays for northbound traffic at the downtown bridges.

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ITS Opportunities in Port Operations

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Intelligent Transportation Systems (ITS) reduce congestion and increase safety and efficiency on our streets, highways, railroads, and airways, in an attempt to create an intermodal system which facilitates passenger and cargo transfer. Considerable research has been and will be conducted in these areas. However, ITS applications concerning intermodal freight operations have not been explored as rigorously. When speaking of freight operations, intermodalism refers to the complex interactions of surface transportation and water and rail modes to move goods, both domestically and internationally. The efficient movement of freight is essential to the economy. The economic value of moving goods via water transportation has been demonstrated and proven in many industries. ITS technologies can increase this economic value by improving the productivity and safety of intermodal freight operations. The global trade demands of the intermodal freight transport industry are growing at unprecedented rates and the available land on which infrastructure improvements can be built is dwindling. Operations must become more efficient if these increased demands are to be met. ITS-related technologies address operational efficiency problems. Cost savings and competitive advantages must be realized quickly if these often-expensive upgrades are to be implemented. Therefore, research must be conducted to evaluate these individual systems and their compatibility and coordination with each other and existing management systems, before large implementation investments are made. This paper attempts to evaluate these ITS-related systems and their applicability to intermodal freight operations by way of an in-depth examination of their effects on intermodal port operations around the world. Key words: ITS, port operations, intermodal, freight.

INTRODUCTION

Port operations and planning must take into account the current increasing demand as well as future demands expected to be handled by the system. Ports must provide efficient links between water and surface transportation and concentrate on information flows as well as the movement of the cargo itself. Advanced technologies must constantly be examined and implemented to enhance the overall efficiency of the port. The global trade demands of the intermodal freight transport industry are growing at unprecedented rates while the available land on which infrastructure improvements can be built is dwindling. This leads to operation under severe constraints of land in addition to human resource controls. New infrastructure cannot always be constructed, thus existing infrastructure must be managed more efficiently to create additional capacity. Advanced applications are required to maximize the efficiency of ports and to provide for this increase in capacity. Intelligent Transportation Systems (ITS) can offer opportunities to optimize the use of the existing transportation system to generate additional capacity using the existing infrastructure. The application of advanced and state-of-the-art technologies have, and will continue to, transform the freight transportation industry.

Transportation systems provide mobility for our nation's commerce. The economic value of moving goods via water transportation has been demonstrated and proven in numerous circumstances. ITS technologies can add to this economic value by improving the productivity and safety of intermodal freight operations. This freight movement encompasses multiple modes. Intermodal terminals are a critical transfer point for moving cargo through the transportation and distribution process, but the transfer between vessels and inland transportation is one of the weakest, least efficient, and most costly links in the intermodal chain (*1*). Pressure on intermodal terminal operators to cut costs, diminish congestion, reduce damage claims, and attain higher overall levels of yard efficiency are forcing advanced technology, particularly information technology, to revolutionize impacts on the physical distribution of freight. Effective intermodal shipment of freight requires not only the transfer of the cargo itself, but also the transfer of information between transportation modes. Advanced technologies are likely to improve transportation productivity with new equipment and vehicle systems to increase capacity, new state-of-the-art terminals using advanced technology for cargo interchange and handling to reduce transfer times and costs, and new information and communication technologies which offer opportunities to generate additional system capacity through sophisticated management of the existing transportation infrastructure.

CARGO AND EQUIPMENT TRACKING

The use of state-of-the-art material and cargo handling technologies including tagging, tracking, and information management systems can offer the ability to expand the capacity of commercial terminals. These systems offer the ability to track, identify, and monitor cargo and equipment in real time. By electronically monitoring the movements and locations of equipment, operators can track the movement and location of all containers entering, exiting, or remaining in the terminal. Equipment and cargo visibility allows for a higher degree of inventory control, which leads to faster, more efficient, and reliable operations.

Currently used automated equipment identification (AEI) technologies include radio frequency (RF), bar coding, smart cards, and satellite-based operations. RF tags use radio signals to communicate real time information including location coordinates, weight, size, and identification numbers to the centralized control/management system. Warehousing and manufacturing applications

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have used RF technology for years to manage traffic flows through gates and to track yard equipment.

Utilizing bar codes on containers dramatically reduces the tedious and error-prone tasks of reading and recording cargo information. Another technology for reading and recording identification letters and numbers on containers and trailers is Optical Character Recognition (OCR). OCR systems use high-speed line-scan video technologies to electronically read identification numbers stenciled on containers. Systems such as these can provide accurate asset tracking and can create load lists automatically, while significantly improving data accuracy and location information.

Smart cards are being implemented for managing the entry and exit of vehicles within the terminal yard as well as for payments of tolls and gasoline. These credit card sized integrated circuit cards have the ability to store and process information. Satellite-based location determination technologies provide unparalleled accuracy to within meters. These satellites use either the Global Positioning System (GPS), geosynchronous orbital satellites, or Low Earth Orbit satellites (LEO) to track vehicles, seacraft, aircraft, etc. The major advantage of a GPS is its ability to operate with minimal additional infrastructure (*2*). GPS satellites have been functioning for years, and tracking vehicles or cargo only requires a GPS receiver to pinpoint its exact location. Other great advantages of LEO are that they are relatively inexpensive to use, they are small, typically 45 kg, and can be launched from a single small rocket fired from an aircraft at an altitude of only 12.2 km. LEO orbit at approximately 800-1000 km, and are close enough to receive signals from hand-held devices (*3*).

The main objective of AEI is to track containers the entire time they are in the terminal and feed back location and status information to the central information systems in real time, where the data can be compared with the selected location (*4*). Dramatic improvements in container throughput, accuracy levels, and overall container management can be expected from these technologies. Other benefits include reduced container stowage times, reduced labor costs associated with identifying, locating, and moving containers within the yard, and simplified inventory checks.

AUTOMATED GUIDED VEHICLES

Automated Guided Vehicles (AGV) utilize unmanned vehicles which are self-propelled by using automated controls. An AGV system includes the vehicle, navigation system and guidepath, automated controls (including the traffic management system to monitor moves, inventory, and vehicle status), obstacle detection system, and appropriate interfaces with other computers and terminal operation systems. AGV systems are used in terminal operations for the retrieval and storage of containers. Onboard computers on each AGV communicate using wireless transmissions with the control center to allow the vehicle to navigate to any point within the terminal. AGV provide efficient and flexible maneuvering with minimum manpower, high container throughput at reduced costs, continuous operation possibilities (24 hours a day, 7 days a week), and consistent container handling operations. AGV systems are generally suited to repetitive actions and can provide benefits to both the port and its customers. The lack of use of AGV technology in the US is a result of labor agreements between port/terminal operators and labor unions (*5*).

AGV systems can be designed to interface with other automated systems such as automated storage and retrieval systems that can provide even greater levels of flexibility. Current AGV navigation systems include vehicles that follow a path of buried wires below the surface, photofluorescent and reflective material applied to the path in stripes, laser based systems which utilize triangulation to determine location, onboard gyroscopes which track the precise heading of the vehicle, and other systems which utilize a grid marked on the floor. AGV are highly maneuverable and can be equipped with GPS or similar AEI technologies to track their movement and location throughout the yard.

The major problem with conventional AGV systems appears to be the interface between the crane and the surface transports, as they must operate in conjunction, invariably leading to one waiting on the other (*6*). If AGV are queuing under a crane, the problem is not so important, but when a crane has to wait for an AGV, then the performance of overall cargo handling can be seriously disrupted. Overall, AGV can still offer a safe and reliable alternative to human operators in environments where material transport requirements are well defined and reasonably static. The overwhelming benefits of AGV include increased efficiency, reduced labor costs, improved safety, and improved inventory tracking.

COMMUNICATION AND DATA EXCHANGE TECHNOLOGIES

The exchange of information is as important to freight movement as the movement of the cargo itself or the equipment that is moving it. In freight transportation, if information doesn’t move, cargo doesn’t move. The more seamless the information flow is, the quicker cargo can get from its origin to its destination. With these points in mind, it is appropriate to realize that considerable savings in time, safety, and efficiency could be realized by the adoption of electronic forms of data exchange.

Electronic Data Interchange (EDI) communications facilitate the smooth hand-off of cargo from mode to mode, as well as automating billing, data entry, tracking functions, and other information exchanges such as cargo manifests, bills of lading, vessel arrival times, in-bond movements, and status notifications. These and other newly developed Information Technologies (IT) are becoming common practice and can provide reduced in-terminal processing and inspection times, increased vehicle throughput, improved data accuracy, enhanced yard efficiency, and eliminated gate paperwork. IT can reduce cycle times, forward documents, manage inventory, plan schedules, and make purchases, all electronically and automatically.

Communications, information, and integrated data systems are the fundamental sectors of IT. IT has already had a revolutionary impact on freight distribution, and as many ports are somewhat constrained in expansion plans to acquire more land they must look to efficiency advancements to increase cargo productivity and expand their EDI interchange (*5*). EDI can allow a user to select any container and gain instant data on the container’s location, weight, and identification number, all through electronic communications. EDI systems can link Customs, shiplines, inland carriers, importers, and exporters. This helps shipping lines transact business conveniently and expeditiously within the port and to transship their containers in the fastest way possible, since automation results in quicker release times.

The automation of information management through EDI allows actions to commence within hours instead of days or weeks. These and other advanced technologies have dramatically changed the freight industry, and will continue to do so only if standardized communication occurs across all modes. The transfer of data between business parties must use very specific industry standards, data sets, and protocols. As shippers, ports, freight forwarders, and transportation companies have computerized their record systems, the only major impediment to transmitting more paperwork electronically has been the incompatibility of the many data systems (*7*).

ITS programs offer opportunities to apply concepts such as dynamic flow control, as developed in the aviation and rail transportation systems, to enhance the mobility of urban freight (*8*). Dynamic flow control is defined as the active, intelligent balancing of transportation and logistics demand and supply to minimize congestion and maximize capacity and flexibility. It requires real time, or near real time data on vehicle locations and network conditions, powerful analytic capabilities, and management's ability to affect and control operations. Efficient and centralized control systems are essential to ensure efficient and highly controllable operations of ports and terminals. The various data collected at the port and terminal must be integrated into a computerized terminal operating system that can maximize gate throughput and reduce costs.

TERMINALS

Techniques can be applied within the terminals themselves that greatly improve container-handling and overall operational efficiencies. One change that requires no new technology is operating 24 hours a day, 7 days a week (*9*). For this innovation to be cost effective, changes in long-standing overtime pay and labor hiring practices must be implemented. Cargo can be delayed in terminals for lack of clearance or problems in communication between parties. Advanced technologies can help to alleviate these inefficiencies. Increased terminal efficiencies can be realized through applying advanced information systems such as smart cards and RF readers and tags at entry/exit gates. The volume of truck traffic at marine intermodal terminals is large, and each truck arriving at the terminal must stop at the entry gate for processing, which includes matching truck and container numbers to shipping orders, identifying the driver for security, and assigning a pick-up or delivery location for the container or chassis. The use of variable message signs for vehicle guidance has been identified as an appropriate approach for certain port terminals to alleviate truck congestion.

AEI technologies will speed up entrance procedures, reduce queues and bottlenecks in the terminals, and enable a quicker throughput. Ports should also improve access to their infrastructure: on the waterside, channels should be kept dredged to depths necessary to handle the larger next generation vessels; and on the landside, adequate highway connections should be built with railroad clearances and yards adjusted to handle doublestack railcars. By using advanced cranes with improved capabilities, new port facilities may be able to accommodate loading and unloading from both sides of a container ship at the same time. These advanced cranes may also be capable of lifting more than one container at a time and could provide semi-automated systems in which learned paths might be developed to ultimately rely less on human operators.

With greater accuracy in container location and identification and computerized information bases, terminal managers can develop and apply management information systems to improve the flow of containers through the terminal with reduced handling (*10*). Intelligent terminal management software, including expert systems and automated planning tools, can control tracking, parking, staging, stacking, loading, stowing, and planning, as well as provide real time inventory and traffic management functions.

NEEDS FOR FURTHER RESEARCH

It is obvious that there are many benefits to applying ITS and related advanced technologies to port and terminal operations to improve efficiency, safety, and productivity through improved technology and information sharing. The difficulty lies in finding ways to promote these applications as major improvements and thereby attracting the public investment needed for implementation. The US Department of Transportation must provide leadership for the private sector and state and local governments, while closely working with the US Department of Defense (DOD) (the largest user of the transportation system) to develop a shared vision of benefits, promote the adoption of industry-wide standards, and to encourage research on, and the dissemination of, industry innovations in freight transportation. Research should be conducted on topics such as freight movement through terminals, improving information exchange, as well as examining some newer technologies being introduced by DOD for military operations that would operate simultaneously with commercial operations.

CONCLUSIONS

Remarkable productivity gains in intermodal transportation over the past 30 years have been achieved due to hard side technology improvements (containerization, larger ships, improved cranes, and doublestack trains). Further productivity gains most likely will come from soft side innovations relying mainly on information technologies. Engineers and planners must apply advanced cargo management and control systems, both shipboard and ashore, to provide the mechanisms for timely and dependable deployment of cargo from origin to destination on a sustained basis. These technologies must be applied to existing problems and integrated with current operational and informational structures and systems at port terminals.

Terminals must be continually upgraded if they are to efficiently handle the increasing volume demands placed on them, a challenge made even more difficult by the fact that many ports will need to handle this increased volume without increasing available physical space. Therefore, advanced information and communication technologies applied consistently across the entire intermodal system can offer important opportunities to increase existing system capacity. Data standards and communication protocols used in control and tracking systems must be coordinated to facilitate data exchange between parties, reduce dwell time of cargo, and increase terminal throughput. Improved terminal designs, advanced computer modeling and simulation systems, and advanced technologies for moving cargo and information, operating in concert with focused logistics and advanced information management systems can be applied to accelerate the movement of freight through intermodal port terminals. State-of-the-art ports such as Charleston, Tacoma, Los Angeles, Rotterdam, Singapore, Hong Kong, and Saudi Arabia have effectively implemented many of the technologies overviewed in this paper. These ports are therefore excellent study sites in efficient port operations and technological implementation. Appropriate research and policy efforts directed at the above technology based opportunities will yield significant gains for the US and global economies and improved cost-effectiveness in the US and world transportation systems.

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Use of Highway Network Level Data for a Project Level Life Cycle Analysis

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The subject of this paper covers a project that was successfully commenced in 1997 and completed in 1998. This paper discusses the development of a method to use network level PMS data for a project level life cycle costing analysis. The method was successfully applied to a variety of road conditions and structures that make up the primary highway network in Saskatchewan, Canada. This project followed on from a project that implemented probabilistic and deterministic network level PMS within Saskatchewan Highways and Transportation. The project that is the subject of this paper was to determine the whole of life implications on level of service for different funding scenarios on different types of road structures. The paper discusses the details of the method identified and the network level data that was use. The paper specifically focuses on description of the Network Level Probabilistic cost/deterioration models; description of Network Level Deterministic deterioration models; how the models were combined to develop a Deterministic Project Level deterioration versus maintenance cost model; application of the project level models in Life Cycle cause and effect models; the method used to analyze the above to develop Net Present Worth and Equivalent Annualized Cash Flow for different level of service starting case scenario.

BACKGROUND

Statement Of Problem

Saskatchewan Highways and Transportation (SHT) is a major provincial highway agency in Canada. SHT performs a benefit/cost analysis for capital construction projects. One of the inputs in the analysis is the cost of maintenance over the duration of the analysis period. It is recognized the cost of maintaining a roadway increases as its condition deteriorates. SHT wants to take this factor into consideration during their cost benefit calculation.

Issues

To accommodate the above statement of the problem facing SHT, a project was commenced in July 1997 to identify the cost of maintenance by using a life cycle costing analysis, taking into account the following:

TABLE 1 Probablistic Condition States

CONDITION STATE	IRI	SUB. FAIL	FAT. BLOCK	PERF. INDEX
1	1	1	1	1
2	1	1	1	2
3	1	1	2	1
4	1	1	2	2
5	1	2	1	1
6	1	2	1	2
7	1	2	2	1
8	1	2	2	2
9	2	1	1	1
10	2	1	1	2
11	2	1	2	1
12	2	1	2	2
13	2	2	1	1
14	2	2	1	2
15	2	2	2	1
16	2	2	2	2
17	3	1	1	1
18	3	1	1	2
19	3	1	2	1
20	3	1	2	2
21	3	2	1	1
22	3	2	1	2
23	3	2	2	1
24	3	2	2	2

- A 30 year analysis period;
- Cost reported in per square meter of roadway surface;
- Expressed as a per year cost;
- The analysis must be specific to Region rather than apply to the whole Province; and
- The results must relate to current Network Level PMS condition states (Table 1). Note : 1=good condition and 3=poor.

ANALYSIS METHODOLOGY

Considerations

It was recognized that SHT had a variety of sources of high quality information relevant to the problem. This included:
- Probabilistic prediction models
- Cost vs. condition models
- Deterministic project level prediction models

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- Detailed activity cost data
- Time series “actual” condition data

The problem was one of determining the best information to deterministically predict condition and cost using all of the sources available.

Discussion of the Method

The problem outlined was essentially a Life Cycle Cost problem. This was because the condition and the cost of maintaining a roadway fluctuated over time. Predicting deterioration of a roadway is complex because the actual deterioration path is dependent upon which treatments are applied, when they are applied (what is the condition of the roadway at that time) and what is the effect on the roadway performance (how does the condition change). In other words, the deterioration path is related to the preservation strategy applied to the network. A strategy is simply a proposed set of treatments for the expected life of the asset. Life Cycle Costing is applied to the strategy to determine the Net Present Value (NPV) and Equivalent Annual Cash Flow (EACF). The EACF is an equivalent annual cost of a strategy brought to current value.

The option expected to give the “best” result (most representative and repeatable) was to:

- Derive a deterioration path from deterministic performance survival models and relate the deterioration to the probabilistic condition states of the network.
- Apply the condition state costs to a specific strategy. The cost models are also deterministic, specific to a pre-defined set of conditions and have been proven through actual use. (The link to the costs is via the defined condition states in the probabilistic models).

The method of analysis consisted of:

1. Determining the deterioration path for a road pavement class for a geographical area by analyzing the deterministic performance model.
2. Determining the Life Cycle Cost of the normal preservation strategy.
3. Determining Life Cycle Costs based on differing start condition states. The longer the analysis period was, the less sensitive the Life Cycle Cost would be to the starting condition of the roadway.
4. Comparing the normal preservation strategy to a variety of other preservation strategies to determine sensitivity of the solution. A preservation strategy involves application of specific treatments at particular distress levels. If the NPV is approximately the same, the model is not particularly sensitive. If there is a strategy with a significantly lower NPV or EACF, then it is clearly the best strategy.
5. Once the “best strategy” had been derived, we now had a working methodology for a solution. A “reality check” was to review the solution with SHT highway practitioners looking at the sensitivity of strategies and the start condition state. Based on the review, the extent of the analysis for other surfaces was then assessed.

Condition Path

Figure 1 illustrates a condition path for a single distress over time. As the roadway ages, the severity of the distress increases. Periodically, a treatment is applied which results in an improvement in condition or a “Gain in Score” (GIS).

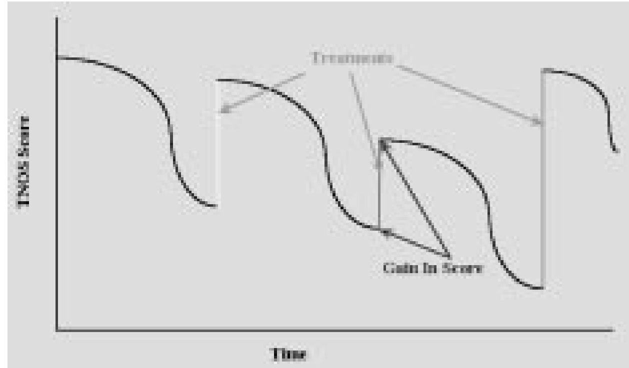

FIGURE 1 Condition path for a single distress.

TABLE 2 Gain In Score and Maximum Gain In Score

Treatments	Gain In Score For Distresses IRI	Sub-Grade Failure	Fatigue Blocking	Rutting
Sandvik Blading	15	10	15	15
Machine Patching	10	10	10	10
Heavy Patching	10	20	15	15
Full Seals	5	8	35	5
Spot Seals	0	5	25	2
Maximum Score	20	30	40	20

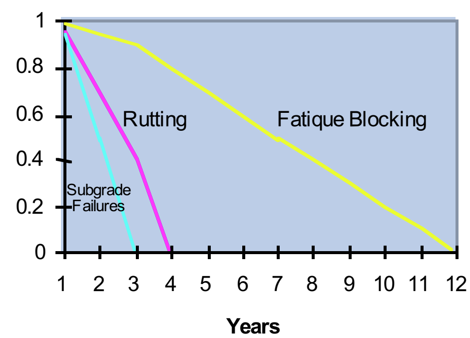

FIGURE 2 Untreated survival curves.

A typical roadway has many distresses that deteriorate at different rates. As illustrated in the diagram, specific treatments improve some distresses more than others. This means that the combination of distresses will determine the specific treatment selected. That treatment may result in improvement to several distresses.

The deterministic model handles different distresses by converting their condition rating to a “score.” Relative importance of treat-

TABLE 3 Deterioration Path for the Analysis

Start State	Year 0	1	2	3	4	5	6	7	8
1	1	1	1	21	21	21	21	21	23
3	3	3	3	23					
5	5	5	5	21	21	21	21	21	23
7	7	7	7	23					
9	9	17	17	21	21	21	21	21	23
11	11	19	19	23					
13	13	21	21	21	21	21	21	21	23
15	15	23	23	23					
17	17	17	21	21	21	21	21	23	
19	19	19	23						
21	21	21	21	21	21	23			

TABLE 4 Life Cycle Costs for Condition State 1

Strategy	0%	2%	4%	6%	8%	10%
Strategy 1						
NPW	18.81	13.75	10.38	8.05	6.41	5.22
EACF	0.63	0.61	0.60	0.59	0.57	0.55
Strategy 2						
NPW	18.25	19.39	10.06	7.83	6.25	5.12
EACF	0.61	0.60	0.58	0.57	0.56	0.54
Strategy 3						
NPW	17.23	12.61	9.59	7.43	5.95	4.88
EACF	0.57	0.56	0.55	0.54	0.53	0.52
Strategy 4						
NPW	16.56	12.17	9.24	7.24	5.82	4.79
EACF	0.55	0.54	0.53	0.53	0.52	0.51
Strategy 5						
NPW	19.30	14.86	10.58	8.20	6.53	5.32
EACF	0.64	0.63	0.61	0.60	0.58	0.56

ing distresses is achieved by assigning the "maximum" score to each distress. The effect of each treatment on each distress is specified through the "Gain In Score" assigned. to it for each distress. These are illustrated in Table 2.

The method that was developed to be a reasonably realistic modeling of the whole of life decision-making was:

- If the treatment had a GIS for a distress, then that distress took on the new treatment's curve.
- If GIS = 0 then it continued down the current curve.

Deterioration Path

The deterioration path is derived from the survival curves in the deterministic model. The curves are shown in Figure 2. Note that the roads that were the subject of this analysis are low traffic volume, low cost roads and the curves reflect that situation. The method was later successfully applied to structural pavements.

The survival curves are used to determine a score for each distress as the road ages. These scores were mapped against the distress bins shown in Table 5. The combination of distress rating gives the probabilistic model condition state which are shown Table 1.

Analysis began with a road in "like new" condition, that is, all distresses "good." The road was deteriorated using the survival curves and the condition states were derived. For other "start states," the distresses were mapped against the deterministic model score for each start condition and the road deteriorated from there. The resulting deterioration paths for analysis are shown in Table 3.

Maintenance Costs

Maintenance costs were determined through a 30-year Life Cycle Cost Analysis. The analysis was based on the following assumptions:

- Using the deterministic model treatment performance and treatment costs;
- For maintenance costs for each condition state, use the probabilistic routine maintenance cost (including overhead);
- Deterioration was based on the untreazted survival curves;
- Apply treatments the year after the condition that triggers the treatment is reached. This recognizes rating is done in the fall for work the following year;
- Apply Gain In Score (GIS) to the previous year's score. The rationale is the road should improve with treatment and that the road will still be in better shape when the rating is done in the fall;
- Apply maintenance costs to the condition state in the year the rating is done. This is based on the fact the road is in that condition state and that is the state crews will respond to.

TABLE 5 Distress Condition Bins

AC Surface	Condition Bin	Rating
Rutting	1, 0, -1, S1, S2, M1, X1	Good
	S3, M2, M3, X2, X3	Poor
IRI	<2.5	Good
	2.5 to 3.2	Fair
	>3.2	Poor
Cracking	0 , -1, S1	Good
	S2, M1, M2, X1	Fair
	S3, M3, X2, X3	Poor
Shear	1, 2	Good
	3, 4, 5	Poor

Maintenance Strategies

Life Cycle Maintenance strategies were derived from the Agency documented practice. This consisted of typical treatments applied to each distress by condition bin for each distress type. The combination and severity of distresses determine the treatment selected for each maintenance strategy.

For the analysis, the Agency practice was used as the basis for treatment selection. The approach was that as deterioration occurred, treatments were selected with a gain in score that would improve the condition of the distress combination. Treatments were selected from those normally applied to the distress. Since multiple treat-

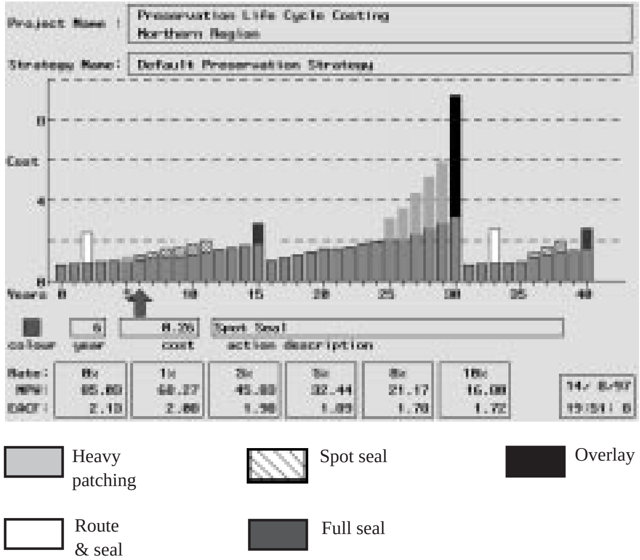

FIGURE 3 Life cycle cost example.

ments can be applied for each distress, various combinations were analyzed to determine the effect on Life Cycle Cost.

Life Cycle Cost Tables

Table 4 shows Net Present Worth (NPW) and Equivalent Annual Cash Flow (EACF) for the five strategies analyzed for Condition State 1 (a new road). The columns represent the result for different discount rates. The table illustrates that the “best” cost strategy is strategy 4.

Strategies 1, 4 and 5 were analyzed for other Start Condition States. Strategy 4 resulted in the “best” cost.

LIFE CYCLE COSTS

Figure 3 illustrates an example of a Life Cycle Cost for one preservation strategy. The example starts with a brand new road and has a complex (real life) treatment program over a 40-year period. The condition related routine maintenance costs fluctuate over time as the overall condition of the roadway changes. The diagram illustrates that condition related costs gradually increase as condition deteriorates and the costs reduce after the application of specific treatments that improve condition. Specific treatments diagrammed include rout and seal, spot seal, full seal, heavy patching and overlay all of which are applied at discreet intervals. Some treatments are shown recurring periodically while some occur several years in succession.

CONCLUSIONS

The overall analysis yielded good results over a variety of pavement types and conditions. When the EACF was compared with the budget predicted to maintain current condition over a long time period the costs per square meter were within 10% of each other. When one considers that the PMS predictions were based on fully probabilistic models and the EACF was not based directly on the probabilistic prediction models this correlation was pleasing and a good reality check on the validity of the analysis.

Managing the Preservation of State Highway Systems

John R. Selmer

With the current national focus on meeting customer expectations, accommodating change, and providing services at the best possible cost, many state departments of transportation are reorganizing or re-engineering themselves. The Iowa Department of Transportation is no different and has accomplished a major reorganizational effort in 1994. Part of this reorganization resulted in the creation of a Maintenance Division as one of the Department's seven major divisions. Accompanying this new Division were several new responsibilities, one of which was the preservation of the state's primary highway system. In the previous structure maintenance personnel were involved in highway preservation; however, the scope has significantly increased in the new structure. Some of the new responsibilities given the Division were not perceived by many as a traditional maintenance role. Our new view is that the Maintenance Division is the "owner/operator" of the system and is in the best position to determine the needs and to prioritize those needs. Based on this reasoning, the following responsibilities have been assigned: 1) Develop fund distribution methodology for the Department's 3R program (Major Rehabilitation) and MP program (Contract Maintenance); approximately $65 million is distributed annually to the Department's six transportation regions; 2) Develop project selection criteria for the Department's 3R and MP programs; this also includes the determination of project letting schedules; 3) Review and refine the preservation process; this involves the integration of the following areas: 4R program (interstate), 3R program, MP program, specialize state maintenance crew, and local area maintenance crews such that their individual efforts are optimized to derive maximum benefit to the system. In order to meet these responsibilities along with others, the Maintenance Division also recently completed a reorganizational effort with its field forces in order to bring greater focus in two areas, operational management and engineering.

INTRODUCTION

Preservation of state highway systems is quickly gaining the attention of top government administrators and the public as the days of highway expansion are beginning to wane. The past pressure to provide new pavements with their associated construction costs in many cases overshadowed future preservation needs, and the consequences are now becoming evident. Many states are finding that current organizational structures and methods along with funding are not sufficient in meeting highway preservation needs. Additionally, those within the organization responsible for funding and programming of future projects may not have the necessary experience and knowledge. In some cases, processes used in an agency lead to competing interests that can undermine the preservation effectiveness that one is trying to achieve.

The Iowa Department of Transportation had, and still has in some cases, many of these challenges. We undertook a major reorganization effort along with a divisional reorganization in which significant changes were made with the hope to better align us with future demands. This paper will discuss the effects of these changes on our highway preservation process and what our vision is for the future.

DEPARTMENTAL AND DIVISIONAL CHANGE

Two reorganizational efforts have taken place within the Iowa Department of Transportation. The first was a department-wide reorganization that was completed in the fall of 1994. Prior to reorganization, the department was divided into several divisions: Air & Transit, Motor Vehicle, Administration, Planning, Rail & Water, and Highway. Each division reported to the Director of the Iowa Department of Transportation. The two divisions with the greatest impact on the preservation process were the Planning and Highway Divisions. The Highway Division was further broken down into several offices which reported to the Chief Engineer who was in charge of the Division. These offices included Road Design, Bridge Design, Right of Way, Contracts, Maintenance, Materials, Construction, and Local Systems. Additionally, the state was divided into six districts lead by a district engineer who also reported to the Chief Engineer.

Two of several significant changes made within this reorganization were the creation of three divisions from the former Highway Division: the Maintenance Division, the Project Development Division, and the Engineering Division; and the creation of six management teams that would direct the actions of the six districts now called transportation regions. This management team no longer had a district engineer but was comprised of a development engineer, a construction engineer, a maintenance engineer, and a transportation planner.

These changes had a considerable effect on the preservation process. With the creation of the Maintenance Division, many questions were asked as to its role within the new structure. One question specifically addressed pavement preservation. Previously, the district engineer would review pavement preservation candidates with the transportation planner. These candidate projects would be submitted to the Office of Program Management within the Plan-

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ning Division for prioritization and eventual programming. The Maintenance Division's role in this process was strictly advisory. With the creation of the three new divisions, it was felt that the Maintenance Division was in the best position to determine the preservation needs of the system. The Maintenance Division would be viewed as the "owner/operator" of the system. The Project Development Division would be viewed as the consultant and the Planning Division would be viewed as the financial officer. As the "owner/operator," the Maintenance Division would determine the priority of preservation and major rehabilitation projects along with the year in which construction would take place. The Project Development Division would now be viewed as a consultant providing their expertise in suggesting alternatives and the associated costs. The Planning Division would essentially act as a financial officer reviewing cash flow and assuring fiscal responsibility. This change did not come without concerns from some about the ability of personnel within the Maintenance Division to accomplish this new role.

Maintenance Division Reorganization

With the Maintenance Division getting new or expanded responsibilities along with the Department's increased focus on quality improvement, it too was reorganized in April 1996. Previous to April 1996, the Division had one regional maintenance engineer in each of the six transportation regions. Reporting to each of the regional maintenance engineers were 3 or 4 resident maintenance engineers. Each of these engineers were responsible for a 4 to 5 county area of primary highways (approximately 400 centerline miles), and had 4 to 6 Maintenance garages within the residency. The resident engineer was responsible for all aspects of the highway system within their area. This included pavement maintenance, bridge maintenance, sign maintenance, access control, utility accommodations, snow and ice removal, etc. In addition to these technical duties, they were also responsible for the supervisory duties of a staff that ranged from 50 to 80 people.

The major change that was made was the creation of twelve new positions, Area Maintenance Managers. There would be two managers within each region and they would take over the operational and personnel responsibilities of the Maintenance garages. This would allow the resident maintenance engineers to apply more focus to the technical side of preserving the highway system.

EFFECTS OF THE CHANGE

What has been the effect of these changes on the preservation process? From a maintenance perspective, the effects have been very positive. Instead of having an advisory role in the preservation process, the Division is now in the driver's seat. The benefits of this are as follows:

- Maintenance personnel have always had a concern and focus on pavement preservation. The best person/persons to assign a task are those who have a strong desire or concern about that task. They will generally have the greatest drive and energy to apply to the process. The previous structure relied on the district engineer who was competent, but also had many competing interests.
- The preservation process has a tremendous impact on the development of the Maintenance Division's work plan and budget. The ability to choose and select preservation projects has added a level of stability to the Division's planning process.
- There is now a greater ability to coordinate maintenance activities with contracted projects such that greater efficiencies and effectiveness are achieved. The resources that we utilize to preserve the state's highways include local garages, a specialized state crew, contracted minor preservation projects, and contracted major rehabilitation projects. These items account for a $100 million annual investment into the system.
- With the divisional reorganization, we now have 20 field maintenance engineers that can devote a greater portion of their time to pavement preservation. They are in the best position to monitor pavement performance, develop preservation strategies, and coordinate the use of the various resources.
- Decisions are now being made by those closest to the pavements. They will feel the effects of the benefits or consequences of their choices.

Training

The assignment of the preservation process to the Maintenance Division did not please everyone. The main concern was that Maintenance personnel were not adequately trained. Some feared that decisions would not be based on sound engineering judgment, but rather projects would purely be selected for pavements that had high maintenance costs. In many cases, these pavements would be on our lower level of service (LOS) routes. Our experience thus far has been the opposite, with our Maintenance engineers being much more critical in selecting projects based on pavement condition, LOS, AADT, economic development, availability of resources, and potential benefit. While the above fear has not materialized, we are still addressing this concern by providing additional training to our Maintenance engineers. Some of the training being provided includes:

- Pavement Distress Identification - The purpose is to develop a common base of communication. When block cracking, fatigue cracking or longitudinal cracking are discussed, the hope is that we will all have the same picture or point of reference.
- Pavement Data Collection - The purpose is to develop an understanding of the processes used in data collection, to become knowledgeable of the available data, and to be aware of operational constraints.
- Pavement Rehabilitation and Treatment Strategies - The purpose is to enhance our skills in this areas and take advantage of recent research.
- Pavement Management Systems - The purpose is to develop an understanding of this management tool especially in the areas of network and project level optimization.
- Pavement Design - The purpose is to determine pavement needs more accurately which will lead to better cost estimates for planning purposes.
- Project Development Process - The purpose is to increase understanding of the Project Development Division's role in preparing projects from design, letting, and eventual construction. By understanding their processes, there will be a reduction in conflicts and delays.
- State and Federal Highway Funding - The purpose is to gain better understanding of program funding and of factors that im-

pact future fund availability.

Currently the top two courses have been completed and the next two will be completed by October 1998. A pavement preservation manual is being developed with all of this material to be used as a resource and to be used as a tool to train new engineers.

PRESERVATION PROGRAMS IN IADOT

The Iowa Department of Transportation has three different programs that address preservation needs. They are Interstate Maintenance (4R), Major Rehabilitation (3R), and Contract Maintenance (MP). Currently, the 4R program is the responsibility of the Project Development Division. The majority of the funds within this program are devoted to the reconstruction of interstate system. The majority of the preservation needs of the non-interstate highway system are addressed through the 3R and MP programs. These programs are administered by the Office of Maintenance Operations. Annually $50 million is expended in the 3R program while $14 Million is expended in the MP program. In general, the 3R program addresses the structural needs of the pavement through the use of asphalt overlays. The MP project focuses on maintenance treatments as follows:

- PCC Full and Partial Depth Patching
- Diamond Grinding
- Joint and Crack Sealing
- Crack Filling
- Thin Maintenance Surface Treatments
- ACC Overlays up to 2”
- Transverse and Longitudinal Joint Repair
- Various Other Treatments.

Distribution of Funds to Transportation Regions

Funds are distributed on an annual basis for both the 3R and MP programs. For the 3R program, funds were allocated in April 1998 for FY 2001, which for Iowa begins in July 2000. Funds were allocated for the MP program again in April 1998 but are for FY 2000. Our fiscal year runs from July 1 to June 30.

MAJOR REHABILITATION (3R)

The pavements are currently grouped into four levels of service, which will be changed to a file level system next year:

- Level Service A - Interstate System
- Level Service B - Major Arterial
- Level Service C - Minor Arterial
- Level Service D - Arterial Collector.

Every January, our Pavement Management System is queried for the Pavement Condition Index (PCI) of each pavement section in LOS B, C, and D. This index is a value from 1 to 100 with the larger index indicating a pavement in better condition. This index takes into account such factors as IRI, D-cracking, longitudinal and transverse cracking, and faulting.

Currently, the Highway Commission has directed that the following cutoff values by used in determining whether a pavement section be considered for rehabilitation. They are as follows:

Level of Service B - PCI < 60
Level of Service C - PCI < 50
Level of Service D - PCI < 40.

With the information from the Pavement Management System and the direction from the Highway Commission, a table is developed to distribute funds to the six transportation regions (see Table 1).

This table lists the miles of pavements within each LOS for each of the regions. Looking at the Central Iowa Transportation Region, we see that there is 75.8 miles of LOS B pavements out of 631.7 total miles below a PCI of 60. This value is then corrected for LOS B pavement that do not have a PCI value (some urban extensions) which in this case is 36.7 miles. The 36.7 miles of pavement is assumed to have the same percentage of deficiency (approx. 12%) as the rated pavements. Therefore, the corrected value is 65.8 + 4.4 for a total of 80.2 miles. All the miles below the cutoff for each level of service are then added. For the Central Iowa Transportation Center, 98.7 miles are below the PCI cutoff values out of the state total of 805.6 miles. For each region, their total miles below the cutoff are divided by the state total to determine what percentage of the 3R fund is distributed to them. Table 2 indicates the actual spread of the funds to each of the regions for the four year period of 1999-2002.

PREVENTIVE MAINTENANCE AND MINOR REHABILITATION (MP)

The distribution of funds for the MP program is similar to 3R but with some differences. Prior to 1996, the $14 million available was simply split six ways such that each region got approximately $2.3 million. This method has been used since the early 1980s. Concern had been expressed over this type of distribution because it did not look at the needs of the system or the resources that each region had available to them. In trying to determine a more appropriate distribution method, we had to consider the underlying purpose of the MP program. Was its purpose only to supplement the local crews? Was it to be a “stop gap” process to hold pavements together until major rehabilitation could be scheduled? Was it to focus only on types of work not easily or efficiently accomplished by local crews? What should be the relationship between preventive and corrective maintenance?

We felt that just trying to supplement local crews was not the most effective use of the funds and that trying to develop a distribution method based on staffing and resources would involve a level of complexity with minimal return and acceptance. A typical example of some difficulties was that some of our regions in less populated areas have smaller garages and this makes it difficult to perform some maintenance functions without combining staff. Therefore, these regions felt that they should have a greater portion of the distribution. The more populated regions with larger garages felt that with the higher traffic volumes, the difficulty in managing traffic control, and limited working windows that they should have a greater portion of the MP funds to address their needs. We were headed down a path where we would have to track ADT, garage staff, regional maintenance budget, pavements scheduled for major rehabilitation, etc. This would involve significant data collection and analysis of numerous factors that would have subjective weights. We asked ourselves what are we really gaining.

Others felt that the distribution of funds should be based purely upon pavement condition and that the process utilized for 3R distribution would be appropriate. The problem with using the method is that it would place more emphasis on pavements in poor condi-

TABLE 1 FY 1999 to FY 2002 3R Distribution

Region	Service Level	Total Miles	Miles Without PCI	Miles Under Target <60<50<40	Corrected Miles Under Target <60<50<40	% Under Target	Corrected % Under Target
CITC	B	631.7	36.7	75.8	80.2	10.3	10.0
	C	407.3	36.7	4.0	4.4	0.5	0.5
	D	484.2	76.9	12.2	14.1	1.7	1.8
Total		1523.2	150.3	92.0	98.7	12.5	12.3
NEITC	B	673.8	84.7	98.9	111.3	13.4	13.8
	C	614.2	44.7	57.0	61.1	7.7	7.6
	D	285.5	28.3	17.2	18.9	2.3	2.3
Total		1573.5	157.7	173.1	191.4	23.4	23.8
NWITC	B	681.8	62.6	63.4	69.2	8.6	8.6
	C	645.1	46.4	22.1	23.7	3.0	2.9
	D	415.2	35.0	19.3	20.9	2.6	2.6
Total		1742.1	144.0	104.8	113.8	14.2	14.1
SWITC	B	302.2	8.9	67.2	69.2	9.1	8.6
	C	541.2	35.4	17.7	18.9	2.4	2.3
	D	66.03	53.9	14.5	15.7	2.0	1.9
Total		1503.7	98.2	99.4	103.7	13.5	12.9
SEITC	B	770.2	72.7	108.6	118.9	14.7	14.8
	C	473.0	31.2	3.3	3.5	0.4	0.4
	D	515.8	41.4	12.8	13.8	1.7	1.7
Total		1759.0	145.3	124.7	136.2	16.9	16.9
ECITC	B	648.5	81.5	92.3	103.9	12.5	12.9
	C	512.1	57.2	35.8	39.8	4.8	4.9
	D	425.0	51.6	16.1	18.1	2.2	2.2
Total		1585.6	190.2	144.2	161.7	19.5	20.1
STATE	B	3708.3		506.2	552.7	68.6	68.6
	C	3192.9		139.9	151.4	19.0	18.8
	D	2786.0		92.1	101.5	12.5	12.6
Total		9687.2		738.2	805.6		100.0

tion than those in good condition. Therefore, the emphasis would be on corrective types of treatments.

The distribution process chosen was based on the simple premise that timely, preventive maintenance treatments would provide the most benefit for our system. The MP program is now broken down into two components: preventive and corrective maintenance. The funds would first be distributed based on preventive needs. The Pavement Management System is queried for certain types of pavements: two year old asphalt pavements, seven year old asphalt pavements, and seven year old PCC pavements; and the following assumptions are made:

- The two year old ACC pavements (the majority of which are overlays) will exhibit reflective cracking and should be routed and sealed.
- The seven year old ACC pavements have reached the point at which a thin maintenance surface should be applied.
- The seven year old PCC pavements should be resealed.

The miles of pavement in each category are determined for each region and the funds necessary to perform the recommended treatment strategy is distributed. Table 3 indicates the amount distributed to each region for 1999. The sum of $8,046,600 shows the statewide preventive needs for 1999 less the amount required for rout and seal. $1,000,000 will be set aside to be distributed for rout and seal candidates at the end of the FY1998 construction season when the asphalt overlay projects have been completed. These amount will be taken “off the top” before the corrective portion is distributed. The corrective portion is distributed according to pavement condition. The same process for 3R distribution is used.

TABLE 3 Preventive Maintenance, FY 2000

Region	Rout and Seal Miles	Rout and Seal Cost	Maint. Surface Miles	Maint. Surface Cost	Reseal PCC Miles	Reseal PCC Cost	Preventive Allocation
CITC		$0	43.63	$741,710	10.02	$80,160	$821,900
NEITC		0	22.94	389,980	22.95	183,600	573,600
NWITC		0	145.6	2,475,200		0	2,475,200
SWITC		0	82.7	1,405,900		0	1,405,900
SEITC		0	124.7	2,119,900	13.81	110,480	2,230,400
ECITC		0	18.5	314.500	28.14	225,120	539,600
							Total=$8,046,600

Table 2 Spread of Funds to Each Region, 1999-2002

Region	FY 1999 Actual	FY 2000 Actual	FY 2001-2001 Target	FY 2001 Guarantee	FY 2002 Guarantee
CITC	$3,850,000	5,570,000	6,130,000	3,060,000	2,450,000
NEITC	13,400,000	11,490,000	11,880,000	5,940,000	4,750,000
NWITC	8,300,000	7,790,000	7,070,000	3,530,000	2,830,000
SWITC	9,750,000	8,780,000	6,440,000	3,220,000	2,570,000
SEITC	9,450,000	7,740,000	8,450,000	4,230,000	3,380,000
ECITC	5,250,000	8,630,000	10,040,000	5,020,000	4,020,000

TABLE 4 Corrective Maintenance

Region	% Under Target	Remaining Fund	Corrective Allocation
CITC	12.3%	$3,953,400	$484,400
NEITC	23.8	3,953,400	939,300
NWITC	14.1	3,953,400	558,500
SWITC	12.9	3,953,400	508,900
SEITC	16.9	3,953,400	668.400
ECITC	20.1	3,953,400	793,500
		Total=$3,953,000	

TABLE 5 Total Distribution

Region	Preventive	Corrective	Total
CITC	$821,900	$484,400	$1,306,300
NEITC	573,600	939,300	1,512,900
NWITC	2,475,200	558,500	3,033,700
SWITC	1,405,900	508,900	1,914,800
SEITC	2,230,400	668,400	2,898,800
ECITC	539,600	793,500	1,333,100
		Total=$11,999,600	

Table 4 indicates the corrective allocation distributed to each region and Table 5 indicates the total allocation to each region.

After the allocation of funds for the 3R program, the process of project selection to programming takes place. This process is briefly described below. The field engineers in each region review and prioritize their proposed project candidates. This list is reviewed by the regional team and submitted to the Office of Maintenance Operations. Site visits are made by representatives from Office of Maintenance Operations and regional staff. Statewide priorities are determined by Office of Maintenance Operations. The statewide priorities generally mirror 80 to 90 percent of the projects submitted by the regions. The role of the Office of Maintenance Operations in this step is to address unique pavement needs or those not properly identified by the Pavement Management System. After candidate projects are selected, alternative concepts are developed by the Project Development Division with assistance from Maintenance and Materials personnel. A preferred alternative is selected and the project is ultimately programmed for construction. The time frame from project selection to construction generally takes from two to three years.

The MP process runs on a more compressed schedule. Additionally, MP projects do not need commission approval. The regions submit their proposed projects and they are reviewed by the Office of Maintenance Operations as to their category: preventive or corrective. If they reasonably comply with the allocations set for each category, proposal documents are prepared within the Office of Maintenance Operations and the project is let for construction. The time frame from project selection to construction generally takes from 6 months to one year. Figure 1 shows the two processes in greater detail.

FUTURE GOALS

Items that still need to be addressed or investigated in the future are:

- Combination of the 4R (preservation portion), 3R, and MP programs into one preservation program;
- Refinement of Pavement Management Optimization Techniques and utilization of this tool in project selection;
- Investigate and determine appropriate location of the Department's Pavement Management section;
- Development of a six member team comprised of field maintenance engineers to determine a process for statewide prioritization of preservation projects.

We still have many tasks to accomplish on the way to enhancing our preservation process. A major step was accomplished by giving those closest to the situation greater decision making powers, but additional authority must and will be given. This can only be accomplished by building a shared vision of the future, providing appropriate training, and by having a willingness to take risks by relinquishing control and empowering others.

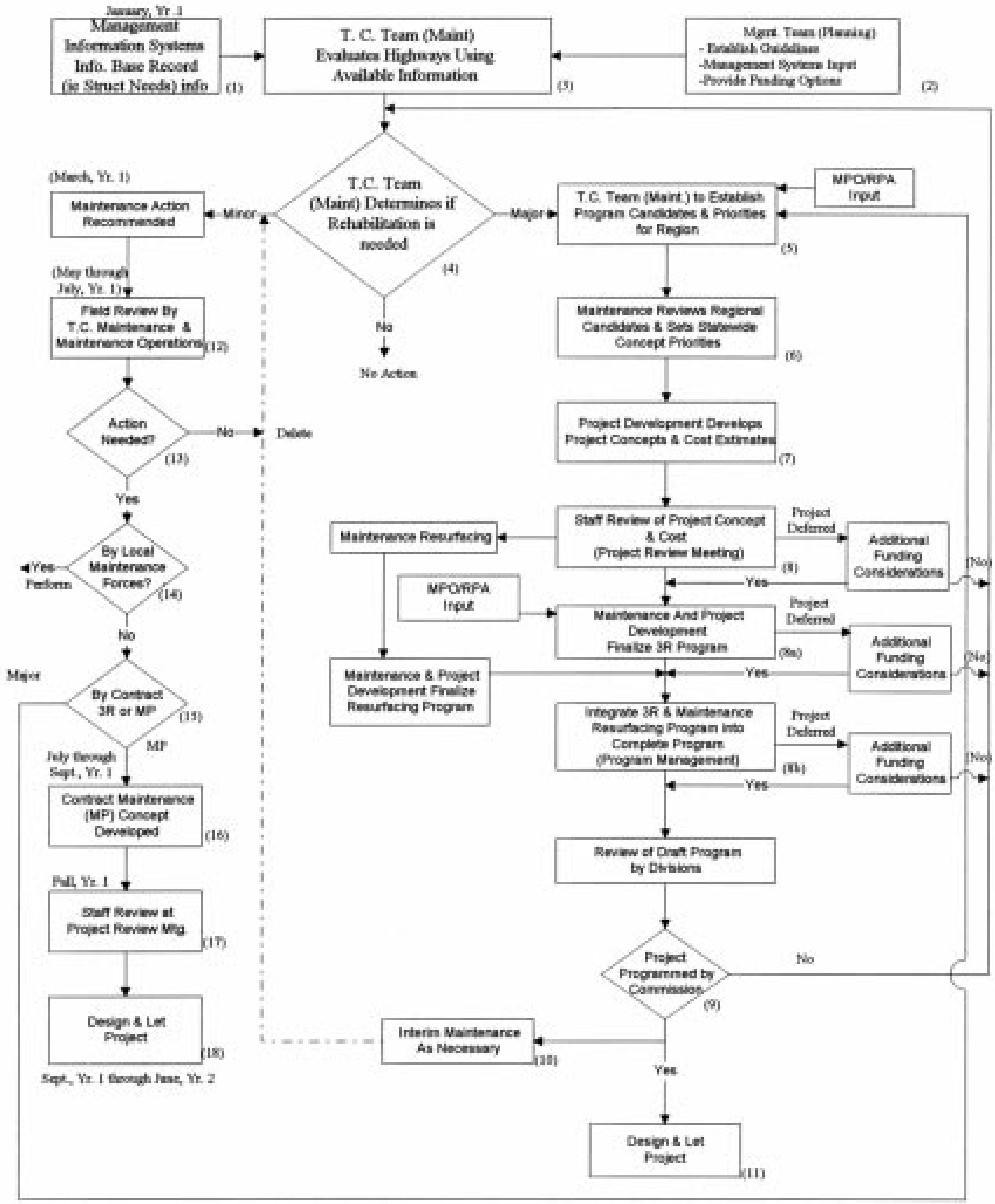

FIGURE 1 Major (3R)/Minor (MP) project rehabilitation process.

Pile Bearing in Burlington Limestone

Brett Gunnink and Chad Kiehne

Three field load tests of drilled shafts socketed in Burlington limestone were conducted using the Osterberg load cell. The objective of these tests was to compare the shaft capacities obtained from the field load tests with capacities predicted using analytical methods and with typical presumptive design capacities. It was believed that the actual capacities of the drilled shafts would be considerably greater than the capacities predicted from presumptive bearing capacity values. Based on the results of this testing, the following conclusions were drawn. Observed values of side resistance are comparable to the predicted values obtained from empirical relationships. Observed values of end bearing pressure greatly exceed the presumptive values of allowable bearing capacity commonly used for the design of shafts bearing on Burlington limestone. The test shafts were not failed in end bearing and it is believed that the ultimate end bearing pressures would significantly exceed the observed end bearing pressures. The actual factors of safety of shafts in Burlington limestone that are designed for end bearing only, using typical presumptive end bearing capacities, will exceed 6. Side resistance will carry a large portion of the load and for service loads, the entire load may be carried by side friction. Key words: foundations, drilled shafts, Osterberg cell, load test.

INTRODUCTION

It is common engineering practice to design rock-socketed drilled shafts for end bearing only, based on conservative presumptive values of allowable bearing capacity. For example, for the Burlington limestone studied in this paper, a typical allowable bearing capacity is 1914 kPa (40,000 psf). The use of conservative values is due in part to the lack of full scale field load test data that would allow for the validation of less conservative design procedures. Often, site investigations terminate at auger refusal, in which case only the location of the rock is known and very little is known about rock strength. Further, the difficulty and cost of performing full scale load tests of drilled shafts in rock, limits the amount of data available for design procedure validation. Recently, the development of the Osterberg load cell provided a more economical means for conducting load tests. To date, the Osterberg load cell has not been used extensively in Mid-America and particularly it has not been used extensively in limestone.

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OSTERBERG LOAD CELL

The Osterberg load cell was developed and patented by Dr. Jorj Osterberg (*3*). The Osterberg load cell is a static load testing device for shafts and piles. An Osterberg cell load test uses an especially designed hydraulic to create pressures in excess of 55 MPa (8,000 psi) at the bottom of the shaft, loading the pile or shaft in end bearing and upward side resistance.

The Osterberg load cell is lowered into the shaft via the reinforcing cage or if no reinforcement cage is used, a small I-beam or channel can be used to place the load cell. The hydraulic lines and telltale rod casings are also attached to the reinforcement cage. The telltale rods allow for the measurement of the movement of the bottom and the top of the cell. These movements and the movement of the top of the shaft or pile are measured using dial gages supported by an independent reference beam.

The Osterberg cell is pressurized using a compressed air driven pump with diluted automotive antifreeze as the hydraulic fluid. The soil and/or rock surrounding the shaft or pile provides the reaction for the load test. As the cell is pressurized, the bottom of the cell moves downward, testing end bearing, while the top of the cell moves upward, testing side resistance. Schmertmann (The Bottom-Up, Osterberg Cell Method for Static Testing of Shafts and Piles, *unpublished data*) fully discusses the advantages and disadvantages of the use of the use of the Osterberg load cell.

TEST METHODS AND PROCEDURES

Shaft Excavation

Hayes Drilling Inc. of Kansas City, Missouri began shaft construction on December 9, 1996. Three shafts were excavated using a truck-mounted rotary drill. A 45.72 cm (18 in.) diameter auger bit with carbide cutting teeth was used to excavate the overburden as well as the rock socket. Water was used as lubrication during the drilling process and to facilitate the removal of the rock cuttings. The base of the socket was cleaned by rapidly spinning the auger bit after the addition of water and then lifting out the rock cuttings.

Osterberg Cell Assembly and Placement

The Osterberg cells used in the base of the three shafts were 33 cm (13 in.) in diameter and approximately 31.75 cm (12.5 in.) high. The cells had a maximum load producing capability of 4000 kN (450 tons). Figure 1 illustrates the Osterberg cell assembly.

After the completion of drilling, a small seating layer of concrete was placed by free fall into the base of the shaft. The Osterberg

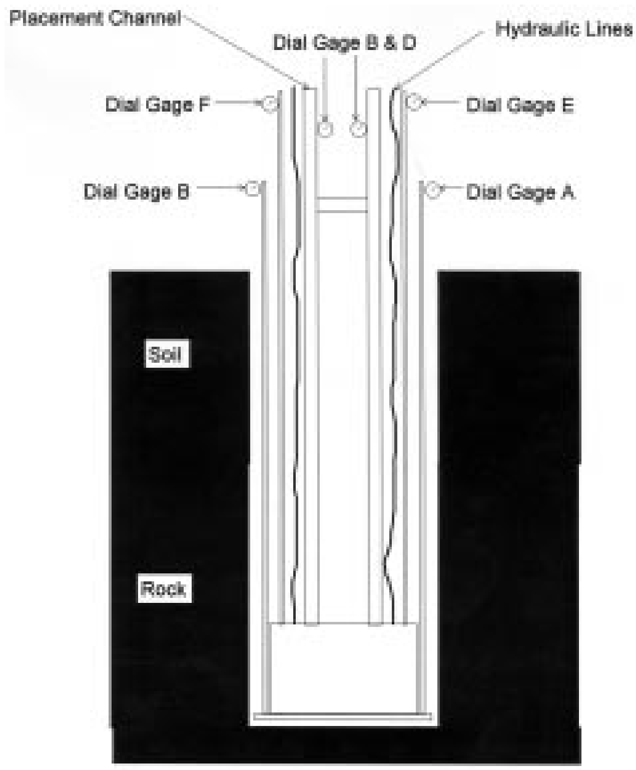

FIGURE 1 Osterberg load cell and movement measurement schematic.

cell base plate was greased to ensure no concrete adhesion. The cell was then lowered into the shaft and seated onto the base layer of concrete. The remaining concrete was then placed by free fall into the shaft. Three concrete test cylinders were made for each shaft so that the strength of the concrete could be measured. The concrete was allowed to cure for 6 days before the load test was performed. The average concrete strength at the time of shaft testing was 47.2 Mpa (5300 psi).

Load Test Procedure

A steel channel reference beam was placed near the drilled shaft assembly. Six digital dial gages were attached to the reference beam or steel channel by magnetic stands. The dial gages were designated A through F (see Figure 1). Machined steel telltale rods were inserted into the telltale casings. Dial gages A and B measured the downward displacement of the base plate telltale rods and dial gages C and D measured the upward displacement of the top of shaft. Dial gages E and F were attached to the channel frame and measured the displacement between the top of cell telltale rods and the top of the shaft or otherwise stated they measured the compression of the shaft.

The Osterberg cell was pressurized in increments of approximately 3445 kPa (500 psi). The pressure was held at each loading increment for a total of 4 minutes. The load increments were increased until side friction shear failure occurred.

RESULTS AND DISCUSSION

Site Investigation

The initial site investigation consisted of collecting eight previous subsurface investigations that were performed in the general vicinity of the three research shafts. These investigations were performed from 1988 to 1995 by Engineering Surveys and Services of Columbia, Missouri for the purpose of new construction.

The overburden consisted of mostly glacial drift. This ranged in depth from zero to over 6 m (20 ft.). The drift consisted of sandy clay, sandy silty clay, gravelly clay and is sometimes underlain by Pennsylvanian shales. These materials are underlain by massive Mississippian limestone bedrock.

Burlington limestone bedrock depths in the area range between 1.8 and 12.8 m (6 and 42 feet). The surface of the limestone is irregular and weathered in some areas. The weathered layer varied in thickness from a few centimeters to over a meter.

Three unconfined compression strength tests of Burlington limestone core samples show a 43.6 MPa (6,336 psi), 73.8 MPa (10,718 psi), and 64.7 MPa (9,395 psi) rock strength. Four core samples provided rock quality designations (RQD) and percent recoveries These include a 90 percent recovery with a 78 RQD, a 100 percent recovery with an 80 RQD, a 100 percent recovery with a 100 RQD, and a 100 percent recovery with an 85 RQD.

During the drilling of the shafts glacial till was found at the surface. It was predominantly clay with some silt, sand and gravel. No shale was found during the drilling process. However, a thin layer of weathered limestone was encountered on top of the limestone bedrock.

After completion of the shafts, a feeler gage was used to scrape the sides of the socket in order to find seams or fractures. Small seams were found in shafts 1 and 2 but no seams were detected in shaft 3. No ground water was encountered in any of the shafts. The depth to limestone was 4.12 m (13.7 ft), 4.02 m (13.2 ft), and 3.77 m (12.4 ft) for shafts #1, #2 and #3, respectively.

Downward End Bearing and Upward Side Resistance Load Movement Curves

The downward end bearing load movement curves were obtained directly from dial gages A and B, which measured the difference between the displacement of the reference beam and the telltale rods extending to the base of the cell. The upward side resistance movement was obtained directly from dial gages C and D, which measured the difference between the displacement of the reference beam and the top of the shaft. The side resistance load is the net load, calculated by subtracting the weight of the shaft from the cell load. The loads for the downward end bearing movement are the cell loads.

Shafts 1 and 3 were loaded until side resistance failure occurred. Shaft 2 was initially loaded to about 1000 kN (120 tons) and then unloaded due to a an equipment malfunction in the hydraulic pump. Shaft 2 was subsequently reloaded until side resistance failure occurred. Figure 2 shows the Osterberg cell load movement curves for shaft 3. The upward shear movement curve is typical of side resistance failure. Side resistance failure occurred at 3500, 1500

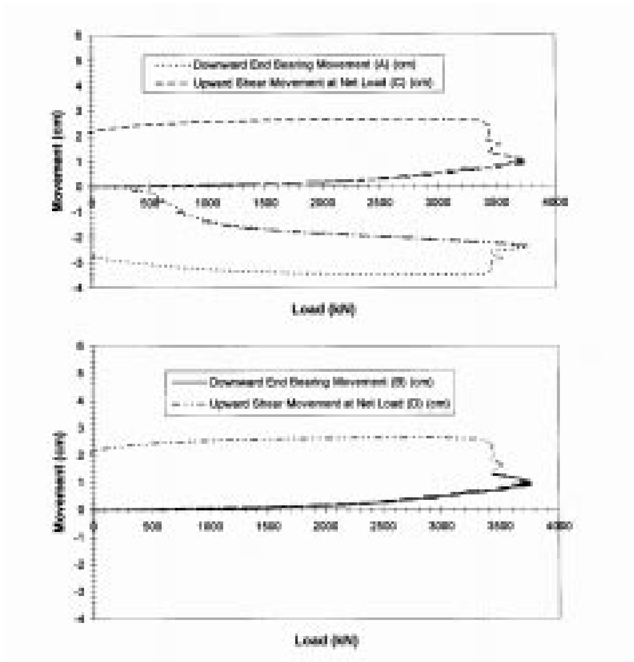

FIGURE 2 Osterberg cell load movement curves for shaft #3.

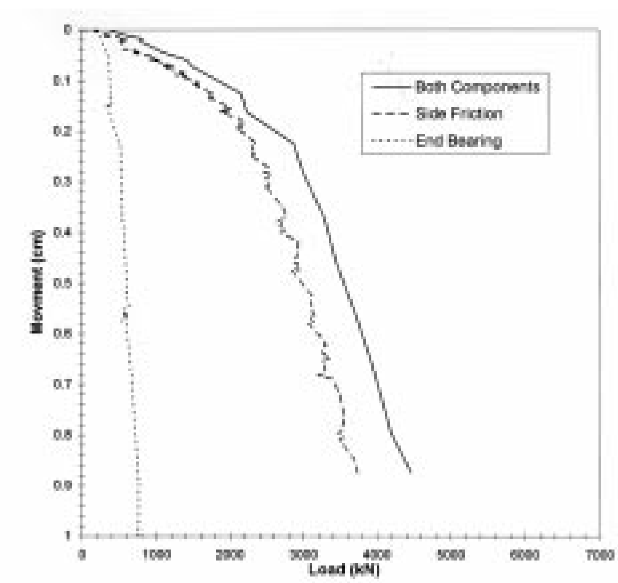

FIGURE 3 Equivalent load movement curve for shaft #3.

and 3800 kN for shafts 1, 2, and 3 respectively. The downward end bearing movement curve show some interesting anomalies. For shaft 3, it appears that the dial gauge B telltale casing became plugged and as a result the telltale rod did not move down with the bottom of the load cell, but rather up with the shaft. Finally, the downward end bearing movements curve shows continuing downward displacement of the load cell after side resistance failure has occurred. This is possible only if simultaneous end bearing and side resistance failures occur; which seems highly unlikely. It most likely indicates that after side resistance failure, ground movement at the surface raised the elevation of the reference beam.

Reconstructed, Equivalent Top Load Movement Curve

Reconstructed, equivalent top load movement curves can be developed by adding side resistance movement data and end bearing movement data. Goodwin (Bi-Directional Load Testing of Shafts to 6000 Tons, *unpublished data*) indicates that the reconstructed curves will represent the load movement of a shaft loaded in the conventional field load test manner if 1) the end bearing load movement in a conventionally loaded shaft is the same as the load movement curve developed by the bottom of the Osterberg cell, 2) the upward side resistance movement curve for the Osterberg cell test is the same as the downward side resistance movement in a conventionally top loaded test and 3) the compression of the shaft is considered negligible and the shaft is assumed rigid.

Equivalent load movement curves were reconstructed up to the maximum test load of 6524 kN (733 tons) for shaft 3. The reconstructed equivalent top load movement curve for shaft 3 is presented in Figure 3.

Observed End Bearing Pressures and Side Resistance

The maximum side resistance of the three shafts was reached and therefore can be compared directly with predicted side resistance values. Due to the limitations of the bi-directional loading of the Osterberg cell the maximum capacity in end bearing was not reached.

Side resistance is typically predicted using empirical relationships between side resistance and either concrete or rock strength. Williams et al. (*7*) and Rowe and Armitage (*8*) provide relationships developed for used with limestone rock. The predicted side resistance capacities were calculated using a concrete strength of 47.2 Mpa (5300 psi) rather than the higher unconfined compressive strength of the rock. The lower value should be used when calculating predicted side resistance because side resistance failure will occur in the lower strength material. The predicted side resistance using the Williams relationship is 1550 kPa (225 psi) and using the Rowe and Armitage relationship it is 1252 kPa (181 psi). The observed side resistance values for shafts 1, 2 and 3 respectively are 2343 kPa (340 psi), 916 kPa (133 psi), and 2278 kPa (330 psi).

The predicted values of side resistance are significantly lower than the values of side resistance observed from shafts 1 and 3. The side resistance value observed from shaft 2 is lower than predicted values. Based on this data, the authors conclude that the empirical relationships are adequate if typical design factors of safety are used.

Due to the limitations of the Osterberg cell it was not possible to reach the maximum end bearing capacity. Since the Osterberg cell loads the shaft from the bottom, the applied load can only be as large as the load bearing mechanism with the lowest capacity. In the case of shafts 1, 2, and 3 it was side resistance. The observed end bearing pressures at termination of testing were 21.4 MPa (3112 psi), 9.1 MPa (1320 psi) and 22.9 MPa (3325 psi) for shafts 1, 2, and 3 respectively.

CONCLUSIONS

It is common engineering practice to design rock-socketed drilled shafts for end bearing only, based on conservative presumptive values of allowable bearing capacity. For the Burlington limestone studied in this paper, a typical allowable bearing capacity is 1914 kPa (40,000 psf).

It is also typical to specify that shafts be socketed 0.61 m (2 ft) into sound rock. The conservatism of this approach to design can be illustrated with the following example.

Given a design shaft load of 2670 kN (300 tons) and an allowable end bearing pressure of 1914 kPa, the shaft would have a design diameter of 1.37 m (54 in.). Using the lowest observed value of side resistance, 916 kPa (133 psi), the side resistance capacity of the shaft would be 2409 kN (270 tons). Based on the lowest observed value of end bearing pressure, 9.1 MPa (1320 psi) the end bearing capacity would be 13,448 kN (1511 tons) and probably much larger. Therefore, particularly at service loads, the shaft load would be carried almost entirely by side resistance and the actual factor of safety would be greater than 6, possibly much greater.

ACKNOWLEDGMENT

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Using NDT to Reduce Traffic Delays in Concrete Paving

JAMES K. CABLE

Fast track paving has centered on the use of construction materials and methods to improve the rate of placement and curing to reduce the traffic delay time. The State of Iowa has been able to make large improvements in the fast track process to meet target traffic delay constraints through material selection and construction methods. At the same time the methods for monitoring concrete strength gain and quality have not changed. In 1995, Lee county and the Iowa DOT cooperated through research project, HR-380, to construct a 7.1 mile project to demonstrate the use of maturity and pulse velocity nondestructive testing methods to estimate concrete strength gain and reduce traffic delay. The results of that work identified the pros and cons of each method and suggested specifications to meet traffic delay demands. The results also identified examples of equipment that could easily be used by project personnel to estimate the concrete strength using the maturity methods. Key words: NDT, maturity, pulse velocity, concrete strength, traffic delay.

INTRODUCTION

Fast track paving is not new to Iowa. It originated here as a way to make concrete a viable paving alternative where traffic delays were a critical factor in the selection of pavement type. Prior to 1995, research efforts in this area had centered on the mix design, and curing methods employed to reduce traffic delay and provide a concrete pavement alternative. The time of opening to general and construction traffic was still related to flexural beam tests and time after paving. This meant that five to seven days were often required to meet the specification and allow for traffic use of the new pavement.

The Iowa Department of Transportation (IDOT) and Iowa State University, Civil and Construction Engineering (ISU/CCE) researchers had conducted various small projects to evaluate the use of nondestructive testing (NDT) devices on several projects in the early 1980's. The largest of these involved the placement of a bonded overlay of Iowa Highway 3 in Franklin County in 1994. The results of that work encouraged the Lee County Engineer to approach the researchers with a proposal to evaluate the use of pulse velocity and maturity concepts to estimate the flexural strength in the construction of 7.1 miles of X-28 (Great River Road) near Keokuk, Iowa.

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PROJECT GOALS

The research team of county, state and university representatives was able to identify the following six objectives for the research:

1. Development of a knowledge base for applying maturity and pulse velocity measurements to determine concrete strength for traffic opening.
2. Identification of effective and efficient monitoring equipment that could relate concrete strength gain to time and temperature measurements in the field.
3. Evaluation of maturity measurements at various pavement depths to understand the effect of the subgrade and temperature gradients on strength gain.
4. Identification of NDT equipment and methods for rapid employment by field staff to monitor concrete strength gain and determine traffic opening times.
5. Relate the early opening to visual pavement distresses found in the first two years of pavement operation.
6. Development of an instructional memorandum for use by field staff in the operation of NDT equipment for measuring pavement strength gain.

DATA COLLECTION

This project marked the first time that maturity base curve development occurred at the project site. A curve was developed at the Iowa DOT Central Laboratory in Ames, Iowa to act as a baseline set of information. It utilized project cements and aggregates from the construction site. On the first day of construction, in late September 1995, a truck load of concrete was selected at random from those supplying material to the paver. Test flexural beams were constructed from a portion of that load and one was instrumented with the maturity meter thermocouple. Beams were placed in a wet sand bed and monitored by the maturity meter. Individual beams were tested by center point loading over the first 24 hours of cure and related to maturity time-temperature factors. This data formed the basis for opening strength decisions for the remainder of the project. A similar test utilizing the pulse velocity meter and compressive cylinders was also carried out by the Southeast Iowa DOT Transportation Center Materials Laboratory.

The research team was able to utilize maturity measuring devices (recording and nonrecording) at 500 ft (152.4 m) intervals to determine field rates of strength gain along the pavement during construction. Tests were conducted with thermocouple wire attached to wood dowels inserted at various depths in the concrete. The maturity near the concrete surface (1 in., 25 mm) was used to identify the potential time for transverse joint development/sawing. Maturity at the midpavement depth (3.5 in., 90 mm) was used for make the decisions on pavement opening to traffic by local officials. Monitoring of each site was continued until the pavement estimated strength exceeded 500 psi (3.41 MPa) that required for opening to general traffic.

The maturity concept was employed on the mainline paving which consisted of a slipformed slab measuring 7 in. (180 mm) in depth and 22 ft (6.7 m) in width, on an existing granular base. Paved bikeway shoulders measuring 5.5 ft (1.7 m) in the urban area and 4.0 feet (1.2 m) in the rural areas were also monitored for strength gain using the maturity measuring devices.

Maturity data was collected by two persons. The first person stayed with the paving operation and installed the recording meters at the beginning and end of each day's placement. They also placed the thermocouples at the 500 ft (152.4 m) intervals for measurement with the handheld digital thermometers. The second person used the digital thermometers to both collect data from the current day installed thermocouples and those installed on previous days. Access was gained by the use of an all terrain vehicle on the new slab. This system proved to be the answer to working with a fast moving paving operation in a narrow right of way.

An elaborate system was developed by the research staff to gain access to the pavement for use of the pulse velocity geophones. This involved insertion of three sections of large metal tubing in the pavement, removal of the concrete in the tubes, removal of the tubes, measurement of the pulse velocity and filling of the holes with fresh concrete. This process did not prove to provide sufficient and accurate information for this type of operation and the process was discarded after the first day of paving.

SUPPLEMENTAL TEST RESULTS

In addition to the maturity and pulse velocity data, information on the relative humidity and ambient air temperature was collected with handheld devices on this project. The results of that data collection were inconclusive in regard to developing a relationship to maturity values.

Visual distress surveys were conducted at three times during a two year period to determine the potential impact of early pavement opening to general/construction traffic. Minor cracking was found in areas of the shoulder where the joint had not been formed in a timely manner. Longitudinal and corner/diagonal cracking were noted in isolated outer wheel path areas and is attributed to uneven settlement of the subgrade between the original roadway and the shoulder widening unit.

Deflection testing of joints at 0.2 mile (0.3 km) increments indicated joint transfer values of greater than 80% in all areas but three. These values are very good for nondoweled pavements such as the test pavement. Three low values found in shoulder widening areas of the subgrade may be the result of inconsistencies in the support across those longitudinal subgrade joints.

Midslab deflection testing resulted in the calculation of subgrade soil support "k" values of 97-225 psi which is representative of this soil type. These same measurements were used to backcalculate the thickness of the concrete. In all cases it indicated, as did the field measurements during construction, that 7 in. (305 mm) was constructed.

CONCLUSIONS

The research resulted in many advancements for the Iowa DOT, the concrete paving industry and the traveling public. They are summarized in terms of the original objectives of HR-380.

1. Information DataBase Development
   a. Mainline and shoulder construction maturity data is now available for multiple days of construction on this project during one time of the construction year.
   b. Data from other Iowa DOT Centers has provided the administration with knowledge of the implications of using maturity for new construction, pavement repair and reconstruction and development of an Instructional memorandum.
   c. Data from the Lee County project was limited minimal changes in air temperature and humidity.
   d. The Lee County project did emphasize the importance of curing methods during the first 24-48 hours after construction, on the rate of slab strength gain.
2. NDT Equipment Selection
   a. Pulse Velocity is not recommended for use in pavement strength gain monitoring due to equipment/space needs, field construction limitations and operator training requirements.
   b. Recording maturity meters should be applied for research purposes only due to their susceptibility to weather damage and theft.
   c. Digital thermometers are very successful in the collection of maturity data by one person. They can be used with or without the application of special connectors to the thermocouple wire.
3. Maturity Depth and Location Measurements
   a. Place thermocouple wires near the pavement surface to estimate strength relative to transverse joint development decisions.
   b. Place thermocouple wires near the pavement middepth to estimate pavement strength relative to traffic opening decisions.
   c. Thermocouple installation should be made at a location 1 ft (305 mm) from the pavement edge and along wood dowels inserted to the appropriate depth to aid in construction and traffic operation decisions.
4. Pavement Visual Distress Relationships
   a. Transverse cracking in paved shoulders was attributed to not forming the joint early enough after paving.
   b. Minor longitudinal and corner/diagonal cracking in the mainline pavement, outer wheelpath, was attributed to irregularities in the subgrade construction.
5. Maturity Instructional Memorandum Development
   a. I.M.383 date October 28, 1997 includes the results of this research effort, Iowa DOT enhancements of early versions of the I.M. developed in 1995-1996 and comments solicited from field staff by the research team.
   b. Development of maturity curves at the project site at the beginning of the project are recommended for project control. Validation curves at scheduled intervals or when any of the project materials changes, is recommended.

This project has resulted in the development of a new tool for Iowa in the search for meeting the public travel demands with proven materials and sound engineering.

Implementing Benchmarking Recommendations in the Offices of Construction for the Iowa DOT

MARK O. FEDERLE AND CHARLES T. JAHREN

The Iowa DOT's Offices of Construction are seeking ways to use benchmarking, the concepts of quality management, and outside facilitation to improve their methods and processes. ISU researchers and the Offices of Construction Benchmarking Steering team have developed a performance measuring system and have collected considerable baseline data. The baseline data has been examined and process improvement teams have been launched in areas that require improvement. In many cases, process improvement team recommendations have been implemented. This paper will present the results of those process improvement team efforts. Keeping continuous improvement efforts on track for many organizations is a challenge. It is easy to develop a vision and mission statement and generate enthusiasm, only to have the efforts die when participants discover the amount of effort and persistence required to continue the efforts. The Iowa DOT Offices of Construction has engaged in a quality improvement effort. The effort is expected to continue for the foreseeable future. The authors will share their experiences regarding starting and supervising PIT teams, including writing charters and member selection; forming a steering team that represents a vertical slice of the organization; encouraging team participation, including the technical staff; helpful interactions between the facilitators and the technical staff; design and use of employee surveys to support the improvement efforts. These experiences will be helpful to others who are participating in quality improvement efforts.

INTRODUCTION

Quality leadership helps organizations to meet expectations in a rapidly changing world. Emphasizing the quality of the organization's products and services in effort to meet customer needs does this. First, measures of quality are developed and monitored; then continuous improvement is used to enhance quality.

Many organizations start on the road to quality leadership and take the first few steps. Then somewhere along the way, the organization loses its quality focus and returns to business the way it was previously done. The Iowa DOT Offices of Construction started its journey in quality leadership in May of 1995 with the formation of the Benchmark Steering Team. Since then the steering team developed a mission statement, a set of key functions and performance measures; it has collected performance data three times (*1*,*2*). Four process improvement teams (PITs) have been launched. They have drawn membership from the entire state. The teams have completed studies and made recommendations and these recommendations are being implemented. Several Work-Unit PITs have been recently organized in the construction field offices. They are recommending improvements for more specific problems. The organization is firmly committed to quality improvement after three years of activity.

PROCESS IMPROVEMENT

Process improvement efforts started after performance measurements for the first year were reviewed; an explanation of the key measures is provided in Chase et al. 1996. The key functions with the lowest ratings on the Offices of Construction Employee Survey were *Resolution of Technical Issues* and *Providing Pre-letting Information. Providing Pre-letting Information* also received a marginal rating from Iowa DOT employees outside of the Offices of Construction. The Offices of Construction Employees help with plan development by collecting pre-letting information. Typical tasks involve tabulating cracks for repairs and surveying wooded areas for clearing density calculations. Process improvement teams (PITs) were launched in each of these areas. Members were selected from a list of people who indicated an interest in serving on such teams when they responded to the first Offices of Construction employee survey. Care was taken to obtain a vertical slice of participants and provide representation from various geographic areas. One lesson learned from commissioning these two PITs is that more care was required in chartering the teams. Direction in what the steering team wanted was crucial to ensure the PITs were effective and efficient in their use of time.

The Benchmark Steering Team decided that it could do a better job of writing charters and assisting PITs if they performed process improvement studies themselves. They selected three areas that flowed out of the PITs recommendation and their own discussions: developing a list of contacts for technical problems, pavement smoothness specifications, and PCC patching problems. Using resources provided by several members, the Steering Team developed the list of contacts. The pavement smoothness review resulted in recommendations that specific areas be clarified in an instructional memorandum. As a result of the PCC patching study, a PCC patching fact sheet was developed that combined information from several sources including the specifications, the standard

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drawings, and the *Construction Manual*. This fact sheet was distributed to the construction field offices. The steering team actively participating in quality improvement was important in helping the team understand more fully the quality improvement process.

Next, a process improvement team was launched to develop recommendations to improve contractor concern regarding safe traffic control. Following the recommendation of the Quick Draw Resolvers, this PIT was set up as a regional group. They were chartered as follows:

1. This process improvement team is charged with improving the effectiveness of the temporary traffic control zones. Temporary traffic control zones are considered effective when they are safe for the traveling public and workers and administratively efficient for the Iowa DOT and contractors. The cost and scope of work should be reasonable and well defined by the plans and specifications.

They made the following recommendations:

1. Require that a certified traffic control coordinator be present on the project whenever work is being performed.
2. Noncompliance penalties will be coordinated on a statewide basis to ensure that they are applied uniformly. A flow chart provided additional assistance.
3. An evaluation form for contractor traffic control was provided.
4. Work zone safety training should be increased.
5. A traffic control information directory should be included in Iowa DOT's Construction Manual.

Written comments from the Offices of Construction Employee Survey, *Documentation of Work Progress and Pay Quantities* key function indicated a considerable amount of concern regarding the Electronic FieldBook, a new computer system. The FieldBook allows field personnel to record work progress by using notebook computers in the field. Bi-monthly contractor payments are issued using this program. The written comments pointed out several difficulties in operating the system and dissatisfaction the documentation, training, and amount of help available. A PIT was chartered to review this situation.

The team flowcharted the entire operation from generating a template for a particular project to the final payment for the contractor. Then they recommended several changes that ranged from quick fixes to major program revisions. It is expected that most of the recommendations will be implemented.

During discussions with the Benchmark Steering Team, concerns about risk management during inspection became apparent. The construction budget for the Iowa DOT has been steadily increasing in the recent past. Meanwhile, the number of Offices of Construction employees has held steady or fallen slightly. Although this has reduced the cost of inspection as a percent of contract cost, it has resulted in an increased workload for the staff. Traditionally, an inspector was able to watch every construction operation to make sure the work was being correctly installed. Today this is not possible. Therefore, the inspector must prioritize tasks and concentrate on the most important item while spot-checking everything else. Inspectors would like to have more guidance on how to prioritize their time. Also, it is vitally important that the Iowa DOT limit inspection and testing to only the most important items that have the greatest impact on quality.

A PIT has been chartered to assess which aspects of PCC paving have most influence on quality. This PIT will review research publications and filter the findings with their experience to develop recommendations that are tailored for the Iowa DOT.

Process Improvement in Field Offices

The Benchmark Steering Team saw many opportunities for quality improvement that could be harvested by launching PITs in the construction field offices. Launching PITs in the field offices would also increase involvement in and understanding of quality improvement. This action would also allow study of many more topics and would reduce travel time. The Benchmark Steering Team encouraged the field offices to develop a list of interesting topics and to choose one or two; a field office PIT would be launched to study each choice. The field offices were especially encouraged to tackle narrow technical issues that repeatedly caused problems. Assistance for meeting facilitation and topic selection was available from the writers. Five studies were launched, three examples are provided below.

Joint and Crack Sealing Work-Unit Improvement Team

One office felt that the procedures for measurement and payment of joint and crack sealing were overly complex. Although there were several different classifications of cracks, contractors often bid the same unit price for some of the classifications. This suggested that some of the classifications were unnecessary. They also pointed out that measuring the crack length required them to be away from the area where the work was being done, thus compromising their effectiveness as inspectors. This group made recommendations for changes to the specifications. These changes are currently being sought.

PCC Pavement Removal Work-Unit Improvement Team

Another group sought out a more efficient way to measure PCC Pavement removal. Currently considerable time is required to manually measure the number of square yards of driveways, parking lots and sidewalks. This did not seem reasonable because when plans were developed, the measurements are made to determine the plan quantity. This group developed a process whereby PCC paving removal would be paid for by plan quantity. The original survey notes would be transferred to the construction field office. Field forces would only need to note changes that occurred after the original survey and check for calculation mistakes.

Field Fences Work-Unit Improvement Team

Field fences are an item that the Iowa DOT occasionally builds. Several details exist for field fence construction in the standard plans. However, these details were developed long ago, and fence-making practice has changed since then. Also, since field fence is rarely built, inspectors cannot remember the details from one time to the next. A field office PIT reviewed this problem and recommended updated standard plans for field fences.

In general, the field office PITs were successfully launched when the leadership was interested in quality improvement and the team selected a topic that was of interest to itself. Some field offices produced products that can be shared with and benefit other field offices, while others developed improved methods for internal of-

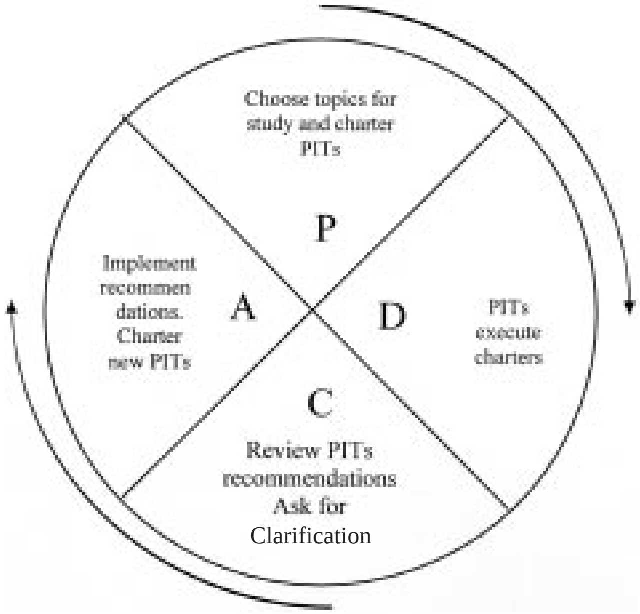

FIGURE 1 Plan-Do-Check-Act cycle for quality improvement.

fice management; it is possible that these methods could be modified and applied at other offices.

REASONS FOR SUCCESS

During the past three years, the quality improvement process of the Iowa DOT has gone through many peaks and valleys. Enthusiasm is easy to generate, but hard to maintain. PITs are easy to empower, but providing adequate and detailed direction requires significantly more work than saying, "Go solve this problem!"

Resource constraints, especially hiring limits, make continuation of the process challenging. We have found that these challenges can be overcome if five factors are constantly and consistently addressed.

Top Leadership Commitment. The top leadership of the Iowa DOT and the Offices of construction strongly encourages quality activities. The state construction engineer leads the Office of Construction. He chose to lead the start-up of quality activities for this office. The director of the Iowa DOT led the start of quality activities for the Department before they were started specifically in the Office of Construction. The Development Division Engineer occupies the level of authority between the State Construction Engineer and the Director. He has also actively encouraged quality activities.

Previous Training. Most people in the Iowa DOT have had training regarding quality activities. In general, Iowa DOT employees have good meeting skills.

Quality Activities in Iowa DOT. The Iowa DOT has PITs operating in areas surrounding the Office of Construction. These support Office of Construction activities in three different ways. First, many of the PITs include Office of Construction Employees. This gives them training and understanding in quality processes. Another is that the other PITs provide input information for Office of Construction PITs and the Benchmark Steering Team. Finally, the other PIT teams often try to solve problems that have been identified by the Office of Construction, especially if the problems cross the boundaries of several offices.

Full Participation in Discussion by Entire Steering Team. During the monthly meetings of the Benchmark Steering Team, the facilitators noticed that there was a tendency for conversations to be dominated by the engineers and while the technical staff was left on the sidelines. Technical staff participation was increased in two ways: First, during discussion periods, the team was broken up into small groups. After the small group discussion, a representative of each group was asked to report to the team. The technical staff felt more at ease about contributing in the small groups. After they had a chance to voice their opinions in the small groups they were more likely to make further contributions to the entire team during the small group reports and at other times. Second, the technical staff was asked to report on specific problems that they or their colleagues had experience recently, especially with regard to resolving technical issues.

Desire for Self-Determination. Some, but not all, Iowa DOT employees are pleased the opportunity to have a hand in shaping their future. Past management styles often did not allow this to the extent that it is encourages as part of the quality improvement program. Therefore, there is a pent-up desire among some employees to make their contribution. It is desirable to identify these employees tend and ask them to volunteer for PITs. They contribute with considerable energy and enthusiasm.

PDCA Cycle for Process Improvement

A Plan-Do-Check-Act (PCDA) (Figure 1) cycle shows how process improvement using PITs works within the Offices of Construction. Here, planning activities include selecting topics for study and developing the initial charters for the PIT teams. The topics are selected by reviewing the output of the performance measuring system to identify areas where improvement is needed the most. The performance measuring system for the Iowa DOT Offices of Construction includes attitudinal surveys Office of Construction Employees, Other DOT employees outside the Office of Construction, Contractors, Law enforcement Officials and Truck Drivers (*1*,*2*).

The steering team should also consider the following:

- Other studies or activity related to possible topics. For example, an area that has high priority for improvement may be currently under study or in the process of change. In most cases it is best not to duplicate the efforts of the study or to wait until the system has reached a steady state after changes have been made.
- Written comments on attitudinal surveys that may provide addition insight.
- Conversations of Steering Team Members with colleagues and other customer groups will also help to clarify how a particular measure may be improved, especially when the measure was part of an attitudinal survey.
- Recommendations of previous PITs

The charter must be carefully written so the PIT fully understands its charge. An initial meeting between the PIT and the steering team is also helpful. Doing occurs when the PIT executes its charter. While the PIT is working, it is wise to have a member of the PIT report progress to the Steering Team and ask for clarifications, if necessary. Checking is reviewing the results PIT study and asking for clarifications. Acting is implementing the PIT recommendations. In some cases the recommendations may include further study by other PITs. In this case another PIT is launched.

Summary

Long term quality leadership requires top management commitment, a pool of participant who have the proper skill for teamwork, and a clear understanding of the organization's mission, key functions, and customers. A steering team that includes a vertical slice of the organization leads the effort. A performance measuring system provides direction to improvement efforts. It focuses attention on areas that are most in need of improvement and follows a PDCA cycle on a regular basis to update the Steering Team on areas where improvement is most needed. Process improvement follows another PCDA cycle for devising plans for improvement and implementing recommendations. Strong foundations and the two PCDA cycles ensure the continued quality leadership.

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Using Concepts of Driver Expectancy, Positive Guidance and Consistency for Improved Operation and Safety

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Kansas State University has had a Traffic Assistance Services for Kansas (TASK) for several years to develop and present training materials to local government personnel with the objective of improving safety. The authors have promoted the concepts of driver expectancy and positive guidance in all of their materials. For example, these concepts are the foundations of the Kansas Low Volume Roads (LVR) Handbook. As a companion to the handbook, Commentary Driving was developed as a tool for evaluating LVR situations where roadway conditions and/or signing "surprised the driver," i.e. violated the drivers' expectancy. These are potentially high-risk locations. Commentary Driving is a technique where a driver drives a route while he/she makes a running commentary of his expectations and particularly his/her expectancy of the road ahead and his/her driving requirements to drive safely. Kansas State University (KSU) developed a number of manuals and course materials over the years to teach the technique. The training progressed from subjects driving vans over specified routes to having subjects view videos. The final step was to develop a commercial production of a self taught interactive video/workbook. With this media it is possible to teach and promote the technique worldwide. The paper discusses the importance of the technique to improved safety on LVR and its potential or a low-cost, valuable tool around which a local unit of government could build a local safety management system (SMS). Key words: driver expectancy, commentary driving, positive guidance.

INTRODUCTION

Roads should be inspected to note if existing signing is adequate and if signs are the proper size, shape and color. State and local governments must also ensure that the appropriate sign or marker is used properly and consistently to give drivers clear information when and where needed. Commentary driving is a procedure developed to assist in evaluating roads and achieving proper, consistent signing at a reasonable cost. The procedure can benefit local governments in defense against tort liability claims and in meeting increasingly tighter budgets because, when used, commentary driving also cuts down on over-signing. It is based on the human factors principles discussed below.

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BASIC PRINCIPLES OF SAFE OPERATING PRACTICE

Three basic principles of human factors that relate to safe operating practices on highways are "driver expectancy," "positive guidance" and "consistency."

Driver Expectancy

Drivers, and people in general, expect things to operate in certain ways. When a driver's expectancy is incorrect, either the driver takes longer to respond properly or he/she may respond poorly or wrongly. If, for example, a driver relies on a curve sign that shows a curve to the right but the road actually curves left, one can imagine the difficulty the driver may have in safely negotiating the curve—especially if he/she is a stranger to the area at night. This has been observed rather frequently in the "Winding Road Sign" in which the bottom or beginning curve points in the wrong direction.

What the driver expects on a road is greatly influenced by the "roadway environment," i.e., what was experienced on the previous section of the road. Studies have shown that what a driver experiences on a road section—presence or absence of traffic control devices, road surface type, condition and width, narrow bridges or culverts, is what the driver expects to continue for the next 1 to 2 kilometers.

Driver expectancy is also affected by those things drivers have learned through past experiences. Driver expectancies are affected by the type of road, such as an interstate highway, state highway, county or township road. The prudent driver expects to drive each of these with different levels of caution.

Positive Guidance

Positive guidance is the concept that a driver can be given sufficient information where he/she needs it and in a form he/she can best use it to safely avoid a hazard. Positive guidance can be given to the driver through a combination of signs, object markers, safe advisory speed signs, and probably most important of all, the view of the road ahead. A prudent driver with adequate view of the road ahead will adjust his/her driving tactics accordingly. This is particularly true if the driver is provided with consistent alignment, adequate sight distance and adequate and consistent signing. If drivers could see the curves far enough ahead to judge their sharpness and adjust to a safe speed, or see approaching cars on cross roads because the intersections were clear of sight obstructions, or if there were no intersections hidden by the crest of a hill, and if all narrow bridges and culverts were visible to drivers from both directions, the road "communicates well." Under these conditions there would be little need for anything more than an occasional stop or yield sign to assign the right of way at the intersection of LVR roads with higher volume roads. The condition just described might be called "roadway positive guidance." Using the edge of roadway to guide traffic provides an easy and effective way of providing positive guidance at narrow bridges and culverts or other roadside obstacles.

Consistency

Consistency relates to the "sameness" of the nature of the road from one section to another. Inconsistencies are sudden changes in the nature of the road, e.g., a sharp curve after a long, straight section. Inconsistencies violate a driver's expectancy; thus either the road should be made consistent, which may be impractical, or something should be done to change the driver's expectancy. For example, in the case of a hidden curve in a nearly straight roadway, the use of a curve warning sign with an advisory speed plate will correctly change the driver's expectancy.

Some inconsistencies are obvious; others are more subtle, but no less dangerous. A very useful tool to find and correct such inconsistencies, information deficient locations and locations where drivers' expectancies are violated is Commentary Driving.

The driver brings a body of knowledge, experience, and skills to the driving task. This *a priori* information is supplemented by the information acquired in preparation for a specific trip. Primacy, expectancy, positive guidance, consistency and *a priori* knowledge affect and are affected by the manner in which the driving task is performed, and how the driver interacts with the roadway environment in which he/she operates. The Commentary Driving procedure can be used to evaluate this interaction.

HISTORY OF THE PROCEDURE

The Commentary Driving procedure was first developed in 1985 and was known as the Simplified Location of Information Deficiencies (SLIDE) procedure *(1)*. In 1988 a joint Kansas University and Kansas State University (KSU) Traffic Assistance Services for Kansas (TASK) project team modified the SLIDE procedure for use on Low-Volume Roads (LVR) and published a manual on commentary driving *(2)*.

OVERVIEW OF THE PROCEDURE

Commentary driving is a procedure in which the roadway is driven and the driver comments on areas that present confusing situations that could be potential hazards. It is most effective if the driver is not familiar with the roads being driven. During the first kilometer or two, the driver/evaluator should form his or her own "expectancies" for the roadway. For example, if the road is wide, straight and smooth for a kilometer or more, we expect it to generally continue that way—*that is our expectation.* If, as we go over the crest of a hill, the road curves sharply, without warning, we are surprised, and thus, our expectancy has been "violated." This is the typical reaction of an average driver. As the driver proceeds down a road, each area where his/her expectancy is violated represents a problem area, i.e., a potential hazard that increases crash risk.

Comments are usually recorded on an audio cassette or videotape recorder to record the location and type of potential problem for more detailed study at a later time. The location is easily noted by an odometer reading. It is stressed that the commentary driving procedure is to "flag" potentially high-risk locations for further study. At these locations, a detailed study should be conducted at a later time to determine if changes are really needed. IDE checklists are described and discussed in more detail below.

USING THE PROCEDURE

A vehicle and one or two persons are needed to drive and follow the procedure. One person is the driver/commentator and the other is the recorder/navigator. This allows the driver/evaluator to focus attention on roadway deficiencies and not the procedure itself.

The driver is the key to commentary driving. The driver will locate problem locations by observations of the environment, the roadway ahead, signs and markings. It works best with a driver who is unfamiliar with the road and is forced to rely more on information given by signs or the roadway itself.

Procedure

The actual procedure of commentary driving involves three basic steps:

Step 1: Select an Appropriate Route

As stated, the commentary driving technique works well on low volume roads, i.e. roads with traffic volume less than 400 vehicles per day. A common length of roadway section for the commentary driving procedure is 5 to 24 kilometers. The length must be long enough to allow the evaluator to form initial "expectancies" of the roadway, but not too long so the evaluator becomes tired and less observant.

Step 2: Drive the Route and Record Verbal Comments

First, initial expectancies of roadway conditions are stated. Second, verbal comments are made while driving the roadway to indicate expectancy violations. When driver expectancies are violated on a roadway, an information deficiency and potentially dangerous situation may exist. While following the commentary driving procedure, expectancies should be stated initially and periodically throughout the procedure.

Roadway conditions which form driver expectancies include roadway alignment, width, shoulders, surface texture and/or signs and markings. Factors that affect information needs of drivers include:

- Consistency—the "sameness" of the road, e.g., straight or winding, etc.
- Positive Guidance—sufficient information to avoid hazardous situations, e.g., obstruction markers, arrows or tapering

- Uncertainty—confusing or insufficient information, e.g., the road "disappears" over a hill with no indication if it continues straight or curves
- Decision Sight Distance—distance required to see and react to a situation in time to avoid a problem.

The driver's comments should concentrate on:

- missing information
- incomplete information
- inappropriate message
- misleading/confusing information
- inappropriate location
- inconsistent information, and/or
- signs obstructed by weeds, brush, etc.

Specific commentary should contain information about the road type and texture, travel direction, curve sharpness, bridge width, right of way at intersections, and other roadway conditions. An audio cassette or video-tape recorder is suggested as a recording device. A hypothetical example of suggested commentary might be as follows when approaching a crest:

"Crest curve ahead, view of road limited . . . tree line indicates that road goes straight ahead . . . not concerned about on-coming traffic . . . wide enough pavement . . . can maintain cruising speed . . ."

Once the driver gets to the crest and sees (or reacts) to what is there, just over the crest, there may be two possible comments depending on conditions relative to his expectations when approaching the crest. Assuming two different situations are possible, the corresponding comments could be:

Situation 1: "the road goes straight as expected" (continue with comments on next section), or

Situation 2: "Hey! Tree line went straight but road turned left sharply . . . "Expectations" violated . . . needed to reduce speed . . . should have had curve warning sign at least . . . possibly speed advisory . . . mark site for study." (Sites are usually "marked" by recording odometer numbers.)

When the driver/evaluator discovers a situation where an information deficiency exists, an appropriate comment should be made. Either the driver or passenger can then record the location by the odometer reading (or other means) and a brief description of the situation to note the location and deficiency for a more detailed study at a later time. Odometer readings can be used later to tie the commentary to specific locations and should be recorded frequently.

Step 3: Detailed Study of Problem Sites

It must be emphasized that commentary driving is not intended to be a complete evaluation technique. *Properly used, it is a technique to flag potential problem sites for later evaluation.* More detailed valuations should be conducted at locations where violations of expectancy or any problems were noted during step 2. It could be that an evaluation shows that there is no problem. This should be documented and filed. The driver/evaluator may choose to conduct the in-depth study shortly after doing the commentary driving, or at a later date, according to priorities.

To make the detailed analysis easier, particularly for local organizations with little or no technical expertise in road safety problems, KSU developed checklists for nine different common deficiency situations, with a tenth for all other or general cases *(3)*.

For each of the nine situations there are specific questions which are structured to lead the investigator through a systematic evaluation of the site. The check sheets may be modified to meet specific location and local jurisdictional needs. Each checklist provides a checklist of suggested treatments appropriate to the situation being investigated. This may also be modified to local standards and/or guidelines. An agency with established policies on road safety should use its own guidelines for the detailed study of potential problem locations.

Suggested Program

It is suggested that all roads in a jurisdiction be driven on a regular basis so that every road is driven on a one or two year rotational basis. Once the initial commentary driving procedure is completed and problems are corrected, the procedure can be done quickly, as only a few problem areas will be found. When major changes have been made to a road, it should always be redriven, preferably by a driver not familiar with the changes that were made.

TEACHING THE TECHNIQUE

The author and predecessors have held short courses and workshops based on the LVR Handbook to teach the techniques *(2,3,4)*. In the early workshops, participants drove university vehicles over low-volume routes in surrounding counties. All routes had been previously selected and evaluated by course instructors and were clearly marked on county maps.

An idea was born to put the entire workshop into a self-taught video and interactive instruction manual. This project was started in the fall of 1991 and completed in the summer of 1993. Details of the video and instruction manual are contained in the video and accompanying manual, "Instruction Manual for Commentary Driving: A Self-Taught Interactive Video/Workbook" *(5)*. To present full details here would be too lengthy. In addition to teaching commentary driving, the video also teaches good signing principles.

CONCLUSIONS AND RECOMMENDATIONS FOR USING COMMENTARY DRIVING

Conclusions

Commentary driving is a very useful technique for highway personnel to use in the everyday safety evaluation of roads and streets in their jurisdiction or of specific projects on their roads and streets. The author believes it is the most cost-effective technique available to evaluate the safety of low-volume roads. The commentary driving procedure pinpoints high risk locations and situations—before crashes occur. In this regard, and where accident data is scarce, the commentary driving technique has many advantages over high accident location techniques. The author believes that it could be adapted to other classes of roads and streets, such as construction work zone sites or sites with complex traffic patterns.

The commentary driving procedure should be included as a part of any local road safety audit or safety management system.

The commentary driving technique is more effective if the driver/commentor is someone not too familiar with the route or section, e.g., engineers from adjacent jurisdictions could drive each other's roads.

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Digital Camera Traffic Accident Investigation System

PATRICK T. MCCOY, SHARAD C. SETH, STEPHEN E. REICHENBACH, AND ASHOK SAMAL

INTRODUCTION

The traditional method used by law enforcement agencies to measure the scenes of traffic accidents is the coordinate method, which involves establishing a base line and measuring perpendicular-offset distances to points of interest. The measurements are usually taken with a 100-foot tape, or a distance-measuring wheel. In many cases, use of the coordinate method requires that traffic lanes be closed for an hour or more in order to allow the investigating officers to make the measurements needed to adequately document the accident scene. The lane closures often increase traffic delay and the risk of secondary collisions. Diagrams of the accident scenes are prepared manually from the measurements.

In recent years, some law enforcement agencies have started using total station surveying equipment to document traffic accident scenes. This system features an electronic theodolite equipped with an internal electronic distance-measuring device and a built-in microprocessor, which make it possible to automatically measure and record distances and angles to a reflector placed at the points of interest at the accident scene. The survey data are recorded in an electronic file, which is processed in the office to generate an accurate, scale diagram of the accident scene. Experience with the total station surveying system indicates that about twice as many measurements can be taken in half the time required with the conventional coordinate method. Also, it reduces the number and duration of lane closures required to measure the accident scene, which in turn reduces the delay to traffic, the potential of secondary accidents, and the risk to the investigating officers.

Unfortunately, the equipment and training required by the total station surveying system are beyond the means of many local law enforcement agencies. However, advances in digital camera technology have made it possible to develop an affordable traffic accident investigation system, that provides the time-savings benefits of the total station surveying system and yet requires minimal training and expense.

In August 1996, the University of Nebraska-Lincoln (UNL), in cooperation with the Nebraska Department of Roads (NDOR) and the Omaha Police Department (OPD), initiated a project to evaluate the application of digital camera technology for traffic accident investigation. The effort involved developing a digital camera traffic accident investigation system (DTAIS) and comparing it to the coordinate and total-station-surveying methods of accident investigation. The project was funded by the Federal Highway Administration (FHWA) Priority Technology Program with matching funds provided by the NDOR, OPD, and UNL.

This paper describes DTAIS and its evaluation. The results of field tests comparing it with the conventional coordinate and total-station-surveying methods of accident investigation are presented. The potential for its implementation by law enforcement agencies is discussed.

BASIC PRINCIPLE

A camera system projects the three-dimensional view of the real world onto its image plane. Every point in the field of view is mapped to a unique point on the image plane. The basic problem is to deduce the mapping accurately enough by locating the projections onto the image plane of a small number of known points in a calibration step. Once determined, the mapping enables the real-world locations of other points in the image to be determined.

The general solution to this perspective-projection problem is well-documented in the photogrammetric literature. The basic approach involves bringing the real-world reference frame into correspondence with the camera reference frame by a sequence of rotations along the three axes of the real-world reference frame. Having thus found a representation for a three-dimensional point in the camera reference, and assuming a pinhole camera, the projection of the point in the camera image plane in terms of the camera lens focal length can easily be determined.

FUNCTIONAL REQUIREMENTS

A meeting was held with a panel of experts to define the functional requirements of traffic accident investigation, which provided the basis for the design and evaluation of the system. The panel included representatives of law enforcement, accident records, vehicle insurance, accident reconstruction, and traffic engineering.

The functional requirements addressed such factors as: (1) degrees of measurement accuracy and precision; (2) types and numbers of measurements; (3) accident report requirements; (4) time

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FIGURE 1 Digital camera.

FIGURE 2 Rod.

constraints; and (5) manpower and equipment limitations. The following is a summary of the functional requirements used to design and evaluate the system:

1. Types of measurements: The system must be able to measure three-dimensional surface coordinates.
2. Range of measurements: The system must be able to operate within the field of view up to a distance of 150 feet from the camera. Larger accident scenes can be measured from multiple camera setups. The system must also be able to measure points located off the roadway as well as on the roadway.
3. Accuracy of measurements: The system must have at least the same accuracy and precision as the manual system currently used by OPD, which is about ± 1 foot.
4. Number of measurements: The system must be able to store the data for up to 50 measurements.
5. Records: The system must generate an accident report and diagram in the format used by OPD.
6. Cost: The system cost should not exceed approximately $1,000 for field equipment and $5,000 for office equipment.
7. Operating conditions: The system must function day and night. However, it should first be developed to work in daylight. Nighttime conditions should be considered after the system has been demonstrated to work in daylight.
8. Ease of use: The operation of the system should require only one person.
9. Training requirements: No more than 2 hours of training should be required for the field procedures, but the office procedures may require up to 40 hours.
10. Speed: The system should provide a 50-percent reduction in the time spent measuring distances at the scene.
11. Portability: The system must be small enough and rugged enough to be carried in the trunk of an OPD patrol car.

SYSTEM

The DTAIS system consists of the following components:

1. Kodak DC-120 digital camera with 1280 x 960 resolution, a 16-degree field of view, automatic focus, and automatic shut off
2. Tripod equipped with a leveling device on which the camera is mounted
3. Extendible, 6-inch x 7-foot, red-and-white stripped rod equipped with a flip-chart with letters and numbers for identifying measurement points.

The camera is shown mounted on the tripod in Figure 1. The rod is shown in Figure 2.

FIELD PROCEDURE

The field procedure involves two basic steps: (1) camera setup and (2) measurement. The camera is positioned at a location and orientation so that all points of interest (i.e., evidence points and roadway features) are within the field of view. Two reference points are selected within the field of view to provide a frame of reference for the measurements. The reference points should be well-defined landmarks at the accident scene, such as manholes, fire hydrants, or utility poles. The two points define a baseline to which the locations of the points of interest are referenced. The camera is then used to obtain images of the rod placed vertically at all points of interest including the two reference points.

In some cases, all of the points of interest cannot be imaged from one camera view, because the accident scene is too large or the view of some points is obstructed. Therefore, multiple camera views are required. In order to relate all of the measurements to the frame of reference established from the first view, it is necessary to connect a subsequent view to the previous view by imaging at least two points in the subsequent view that were imaged in the previous view.

ACCIDENT REPORT PREPARATION

The accident report is prepared with the DTAIS software. The software is written in C++ using Microsoft Visual C++ and designed to run on Windows 95. It requires a Pentium or better processor with 16 Mb of memory and enough disk space for the image files.

The files of the images of the points of interest taken at the accident scene are input to the program. Thumbnails of the images are displayed on the working screen as shown in Figure 3. The images are selected one by one from the thumbnails for processing. When an image is being processed, it is displayed in the main portion of the screen below the thumbnails. The user clicks on the center of the rod with the mouse and the 5-foot reference interval (i.e., from the top of the upper red strip to the bottom of the lower red strip) is

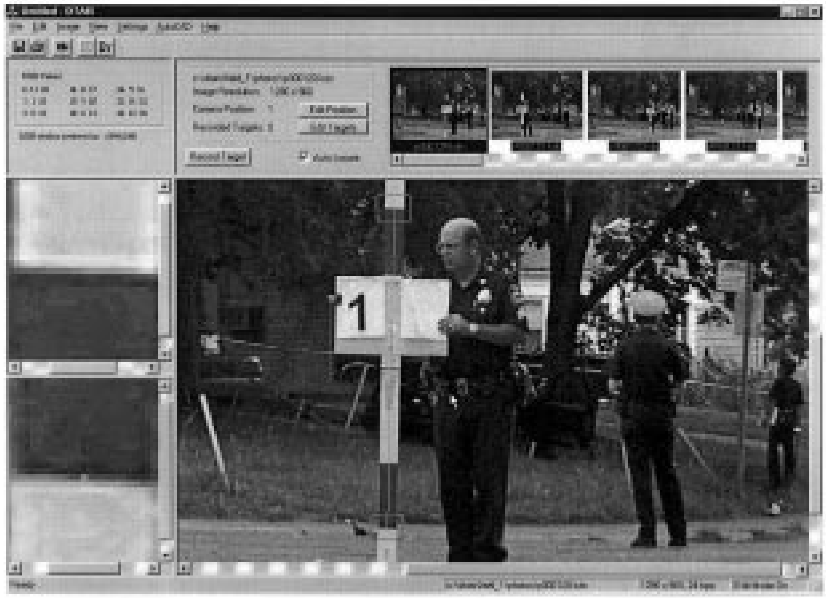

FIGURE 3 Working screen.

FIGURE 4 OPD officers marking evidence points at mock accident scene.

automatically located and its image coordinates determined. The Record Target button activates a dialog box which enables the user to enter a narrative description of the point and store the information along with the point's image coordinates. After all of the images have been processed, the user clicks on the ∑Y icon on the tool bar, which causes the program to compute the real-world coordinates of the points and generate a table containing the descriptions and real-world coordinates of the points. In addition, an AutoCAD file is generated, which contains the points plotted to scale. The program output includes the tabulated description of the points and the AutoCAD plot.

EVALUATION

The evaluation of the DTAIS system was conducted in two stages. In the first stage, the system was used to measure three mock accident scenes created at the UNL Midwest Roadside Safety Facility. These scenes were the results of full-scale crash tests of bridge railings and guardrails conducted at the facility. After each crash test, OPD officers identified and marked the evidence points with spray paint in accordance with their normal procedures as shown in Figure 4. Then they measured the points using both the coordinate method and DTAIS. Two members of the Kansas Highway Patrol Critical Highway Accident Response Team participated in the measurement of the third mock accident scene. They measured the mock accident scene using their total station surveying equipment as shown in Figure 5.

DTAIS was compared to the coordinate and total station surveying methods in terms of measurement accuracy, time required, and degree of difficulty. It was determined that DTAIS met the functional requirements specified by the expert panel, except that its operation required two officers instead of one. However, the total station surveying method requires two officers and the OPD typically uses two officers to measure an accident scene with the coordinate method. Therefore, it was concluded that the system was ready for field testing by the OPD.

In the second stage of the evaluation, the OPD used DTAIS to investigate accidents occurring in Omaha. The officers used both the coordinate method and DTAIS to measure each accident scene. In addition, they prepared their routine accident report and an accident report using DTAIS software. Members of the study team accompanied the officers to the accident scenes to document the investigation procedures and provide technical support. The documentation of each accident scene investigation included a comparison of the coordinate method and DTAIS with respect to measurement agreement, field measurement time, report preparation time,

FIGURE 5 CHART officers measuring mock accident scene with total station.

level of effort, and degree of difficulty. Problems encountered and feedback from the officers were noted. A total of four accidents were investigated.

The results of the field investigation indicated that the DTAIS met the functional requirements specified by the expert panel. In addition, the field investigation experience resulted in a number of refinements to the system that will significantly improve its implementation. These refinements included: (1) revisions to the Operating Procedures Manual to provide camera location guidelines and clarify the explanation of base lines and reference points; (2) revised calibration procedure, which eliminates the calibration step for each camera position and replaces it with calibration at the police station; (3) modifications to the software designed to make it easier to use and more compatible with current OPD practices and terminology; and (4) development of an extendible rod which will facilitate the field procedures and improve accuracy.

The field investigation also identified some barriers to implementation of the system which must be addressed. Implementation of the system by OPD requires that these barriers be eliminated. The barriers and the actions currently underway to overcome them are summarized in Table 1.

CONCLUSION

The evaluation of the DTAIS has demonstrated the feasibility of using digital photography for the investigation of traffic accidents. This technology can provide the necessary degree of accuracy and substantial savings in the time required to measure accident scenes and generate accident reports. The UNL, in cooperation with the NDOR, OPD, and FHWA, are refining the system to address the implementation barriers that were identified. It is anticipated that this effort will be completed by the end of 1998.

ACKNOWLEDGMENT

Members of the Project Advisory Committee are Officer Larry Bakker, OPD; Milo Cress FHWA; Frand Doland, FHWA; Leona Kolbet, NDOR; Donald McDonald, State Farm Insurance; Sergeant

Table 1 Barriers to the Implementation of DTAIS

Implementation Barrier	Action
Software too complicated.	Improve software user interface.
Too many camera setups.	Increase camera’s field of view by zooming out and use camera with higher resolution. Also, investigate the feasibility of adding a GPS unit.
Nighttime operation.	Improve software to include automatic interval finder for nighttime images.
Operation during inclement weather.	Investigate the feasibility of weatherproofing the camera.
3D measurements.	Improve software to provide 3D analysis.
Cumbersome method of tagging ID and description to points.	Investigate the feasibility of keying point ID and description directly into camera.
Incomplete accident diagram.	Improve system to enable generation of complete accident diagram including evidence points and roadway features.
Limited awareness of technology.	Make presentations describing system to law enforcement agencies and association meetings and publish articles describing system in law enforcement journals.

Richard McWilliam, OPD; Officer Sean Quinlan, OPD; Jim Pearson, NDOR; David Peterson, NDOR; Lieutenant Robert Rockwell, OPD; Trooper David Sankey, Nebraska State Patrol; Sergeant Greg Vandenberg, Nebraska State Patrol; and Fred Zwonechek, Nebraska Department of Motor Vehicles. The cooperation of the Kansas Highway Patrol, Colonel Lonnie R. McCollum and Captain Bob Giffin is acknowledged for allowing Master Troopers Steve McKenzie and Jim Todd to participate in the evaluation.

The Involvement of Regional Planning Organizations in the Statewide Transportation Planning Process

KENT B. VAN LANDUYT

The element of local input is important in statewide transportation planning. The Missouri Department of Transportation (MoDOT) developed a process to use the regional planning organizations (RPO) to determine which transportation needs and values are most important to the region and the state. The department chose the regional planning organizations because they geographically covered the state, were knowledgeable about planning activities, had experience in working with federal programs, and had the desire to be involved in the transportation planning process. The initial activities began in 1993. Each RPO was required to appoint a transportation advisory committee that was composed of local officials and citizens, gather general transportation comments, and work actively with MoDOT. The success of the activities in the initial phase led to expanded RPO planning activities in 1995. The expanded process directed each RPO to develop an annual transportation work program that identified specific transportation planning activities they would conduct for the department. The additional items included an evaluation process of transportation needs, a public involvement process, development of regional data, and professional staff development. The program has been successful in providing transportation information that can be used in the MoDOT decision-making process. The program will be continued in the upcoming years to assist the department with updates to the long-range transportation plan and the selection of projects to meet those transportation needs. Key words: transportation, planning, ISTEA, regions, local involvement.

BACKGROUND

In the 1980s, MoDOT became more aggressive in seeking ideas on transportation issues from outside the department. The department joined the ranks of many organizations that chose to become "customer" oriented. The passage of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) by the U. S. Congress, and the subsequent regulations (*1*), also increased awareness of the need for outside involvement in transportation issues.

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The need to expand its outside involvement in the transportation planning process was evident. The department used internal staff committees and outside contacts as information sources in developing the involvement process.

The department found an existing structure in Missouri that would be advantageous in the development of an expanded planning process—the regional planning organizations. This idea led to the formation of a direct working relationship between all the regional planning organizations and the Missouri Department of Transportation.

At the onset, a meeting was held with several RPO executive directors to establish the direction of a partnership with MoDOT. The interest expressed by the RPOs led to the development of a contract to initiate the first phase of the working relationship.

Phase One

The first phase of using RPOs in the planning process began in late 1994 with a contract between the department and each RPO. The contract required the RPO to provide basic planning information to the department. It authorized reimbursement of expenses, up to $5000 per year, for their approved transportation planning activities. The region was required to provide a 20% match of local funds on all expenditures.

The basic requirements of the RPOs in phase one were:

- Establish a Transportation Advisory Committee (TAC) which would meet on a regular basis
- Join the department in public transportation meetings
- Assist the department in the public involvement process
- Keep MoDOT informed of transportation issues in their region
- Consider all modes of transportation in their activities.

Transportation Advisory Committee (TAC)

The RPOs had specific requirements in the development of the TAC. Its structure had to include representatives from each county, with the option of additional representatives from any municipalities; some general citizen representation; and ex officio representation from each MoDOT district within the regional planning area. The department felt this cross section of members would represent the interest in the region and the inclusion of MoDOT would provide a direct connection with the department, without having to go through a number of people or bureaucratic levels.

The TACs were required to meet on a regular basis. The majority of them chose to meet quarterly. Some had their first few meetings every two months and then they went to a quarterly schedule.

The TACs were expected to give written and verbal input to the department on transportation needs and their ideas of what was important in their area. They were also expected to assist the department with any public meetings that would involve both agencies.

The TACs were recognized as a committee appointed by the RPO Board of Directors. If they had formal positions to take on an issue they would make recommendations to their Board. The Board would then decide if they wanted to take a formal position on a transportation issue.

The contract language for this process was kept very simple. It stated "The United States Congress has authorized in the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA, P.L. 102-240, 23 USC Sec. 307 and others) funds to be used for transportation planning activities. The purpose of this agreement is to grant the use of such transportation planning funds to the Agency." The remainder of the contract was all "boiler plate" items. This general contract statement allowed the department extensive flexibility in the use of federal transportation planning funds as reimbursement to the RPO for their planning activities.

The first phase was general and focused on developing a good relationship between the regions and department. Since there was a maximum of only $5000 available per year we did not want to tie the process to a bureaucratic process that would be more costly to administer than the amount of funds available.

Parallel MoDOT Activities

The Missouri Highway and Transportation Commission appointed a subcommittee to review the department's activities as they related to the counties and regional planning organizations in 1993. That subcommittee recommended the department work with local governments to address transportation issues.

A MoDOT news release quoted Commissioner Williamson on the commission's interest in working with the RPOs:

> Our planning process is open to anyone interested in transportation, and regional planning commissions help coordinate the information needed for transportation planning. Regional planning commissions are made up of local elected officials who have a significant interest in all types of transportation—aviation, rails, highways, waterways, transit. The planning commissions help avoid duplication and encourage coordination among the various types of transportation systems (*2*).

The department also established a breakthrough team in 1994 to look at the department's project development process. The department adopted the recommendation of that team to include the process of using the RPOs to evaluate transportation needs and assist the department in the prioritizing of projects.

These additional ideas on the application of RPOs to the transportation planning process led to the development of the second phase of activities. In 1995 a team of department and RPO staff members developed the guidelines for the second phase.

Phase Two

The phase two guidelines increased the involvement of the RPOs. The amount of funds available for reimbursement to each RPO was increased to $21,400 for calendar year 1997 and $26,400 for calendar year 1998, an amount developed by the department and RPO team. All expended funds were required to be matched by 20 percent local funds. The RPOs were given additional requirements to meet in order to be eligible for the these funds. The requirements were:

1. Each RPO was required to develop a work program in cooperation with the MoDOT districts in their region. The work program was required to identify work tasks, explain what they accomplished in the last year, describe what they wanted to accomplish this year and list future expectations on each task. The work program was also required to show the amount of funds they expected to spend on each task.
2. The expectations of the Transportation Advisory Committee were expanded as a result of the recommendations made by the department breakthrough team. This included the development of evaluation criteria.
3. The RPO was required to submit a progress report with each quarterly billing and an annual report of activities on each work program task. The report was required to be concise but in enough detail to identify activities conducted during the year. Each RPO was required to document their achievements within the context of the work program. An example of this would be statements that support the accomplishments in the past year, the activities planned for the current year and activities identified for the longer term (next few years).
4. A work program might include a section entitled "general transportation planning" that would include additional tasks, such as mapping requests, policy ideas and statewide transportation perspectives.
5. Each RPO was expected to encourage staff development in transportation planning.
6. The development of the work program would be the primary responsibility of the RPO, with the assistance of each of the MoDOT districts within the RPO region.
7. Other areas of RPO activities would include working with the department on public involvement activities, educating the public on transportation issues, data gathering, and assisting local communities in the application process for transportation grants.

All eighteen Missouri RPOs started in phase one; each chose to expanded their transportation planning processes to include the phase two guidelines. The application of the process varied by RPO, due to their regional characteristics, but the same general outcome was sought. The department and RPOs felt it was important for each of them to have the flexibility to develop their own values and needs process. The regions have diverse factors that drive their economy, such as tourism, agriculture, urban fringe, government installations, or a combination of these. It is these regional differences which needed to be recognized; therefore, the idea was to let each region develop its process in terms of the major characteristics of its area.

As this process continues to develop with each RPO, it is the intent of the department to involve the RPOs in the long range planning, public involvement, data gathering and local transportation awareness campaigns. The guidelines for these activities will be focused on meeting the federal transportation planning requirements and obtaining meaningful information on all transportation modes. The level of involvement will vary by region due the activities of the area, the district and RPO staff limitations, and available funds.

FINDINGS

The TACs held meetings with citizens in each county to develop the weighted values. The attendance at the local meeting ranged from a small group of approximately 10 in some counties to as many as 60 or more in other counties.

The TAC was directed to develop a method to prioritize transportation projects based on criteria they would develop with the help of planning professionals. It was acceptable for these criteria to be unique to their region because of its individual characteristics. The criteria developed by each TAC usually included safety, economic development, connectivity with other systems, preservation and tourism as the major evaluation elements. The TACs gave these elements a value, assigned a value to each identified transportation need for each of the factors, and developed a composite score for the project. This composite score for the project was used by the department in ranking the region's priority.

The RPO staffs found that the group meetings were more successful if the staff developed most of the material on their own. The TAC involvement was not as successful when they were asked to develop the entire process on their own. The general idea was that the TAC members did not mind commenting and giving viewpoint, but did not want to spend the time developing the process.

Each RPO developed this process independently of others because of the unique characteristics of their region. This resulted in a few different values but safety and economic development were consistently the higher-rated values throughout the state.

The concept of preservation evolved with an unusual understanding. The preservation of the existing transportation system did not get a high rating from the participants. The department was surprised that the regions did not rank preservation as a high value and sought the reason for this phenomenon. When the participants were questioned about the reasons behind their ratings their standard response was the public expected the department to keep the existing roads in usable condition. Their response was, “You don’t go buy a new car if you can’t afford to keep up the one you own.” The department learned that the public expected the department to maintain the existing system. The building of additions and expansions to the system were identified needs beyond a well-preserved system.

The success of the program varied due to interest of TAC members as well as the commitment made by district and RPO staff. The process must demonstrate success for the TAC to be effective. They have to believe the process is working and the department is using their input. MoDOT was able to incorporate their recommendations almost immediately on some items and schedule other recommendations into the department’s plans. Some items such as signing, turning lanes and problem locations were addressed almost immediately. The major projects were included as recommended additions to the department program.

Missouri has found this program to be beneficial to the department. It strengthens the department’s awareness in the local community through local involvement, allows the department to understand the transportation needs identified by the local people, builds credibility through a formal structure, and guarantees a consistent flow of information throughout the state.

The Missouri program has also gained interest outside the state. Several Departments of Transportation and Regional Planning Organizations have made inquires to MoDOT on the process. The process was also featured in the January 1998 issue of *Economic Development Digest*. The article focused on the planning activities of the Northwest Missouri Regional Councils of Governments and the Meramec Regional Planning Commission (*3*).

The process is also gaining some national attention in the U.S. Congress. As of spring 1998, several proposals for the reauthorizaton of ISTEA have references to local involvement in the transportation planning process. The interest is there for the states to involve the public and local officials in the transportation planning process; it will take the continued interest at all levels to develop the ideas into a working process in each state and region.

SUMMARY

The Missouri Department of Transportation started working with the regional planning organizations in the late 1980s, developed a formal process in early 1990s and expanded the process in 1996. The process focuses on transportation planning issues. It includes the requirement that each RPO establish a Transportation Advisory Committee that must meet at least quarterly, must develop an annual work program of activities, and must submit expenses to MoDOT for approval and reimbursement with transportation planning funds. The major activities addressed by the process include public involvement, transportation planning, needs identification, and prioritizing of projects.

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Response of State Transportation Planning Programs to the Intermodal Surface Transportation Efficiency Act of 1991

MICHAEL A. LIPSMAN AND CLYDE KENNETH WALTER

The 1991 Intermodal Surface Transportation Efficiency Act refocused the nation's transportation policy and programs to emphasize a national intermodal transportation system. This paper describes the results of a survey which shows that the states have identified the critical multimodal issues but may lack necessary planning resources, such as skills for adapting basic investment analysis methods and geographic information systems to meet multimodal concerns. Key words: transportation planning, ISTEA 91, intermodal, multimodal, states.

RE-FOCUS POLICY AND PROGRAMS

Passage of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 marked the beginning of a new era for transportation policy and programs in the United States. With this legislation the federal government's almost singular focus over three decades on development of the Interstate Highway System was brought to a close. In its place the legislation redirected national transportation policy toward total system integration and intermodalism.

ISTEA Programs

The Act's declaration of policy said the programs were intended to promote a National Intermodal Transportation System consisting of "all forms of transportation in a unified, interconnected manner, including transportation systems of the future" (Section 2). In addition, the legislation promoted the more efficient use of energy, reduced air pollution, and the increased competitiveness of United States businesses in world markets. Other departures from past policy include: (a) an increased emphasis on system preservation; (b) a greater reliance on the private sector to fund infrastructure investment needs; (c) an increased flexibility for state and metropolitan area governments to allocate program funds between highway and transit programs; (d) an increased emphasis on freight system planning; (e) a closer coordination of transportation planning activities between state and metropolitan area governments; and (f) a heightened recognition of the impact of transportation on the natural and man-made environments.

Further evidence of Congress' commitment to move national transportation policy from a modal to a system-wide focus was Title V (Section 5005), which established an Office of Intermodalism in the U.S. Department of Transportation and the creation of a National Commission on Intermodal Transportation. The Commission's 1994 report (*1*) made recommendations in three major categories: (a) making efficient intermodal transportation the goal of federal transportation policy; (b) increasing investment in intermodal transportation; and (c) restructuring government institutions to improve support for intermodal transportation.

State Planning Agencies

ISTEA encouraged state and metropolitan area governments to adopt similar changes by requiring them to implement six management systems (later made optional) covering pavements, bridges, highway safety, traffic congestion, public transit, and intermodal transportation, and to develop new long-range transportation plans emphasizing intermodal transportation and a system-wide perspective (Sections 1025 and 1034). In response, state transportation planning agencies have undertaken a comprehensive review of their long-range transportation plans. A review of nineteen plans confirmed a wide range of progress toward achieving the intermodal element of ISTEA. A survey of state transportation departments was conducted to better understand how states have responded to ISTEA and to provide a description of the gradually changing emphasis of transportation planning at the state level from mode specific to multimodal.

Multimodal Versus Intermodal Transportation Planning

The states' interpretations of ISTEA demonstrated some inconsistency in terminology. Although "intermodal" may have been advantageous in forming the ISTEA acronym, "multimodal" is probably a better term for describing the integration of transportation, planning and business logistics. In this context, it may be useful to consider "multimodal" as including effects among transportation modes due to changes in use of one mode, whereas "intermodal" would apply to a given movement involving more than one mode (*2*).

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TABLE 1 Transportation Management Systems, Completed or In-Progess

	Percent of states (n=46)	Chi-square (df=3)
Pavement of federal aid highways	100	0.52
Bridges	98	0.45
Highway safety	85	0.49
Traffic congestion	83	0.71
Public transportation	64	2.34
Intermodal transportation	63	1.48
Traffic monitoring for highways	91	0.19
Chi-square critical values:	.05 = 7.815; .10 = 6.251	

TABLE 2 Passenger-Related Modes and Combinations

	Percent of states (n=44)	Chi-square (df=3)
Private passenger vehicles	98	0.93
Bicycle/pedestrian	98	1.96
Urban transit (all modes)	91	0.68
Intercity bus	89	1.85
Air passenger	84	1.38
Intercity railroad passenger	84	1.19
Light trucks and vans	80	2.06
Bus-rail	59	1.66
Bus-air	52	2.58
Air-rail	45	0.97
Water passenger	36	1.02

TABLE 3 Freight-Related Modes and Combinations

	Percent of states (n=44)	Chi-square (df=3)
Freight railroads	89	3.28
Freight motor carriers	84	2.03
Truck-rail	82	4.53
Air freight carriers	73	0.34
Truck-air	61	1.30
Rail-water	59	2.27
Truck-water	59	2.27
Water freight carriers	59	0.46
Pipeline	36	7.28*
Other	14	1.96

*Differences significant at the .10 level

SURVEY OF STATES' RESPONSES

Questionnaire

A four-page questionnaire was sent to the planning director in each of the 50 states to assess how their planning programs have been adjusted to reflect the direction set by ISTEA. The purpose of the survey was threefold: (a) to identify states' multimodal perspectives and barriers encountered in developing multimodal plans; (b) to determine resources and skills needed for multimodal planning; and (c) to identify preferences for future ISTEA-type legislation. Forty-six states returned the surveys in complete or partially usable form.

Results

Virtually all (96 percent) of the states responding had produced new or updated transportation plans following ISTEA. When asked about their progress in developing the six transportation management and monitoring systems, the intermodal transportation management system was in last place, with 63 percent of the states having that system completed or in-process. Seventeen states (37 percent) indicated no plans to complete this management system. ISTEA may have increased the attention given to transportation involving more than one mode but, as Table 1 shows, systems for the more traditional topics like managing pavement and monitoring highway traffic drew more than twice the attention afforded intermodal transportation.

The states were next divided into two groups by geographic area (e.g., largest 25 states in square miles versus smallest 25), by population, by population density, and by percent urban population. For each group comparison, the number of positive responses in the first group was compared with the number of positive responses in the second, and chi-square values calculated, to test the null hypothesis: "The progress toward completing a particular Transportation Management System does not vary by group (when states are grouped by area, population, population density or urban concentration)." There was no indication that states' efforts for their Transportation Management Systems varied by size of state, population, population density or urban portion.

Specific Modes or Transportation System Combinations

Another indication of the attention given to intermodal transportation was furnished by responses to a question of which modes or systems were addressed by the states' new or updated plans. Responses for passenger and freight transportation were separately accumulated and are ranked in Tables 2 and 3. As might be expected, passenger travel by automobile received specific mention by nearly all states, but bicycle and pedestrian travel were a close second. The three intermodal combinations—bus-rail, bus-air, and air-rail—were included in 59 percent, 52 percent and 45 percent, respectively, of state plans. Passenger transportation by water was included by only 16 states (36 percent) in their transportation plans. The chi-square test of the null hypothesis, "Passenger modes and combinations included in state transportation plans do not vary by area, population, population density or urban concentration," produced no values exceeding the critical levels.

The freight combination of truck-rail was included in 82 percent of the state plans, just behind the individual modes of rail and truck. The other intermodal freight combinations of truck-air, rail-water, and truck-water were included in over one-half (59 to 61 percent) of the states' plans; see Table 3.

The null hypothesis, "Freight related modes and combinations included in state transportation plans do not vary by area, population, population density or urban concentration," was rejected for

TABLE 4 Infrastructure Needs in Traffic Network Models

	Percent of states (n=44)	Chi-square (df=3)
State highways	39	0.71
County and city roads and streets	18	0.38
Intercity rail passenger facilities	14	0.69
Urban transit facilities	14	1.85
Intercity bus terminals	11	1.67
Intermodal freight facilities	11	3.33
Commercial airports	9	3.20
Intermodal passenger facilities	9	2.79
Rail freight track and yard facilities	9	2.79
Air freight facilities	7	4.89
General aviation airports	7	4.89
Pipeline terminal facilities	5	5.87
Water freight port facilities	5	2.67
Water passenger terminals	2	4.00
Other	2	4.00

TABLE 5 Multimodal Issues in New or Updated Statewide Plans

	Percent of states (n=44)	Chi-square (df=3)
Urban rail-highway conflicts	57	0.77
Rural rail-highway conflicts	55	1.52
Intercity bus and rail terminal joint location	48	1.67
Freight terminal impact on roads	36	3.19
Rail and water freight terminal joint location	36	2.26
Passenger terminal impact on roads	30	3.31
Reduces highway use due to rail freight	30	2.80
Intercity bus and air terminal joint location	27	6.26*
Highway investment on motor carrier terminal location	23	2.40
Passenger terminal impact on parking	20	0.44
Highway investment on warehouse and DC location	20	2.22
Other multimodal issues:	20	6.74*
Access (friendly design, landside, intermodal)		
Diversion from highway		
Focus on truck-rail transfer		
Integrating bicycle/pedestrian with roads and bridges		
Interconnected rail-freight-air-water service on highway operations		
Light rail-airport, commuter heavy rail		
Truck-rail movement of agricultural products		

* Differences significant at the .10 level

pipelines (at the .10 level). The transportation plans of larger states (27 percent versus 9 percent of the smaller states), less dense states (25 percent versus 11 percent of the more dense), and more urbanized states (23 percent versus 14 percent of the less urbanized) were more likely to include pipelines. This result is consistent with the observation that the oil-producing states tend to be large, with dispersed populations, and the oil-refining states tend to be industrialized with major population centers.

Infrastructure Needs Addressed By Traffic Network Models

Intermodal freight facilities and intermodal passenger facilities were included in 11 and 9 percent, respectively, of the states' traffic network models. The infrastructure needs addressed by these models were dominated by state highways (39 percent) and county and city streets and roads (18 percent). As displayed in Table 4, terminals and facilities for other individual modes received scattered mention. If the "intermodal" objective of ISTEA is to be achieved, the traffic network models used by state planners will need to be updated to include trips or hauls involving more than one mode and the infrastructure required to allow the intermodal transfers of freight and passengers. The null hypothesis, "Infrastructure needs in traffic network models do not vary by area, population, population density or urban concentration," was not rejected for any of the categories listed in Table 4.

Multimodal Issues in New or Updated Plan

When presented with a list of eleven specific multimodal issues, the top three identified by state planners were urban rail-highway conflicts (57 percent), rural rail-highway conflicts (55 percent), and intercity bus and rail terminal joint location (48 percent). As Table 5 indicates, highway-related issues appeared frequently—such as freight and passenger terminals' impacts on roads—and were included in comments in the "other" category.

Null hypotheses testing suggested two issues for further inquiry. First, rejecting (at the .10 level) the statement, "The inclusion of intercity bus and air terminal joint location in new or updated statewide plans does not vary by area, population, population density or percent urban population," lends support to interpreting the data as showing that states with larger areas and lower densities were more likely to include this issue. Smaller states were less likely to have airports and lower density states may be more likely to employ intercity bus transportation rather than light rail.

The second null hypothesis to be rejected (also at the .10 level) concerned "other" multimodal issues: "The inclusion of other multimodal issues (see Table 4) does not vary by area, population, population density or urban concentration." Geographically smaller states were more likely to list "other" multimodal issues in response to this question.

Methods Used for Identifying Future Needs for Transportation Infrastructure Investment

Explanations of the lack of integrated planning may be indicated by the methods which states use to identify future needs for transportation infrastructure investment. The rankings of Table 6 show that more than twice as many responding states (67 percent) employ non-network models than network models. In other words, individual projects were analyzed without full consideration of their effects upon the transportation network. Twenty-four percent had a passenger network model, but no freight model, while 9 percent had separate network models for passenger and freight transportation. Fifteen percent claimed single integrated network models. Given the complexity of a state's transportation system, resorting to less-than-complete analyses is not surprising. Forty-eight percent of the states employed benefit-cost analysis, a structured model that attempts to compare total societal benefits to total societal costs,

TABLE 6 Methods for Identifying Transportation Infrastructure Investment Needs

	Percent of states (n=46)	Chi-square (df=3)
Mode and system non-network models	67	0.87
Benefit-cost analysis	48	1.09
Mode specific network models	26	7.27*
Passenger network model, but no freight model	24	1.76
Regional economic impact models	24	2.46
Single integrated network model	15	3.88
Separate passenger and freight network models	9	1.07
Other:	13	0.00
HPMS package, system-wide model is under development		
Person trip model, separate transit model		
Policies		
Public involvement		
Regional prioritized needs		
REMI for economic impact analysis		
Differences significant at the .10 level		

TABLE 7 Training Needs Identified Due to Multimodal Emphasis of ISTEA

	Percent of states (n=46)	Chi-square (df=3)
Geographic information systems	72	4.13
Transport economics	54	0.64
Benefit-cost analysis	50	0.83
Financial analysis	50	0.83
Transport network development and modeling	50	1.05
Inter-city freight demand forecasting	48	0.87
Railroad system planning	37	0.94
Public finance	33	1.07
Public transit system planning	33	2.90
Inter-city passenger demand forecasting	30	5.69
Business logistics	26	0.93
Air transport system planning	24	3.54
Water transport system planning	24	3.38
Intra-city passenger demand forecasting	13	2.22
Other:	15	1.58
Application of business planning practices to non-govt transportation		
Bicycle/pedestrian planning		
Dept. has extensive on-going training program		
Future planning skills and needs study		
Information systems in general, e.g., data warehousing, Oracle		
Not at this time, but see a need for GIS databases		

"whether they be monetary or nonmonetary in nature" (*1*). Twenty-four percent also used regional economic impact models, a sign that wider analysis is taking place. Six individual states (13 percent) mentioned using or developing other network or regional models.

The chi-square test of the null hypotheses, "The use of mode specific network models for identifying future needs for transportation infrastructure investment does not vary by area, population, population density or urban concentration," produced one value exceeding the critical value (at the .10 level). States with more urbanized population and with larger land areas were more mode-specific. Conversely, states with more dispersed population may have broader infrastructure needs.

Training Needs Identified Due to Multimodal Emphasis of ISTEA

Even if more states had integrated models available, the skills necessary for applying them may need upgrading. As Table 7 shows, a majority (67 percent) desire training in Geographic Information Systems. Over 40 percent of the states indicated a need for training in five additional categories: transport economics, benefit-cost analysis, financial analysis, transport network development and modeling, and inter-city freight demand forecasting. Seven more training needs were identified by one-fifth to one-third of the respondents, further evidence of the education support that will enable implementation of multimodal transportation planning. Chi-square tests identified no null hypotheses of the form, "(Specific) training needs due to the multimodal emphasis of ISTEA do not vary by area, population, population density or urban concentration," that were rejected.

Funding Flexibility

Consistent with the shifting emphasis to multimodal transportation was increased flexibility in spending federal funds. Where most prior allocations were restricted to one mode, such as a highway project or transit, the objective of ISTEA was to encourage projects involving more than one mode by reducing the restrictions on spending. When respondents were asked if they felt that ISTEA had accomplished the objective of making federal funding programs more flexible, 57 percent said yes and 43 percent said no.

While a clear majority agreed that the federal funding programs were more flexible under ISTEA, it is notable there were 19 states responding "no." Accompanying the negative responses were the following comments: (a) "In general, funding is less flexible under ISTEA than under previous acts;" (b) "Flexibility between transit and highways, but between them and other modes has not occurred. Intent was good, but too many limitations;" (c) "Cannot easily use funds on intercity passenger and freight (rail, ports);" (d) "Important progress has been made but . . . the problem of less than full funding continues to hinder flexibility;" (e) "ISTEA changed very little unless states endorsed the change themselves."

Funding flexibility to allow, even promote, multimodal programs appears to be one step in the right direction. Future legislation may build upon the partial success of the ISTEA in reducing funding restrictions.

SUMMARY AND CONCLUSIONS

This paper described the gradually changing emphasis of transportation planning at the state level from mode specific to multimodal. The survey provided a better understanding of how states have adapted to this new perspective following ISTEA 91. Although vehicle-oriented Transportation Management Systems received the most attention, over 60 percent of the states surveyed had an intermodal management system either completed or in process. Over half the states' plans included bus-rail and bus-air intermodal coverage for passengers, and truck-rail, rail-water, truck-water, and truck-air intermodal projects for freight.

The results also helped explain the general lack of integrated planning. State highways were addressed in traffic network models more than twice as often as any other infrastructure need. When asked specifically about multimodal issues, rail and highway conflicts, in both urban and rural settings, predominated. Nearly half the states included intercity bus and rail joint terminal locations as an intermodal issue in their new plans. States larger in area and lower in population density were also more likely to include intercity bus and air terminal joint locations.

Two-thirds of the states employ mode and system non-network models, rather than integrated network models for either passenger or freight transport (although larger and more urban states used mode-specific models more than did small, rural states). Nearly one-half used benefit-cost analysis. Over 70 percent of the states reported they need additional training in geographic information systems to use this tool for multimodal analysis. Over 50 percent require more training in transportation economics, transport network development and modeling, benefit-cost analysis, and in financial analysis. Clearly, the states have said that their skills necessary for applying more complex methods of analysis need upgrading to meet the multimodal emphasis established by ISTEA 91.

The original provisions of ISTEA expired on September 30, 1997. The "Transportation Equity Act for the 21st Century" became law on June 9, 1998. A logical extension of this paper will be to perform a follow-up survey with these same state planning agencies, after they have had sufficient time to review the new bill, to learn their perceptions of whether or not TEA-21 continues the multimodal focus of ISTEA and meets the needs identified as lacking in the 1991 law. Future transportation bills may be stronger if Congress recognizes, through the use of studies such as described above: (a) the issues that are important to the states, (b) the methods employed by state planners as they determine where best to invest their funds, and (c) less tangible needs of training to utilize new methods of analysis.

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Des Moines Metropolitan Area ITS Strategic Plan

Thomas Kane, T.H. Maze, and Vincent P. Pearce

The Des Moines Area Metropolitan Organization (MPO) completed an early deployment study for the Des Moines metropolitan area in late 1997. The purpose of the study was to develop a strategic plan for Intelligent Transportation System (ITS) deployment and to provide inertia for the development of ITS infrastructure. When the Federal Highway Administration sponsored the Des Moines metropolitan area's early deployment study, Des Moines was the smallest metropolitan area to undertake such a study. Therefore, there were no similar sized urban areas from which to draw examples. Further, although the metropolitan area and traffic volumes are growing, congestion is not seen as a significant problem in Des Moines. As a result of minimal traffic congestion, there was and is some skepticism among the transportation stakeholders in the need for Intelligent Transportation Systems (ITS). The development of ITS infrastructure is seen as even more problematic when the capital requirements for ITS must compete with the capital requirements of other, traditional transportation improvements. Despite the initial skepticism regarding the need for ITS and the ability to afford ITS, Des Moines area transportation stakeholders have become very supportive of the initiatives identified in the plan. The generation of support for ITS was developed through two galvanizing issues. The first was a focus on safety benefits of ITS as opposed to congestion reduction benefits. The second was to focus on the use of ITS to mitigate the impacts of the reconstruction of I-235. I-235 cuts across Des Moines running through the north side of the central business district and has been the single most important factors in forging commuting and development patterns in Des Moines areas. As a result, arterial streets which parallel I-235 will be greatly impacted by the diversion of traffic from I-235 which is likely to occur during reconstruction. ITS's ability to manage traffic under dynamic conditions provided an incentive for transportation stakeholders to rally behind the ITS strategic plan. The proposed paper will discuss the process used to develop the plan and review the plan's recommendations. One of the issues the plan attempts to address is the identification of technology appropriate for an urban area the size of Des Moines.

INTRODUCTION

Section 6055(b) of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) authorized the Federal Highway Administration (FHWA) to provide grants to state, local transportation agencies, and metropolitan planning organizations (MPOs) to conduct studies for the development of multi-year intelligent transportation systems (ITS) strategic deployment plans in metropolitan areas and along intercity corridors (*1*). The Intelligent Vehicle Highway Systems (IVHS) Early Deployment Planning Program was designed by FHWA to achieve ISTEA's objective of initiating the strategic planning process. The program initially targeted the 75 largest metropolitan areas, 30 major intercity corridors linking metropolitan areas and several statewide programs for the develop of ITS strategic plans. To date, the FHWA has 90 early deployment planning studies completed or underway (*2*). The Des Moines metropolitan area's ITS strategic plan was developed under this FHWA program.

Currently the Des Moines metropolitan area is the ninety-second largest metropolitan area in the U.S. (measured by population), therefore, Des Moines was not one of the targeted large urban areas. At the time the study was initiated (1995), it was the smallest metropolitan area to undertake such a planning effort (1996 estimated population of the Consolidated Metropolitan Statistical Areas is 427,000) (*3*).

Urban applications of ITS have been principally implemented to better manage traffic and incidents, provide traveler information, and manage public transportation in congested urban areas. Because of ITS's ability to mitigate congestion and unproductive traveler delays, the earliest applications of ITS technology appeared in large congested cities, and current ITS deployment activities are largely concentrated in large urban areas. This resulted in the Des Moines study having few examples of deployment in similar-sized cities to refer to, which raised the issue of what, if any, ITS deployment is appropriate in an area the size of Des Moines, with modest levels of congestion.

Although the Des Moines metropolitan area is experiencing growth, a respectable 8.8 percent rate of population growth from 1990 to 1996, congestion is not perceived to be a significant problem in the Des Moines area (*3*). The lack of significant congestion in the Des Moines area resulted in some skepticism about the need for ITS among the staff members of agencies responsible for transportation infrastructure in Des Moines area jurisdictions. The development of ITS infrastructure becomes even more problematic when the capital requirements for ITS must compete with the capital requirements of other, traditional transportation improvements. Despite the initial uncertainty regarding ITS, the transportation stakeholders in the region have become supportive of the ultimate ITS strategic plan because its recommendations are based on two galvanizing issues: 1) traffic safety and incident management on the area's freeway system, and 2) management of traffic during the reconstruction of the area's single most important highway facility, I-235.

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ITS Planning Principles

Because intolerable congestion does not exist in the Des Moines metropolitan area, its freeway, transit service, and arterial streets provide a relatively good level of service. This implies that the Des Moines metropolitan area, unlike more congested urban areas, is not motivated to develop ITS services to avoid or mitigate large investments in capacity improvements to temper burgeoning congestion. Instead, the region has the opportunity to build up and target ITS infrastructure strategically, without being pressured to make investments to alleviate existing congestions. With the opportunity to be proactive, rather then reactive, the Des Moines ITS strategic plan was developed based on the following principles:

- Identify achievable, economically feasible, and sustainable early winners for ITS projects.
- Build the core infrastructure incrementally using interoperable systems, while recognizing that the development of the ITS infrastructure and the services identified require a long-term commitment.
- Develop core ITS infrastructure in partnership with other transportation development programs and stakeholders with similar objectives. Capitalizing on opportunities to work in parallel with other projects will help to accelerate the construction of ITS infrastructure.

Study Organization

The Des Moines Early Deployment study covered the MPO planning area which includes portions of four counties and 16 municipal jurisdictions. In addition to the metropolitan area local jurisdictions, the Iowa Department of Transportation (Iowa DOT), the Iowa Department of Public Safety (Iowa DPS, the home agency of the Iowa State Highway Patrol) and other metropolitan and local jurisdictions and private organizations (e.g., transit, enforcement, incident response, motor carriers, etc.) have a considerable stake in the development of ITS services. However, because ITS is not planned and deployed on a jurisdiction by jurisdiction basis, the agencies with regional responsibilities must lead deployment efforts. The Iowa DOT is the only agency with infrastructure management responsibilities throughout the region and, therefore, the ITS strategic plan recommends that the Iowa DOT take the lead, while the Des Moines Area MPO serves as the champion for the plan's implementation in partnership with local jurisdictions.

The ITS strategic plan's development was steered by a committee representing the major public and private transportation infrastructure stakeholders in the region. Following the appointment of the steering committee, the first major step in the study was to develop a transportation inventory of items relevant to the development of a strategic plan. The inventory included information related to mapping and data management systems, travel and transportation management systems, public transportation services and facilities, and the current status of the use of ITS services in the Des Moines metropolitan area. This part of the project resulted in a geographic information systems (GIS) database, which contained much of the inventoried information, including traffic signal and signal systems, traffic counts, traffic accidents, and programmed facility improvements.

Using the inventory of existing systems, facilities, traffic and transportation characteristics and attributes, and existing ITS services, the steering committee targeted five topic areas for further development. The five topic areas are listed below, and although the topic areas do not cover all the ITS market packages, these were deemed to be the most important for application of ITS in the Des Moines metropolitan area.

- Incident Management
- Traveler Information
- Advanced Traffic Control
- Commercial Vehicle Operations
- Data Management

For each topic, a different approach was taken to study related issues and to identify candidate ITS applications. For incident management, traveler information, and advanced traffic control, a committee was developed to identify goals and objectives, institutional issues, and systems requirements. For commercial vehicle operations, project staff worked directly with the Iowa Motor Truck Association (IMTA), and the IMTA convened IMTA members to review the work developed by the project staff. Data management issues were identified through project staff discussions with technical staff for the constituent agencies and a meeting with constituent groups. The work in each of these topic areas resulted in the identification of specific market packages for further focused refinement.

To assist the subcommittees in visualizing traveler information systems, two static Internet home page systems were built. One of the systems presented transit information, including route and schedule information for all of the Des Moines Metropolitan Transit Authority's (MTA) fixed route service. The other provided information and identified points of interest to truck operators (e.g., locations where vehicles could be serviced or drivers could receive dental or medical services on a walk-in basis).

The next step in the planning process was to conduct a review of ITS technologies. To do this, a detailed evaluation was conducted of 169 technologies with respect to 12 criteria. The criteria included categorization and description of the technology, support of the technology, technology costs, and judgment evaluation of the technology's benefits and negative and positive attributes.

Given an understanding of the technology, an understanding of the issue to which ITS can be applied, and an inventory of what already exists, the project staff worked with the steering committee to identify specific ITS projects to be deployed over a 20-year planning horizon. The plan identified 45 separate projects or phases of activities to be developed in phases across the planning horizon. Most of the activities identified are to be completed or will be under way within the first five years of the planning period (1997 to 2002). The I-235 reconstruction, planned to start in the year 2002, provides a watershed for the proposed ITS projects. Prior to reconstruction, the focus is on the incremental establishment of ITS services in the urban area and implementation of management systems to ease congestion during reconstruction. During reconstruction, the focus turns to implementing ITS infrastructure on the I-235 corridor as part of the reconstruction. After reconstruction, the focus is turned to deployment throughout the urban area.

DEPLOYMENT RECOMMENDATIONS

Although it is clearly beyond the scope of this paper to discuss the proposed ITS deployment in any detail, proposed deployments are summarized below. Clearly the predominate area of deployment involves several interrelated ITS market packages to manage traf-

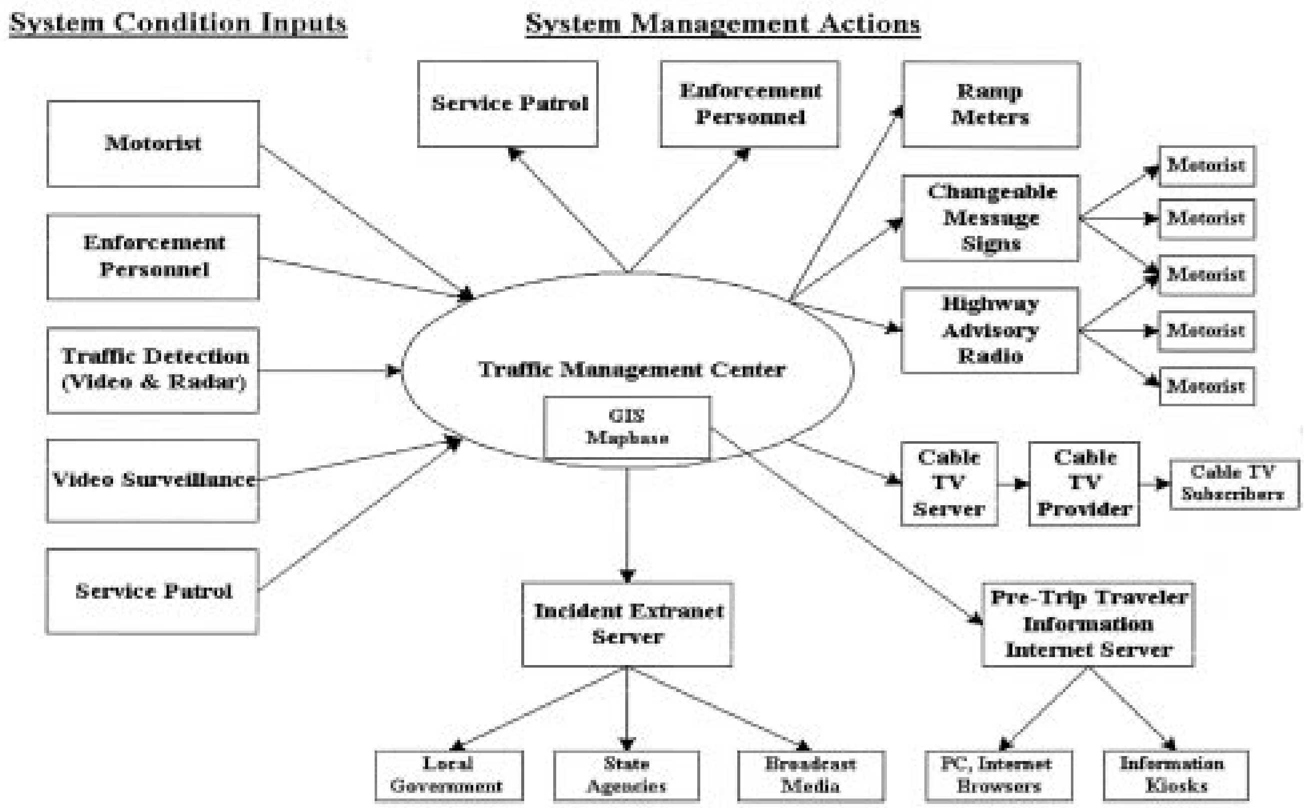

FIGURE 1 High-level transportation management center system.

fic (both on arterial streets and the freeway system), to provide traveler information, and to manage incidents. The development of a transportation management center (TMC) is a common requirement to traffic and incident management and traveler information. The plan addresses other areas where more modest deployment of ITS applications are planned involving advanced public transportation systems and commercial vehicle operations.

Traffic and Incident Management and Traveler Information

Currently there is no formal coordination of highway operations throughout the urban area. For example, which organization is responsible for clearing and managing an incident on the interstate system depends on the incident's location. Incidents occurring on the interstate system that are within a municipal boundary are the responsibility of the municipality. On parts of the system which are located in unincorporated areas, incidents are the responsibility of the Iowa Highway Patrol. In addition, the Iowa DOT recognizes that it shares the responsibility of the operation of the urban interstate system. As one example of the Iowa DOT concern for traffic management, it has erected changeable message signs (CMS) at the I-35 and I-80 northeast and southwest entrances to the Des Moines metropolitan area's interstate system.

Incident management operations of governmental organizations are coordinated informally, but no formal agreements or management structure exists, except in the case of extreme emergencies, where the Emergency Management Division of the Iowa DPS uses its statutory authority to manage an incident.

Plans for systems to support traffic and incident management and traveler information were divided into three time frames: 1) prior to I-235 reconstruction, 2) during I-235 reconstruction, and 3) following I-235 reconstruction.

Prior to I-235 Reconstruction

All the traffic and incident management and travel information systems planned are based on the assumption that a transportation management center (TMC) will be developed. The TMC will receive data from field data collection units (detectors and cameras) and observation data from field personnel (service patrol operators and enforcement) and motorists, fuse the data and coordinate or direct field personnel, manage traffic control systems and incident responders, and provide traveler information.

Although no firm recommendations were made on the location of the TMC, an attractive location involves locating the facility jointly with the central Iowa Highway Patrol dispatching center in STARC Armory. There is ample room in the armory for a TMC and there should be synergy between the Highway Patrol dispatchers and the TMC operators. The implementation of the TMC also requires that adequate communication system capacity be developed and a fiber optic network communication system is proposed with a loop that follows the I-235 and I-35/80 highway loop and later an outer loop around the east and south sides of the urban area following U.S. 65 and Iowa 5 (interstate design standard facilities). This design results in two self-healing, concentric loops.

One of the principal focuses for ITS application in the Des Moines metropolitan area is to prepare for I-235's reconstruction and traffic management during reconstruction. Therefore, to establish priorities for the incremental deployment of ITS, the study team used the urban travel demand model to determine routes which are most likely to be impacted as a result of capacity reductions during I-235 reconstruction. To do this, the twenty-four hour travel demand was run under scenarios where capacity at locations along I-235 were reduced to zero, and the resulting increase in traffic volumes were displayed on a map using a geographic information system (GIS). The results clearly identified which routes need to be improved in advance of the reconstruction. This analysis not only helped to target improvements on the interstate system, but also identified the need to improve traffic control and signing along parallel arterial streets. The resulting recommendations included:

- Development of a highway service patrol, consisting of private or public sector operators with direct communication to the TMC and with an established control structure over the deployment and activities conducted by the service patrol (regardless of whether they are public or private)
- Development of a freeway incident management plan for the entire metropolitan area
- Placement of surveillance cameras along I-235 and along major diversion routes
- Placement of traffic detectors at locations throughout the I-235 and I-35/80 loop (using radar and video technologies)
- Demonstrate and test ramp meters along the existing freeway system and on ramps with high accident rates
- Placement of changeable message signs along principal diversion routes and at locations in advance of possible route change locations (intersections)
- Implementation of a low-power highway advisory radio (HAR).
- Development of a GIS mapbase which may be used to transmit incident condition/location information to responding agencies using an Extranet; transmit traffic conditions using the Intranet to kiosks and personal browsers; and through a similar server, send information over the government access cable TV channel
- Develop agreements between local jurisdictions and implement interjurisdictional signal coordination along diversion routes which cross municipal boundaries.

Figure 1 shows a high-level system architecture for the TMC.

During Reconstruction of I-235

During the reconstruction of I-235, the traffic and incident management and traveler information systems are likely to be most effective in managing traffic diverted from I-235. Reconstruction of I-235 is expected to last roughly five years, and during this period, only a small number of ITS field assets are recommended in the plan. These assets consist of locating CMS and HAR along I-35 and I-80 past the systems interchanges on the northeast and southwest ends of the metropolitan area.

Following Reconstruction of I-235

Following reconstruction of I-235, additional field assets (detectors, CMS, video, and cameras) will be located along the interstate design standard facilities being constructed around the east and south of the metropolitan area (U.S. 65 and Iowa 5).

System Costs

The systems recommended for traffic and incident management and traveler information were designed to keep in perspective the appropriate system cost and technology for a medium-sized urban area like Des Moines. Parts of the system should be implemented in conjunction with the reconstruction of I-235 and the construction of the new south outer loop. In addition, the TMC is proposed to be located in an existing facility with existing communications infrastructure which further reduces costs. Further, the highway system is not densely populated with field devices (detectors, cameras, CMSs, etc.).

Depending on how the communication services are procured, the complete system is estimated to cost $150,000 to $300,000 per interstate mile under management, including the costs associated with TMC. The low estimate assumes the Iowa DOT will barter right-of-way easements for communication services, and the high estimate assumes that the Iowa DOT will develop its own communication network. The relative frugality of the system is illustrated when costs are compared to planning estimates used by the Minnesota Department of Transportation (MnDOT). The MnDOT estimates that it will cost $500,000 per mile of freeway to place an existing freeway under management, not including the cost of the TMC.

Advanced Public Transportation Systems

The transit authority in Des Moines (the Des Moines Metropolitan Transit Authority or MTA) already has a program where automatic vehicle location (AVL) systems have been installed on transit vehicles and, therefore, is already using ITS technology. In addition to its existing ITS features, two additional ITS functions were recommended for the MTA: 1) electronic fare payment, and 2) traffic signal prioritization.

Electronic Payment

Electronic fare payment typically allows the rider to pay using a card. Information is stored on the card using either a magnetic strip or a microchip. The services offered with a card can be viewed by the openness of the card's use and the functionality of the card. For example, a simple card with a magnetic strip on it may allow the user to store value either equivalent to the value of a specific number of trips or equivalent to riding the transit system over a certain length of time (week, month, or year). Such a card is a closed card because it can be used for only one purpose. On the other hand, a credit card is an open card because it can be used for multiple purposes and by multiple merchants, and possibly even for paying transit fares.

A card with a microchip imbedded is considered a smart card because the chip allows the card to perform computations rather than calculating the transaction through a host computer, similar to a common credit or debit card. Not only does a smart card allow for computations on the card, but it can also store much more information. Thus a smart card allows much more functionality.

In the case of the MTA, it is recommended that they initially start with a closed, paper ticket and magnetic strip system, while looking for other organizations or institutions which are also interested in electronic payment. It generally takes several applications to make the fixed cost associated with a smart card itself and the reader technology worthwhile. Commonly a financial institution offering credit, debit, and banking services on a smart card can provide enough uses for card holders to make migration to a smart card worthwhile. Later when other organizations or institutions, in partnership with the MTA, create enough uses to make smart cards cost effective, the MTA can migrate to the use of smart card technology.

Traffic Signal Prioritization

For bus operations on arterial streets, typically 30 percent of the their run time is spent being delayed at traffic signals. Where traffic signal prioritization has been used, typically the delay at intersections is reduced by half. Prioritization does not preempt the normal operation of a traffic signal, but instead extends the green to allow an approaching bus to pass while the traffic signal is still green or advances the phasing so that green is started early for an approaching bus. This creates benefits both for the bus patrons, by reducing their delay, and for other motorists by moving the bus through the intersection more quickly.

Currently the downtown Des Moines traffic signal system is being upgraded. It was recommended that signal prioritization be implemented as part of the new system. Assuming experiences in downtown Des Moines are positive, prioritization could be adopted by signal systems in other portions of the metropolitan area in the future.

Commercial Vehicle Operations

Commercial Vehicle Operations (CVO) market packages mostly involve either functions which are the private responsibility of the carrier (e.g., automated dispatching and fleet management) or are under the purview of the state and federal government (e.g., electronic screening, automated safety inspections, and electronic procurement of credentials). Although most CVO applications are not under the control of public organizations in the urban area, three actions were recommended. They are:

- Encourage state officials in Iowa and in adjacent states to adopt the applications market packages and system architecture defined as part of the national ITS program
- Maintain the static commercial traveler information system developed as a task within this project. (For more information see [*4*])
- Purchase the necessary computer hardware and software to allow the City of Des Moines Fire Department Hazardous Material Response Team to join the Operation Response Team.

CONCLUSIONS

This paper summarizes recommendations made in the Des Moines ITS Strategic Plan. Des Moines does not have the traffic congestion which has caused larger urban areas to embrace ITS. As result, the principal forces which are motivating the deployment of ITS are not to mitigate reoccurring or incident-induced congestion. The two principal motivations are to increase traffic safety and to reduce and manage the impacts of the reconstruction of I-235. In addition, because the Des Moines metropolitan area has modest requirements for traffic and incident management and traveler information, the density of field devices and the related costs are much lower than similar deployment in larger urban areas.

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System Architecture Efforts in the Gary-Chicago-Milwaukee Corridor

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The Gary-Chicago-Milwaukee (GCM) Corridor is one of four "Priority Corridors" throughout the country. These corridors were selected for special federal transportation funding based on very specific transportation and environmental criteria. One of the most significant efforts within the GCM Corridor is the Multi-Modal Traveler Information System (MMTIS). This effort is geared at providing a system for integrating Intelligent Transportation Systems (ITS) throughout the corridor. To support this effort, the MMTIS contract has produced the initial design and system architecture necessary for IDOT, and other GCM agencies, to build interoperable Advanced Traffic Management Systems (ATMS) and Advanced Traveler Information Systems throughout the GCM Corridor. In addition, there are at least six new multimillion dollar computer system projects in the GCM Corridor either currently underway, or beginning within the next two years. Parallel to these efforts, the United States Department of Transportation has recently invested millions of dollars in the development of a National ITS Architecture. This paper will discuss the work underway within the GCM Corridor and its relation to the National ITS Architecture efforts. Key words: traveler information system, Intelligent Transportation Systems, system architecture, Gary-Chicago-Milwaukee corridor.

INTRODUCTION

The Gary-Chicago-Milwaukee Corridor extends approximately 130 miles along the edge of Lake Michigan. It spans three states and 16 urbanized counties. Within the Corridor, various ITS efforts and programs have been developed over the last 40 years. This has resulted in many legacy systems that were developed before the National System Architecture was developed. The Corridor Transportation Information Center, which is discussed in further detail later in this paper, is one of these legacy systems. It grew out of *ADVANCE*, the dynamic invehicle navigation system, and was/is a prototype for a realtime traveler information system. The GCM partners are now building the successor to the C-TIC and the cornerstone of the future corridorwide efforts, the Gateway transportation information center.

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BACKGROUND

The Intermodal Surface Transportation and Efficiency Act (ISTEA) of 1991 created four corridors throughout the United States that were to receive priority attention due to their congestion problems. The GCM Corridor was one of these priority corridors. It includes all major roadways, airports, transit and rail systems, ports and intermodal transfer stations. The GCM Corridor thus offered the opportunity to support national ITS operational tests and to provide a multistate testbed for implementation and evaluation of ITS.

The GCM Coordinating Committee represents the major partners in the Corridor and is responsible for providing direction to the development of ITS. It consists of representatives from each of the three state Department of Transportation (DOTs) (Wisconsin, Indiana, and Illinois) and the Federal Highway Administration (FHWA). Under the auspices of the GCM Coordinating Committee, a Corridor Program Plan was developed which proposed the development of the Gateway. The Gateway will be a regionwide transportation information center collecting information from operating agencies and other sources within the three state area and distributing this information back to the operating agencies and other users. The remainder of this paper details the Gateway design efforts which were undertaken by De Leuw, Cather & Company with assistance from JHK & Associates and HNTB Corporation.

METHODOLOGY

Before it was possible to develop a system architecture design for the Corridor, it was necessary to inventory ITS projects that currently exist or are planned. Using a combination of interviews and questionnaires, data was collected on the 80 plus ITS projects, both public and private, throughout the Corridor and the needs of the users. Over nine different location referencing systems (latitude/longitude, street name, etc., each of which had different variations/bases) were identified in use within the Corridor which has the potential to hinder data exchange.

The next step in the definition of the system architecture design was to develop goals and objectives for ITS within the Corridor. These goals involved such items as allowing for two way data exchange; National Communications for ITS Protocol (NTCIP) compliance; conformance with the National ITS Architecture; and joint control of field devices. Each of these goals in turn were restated in performance criterion and applicable measurements. As discussed in the next section, these goals and associated criterion were used in the analysis of alternative system architectures for the Corridor.

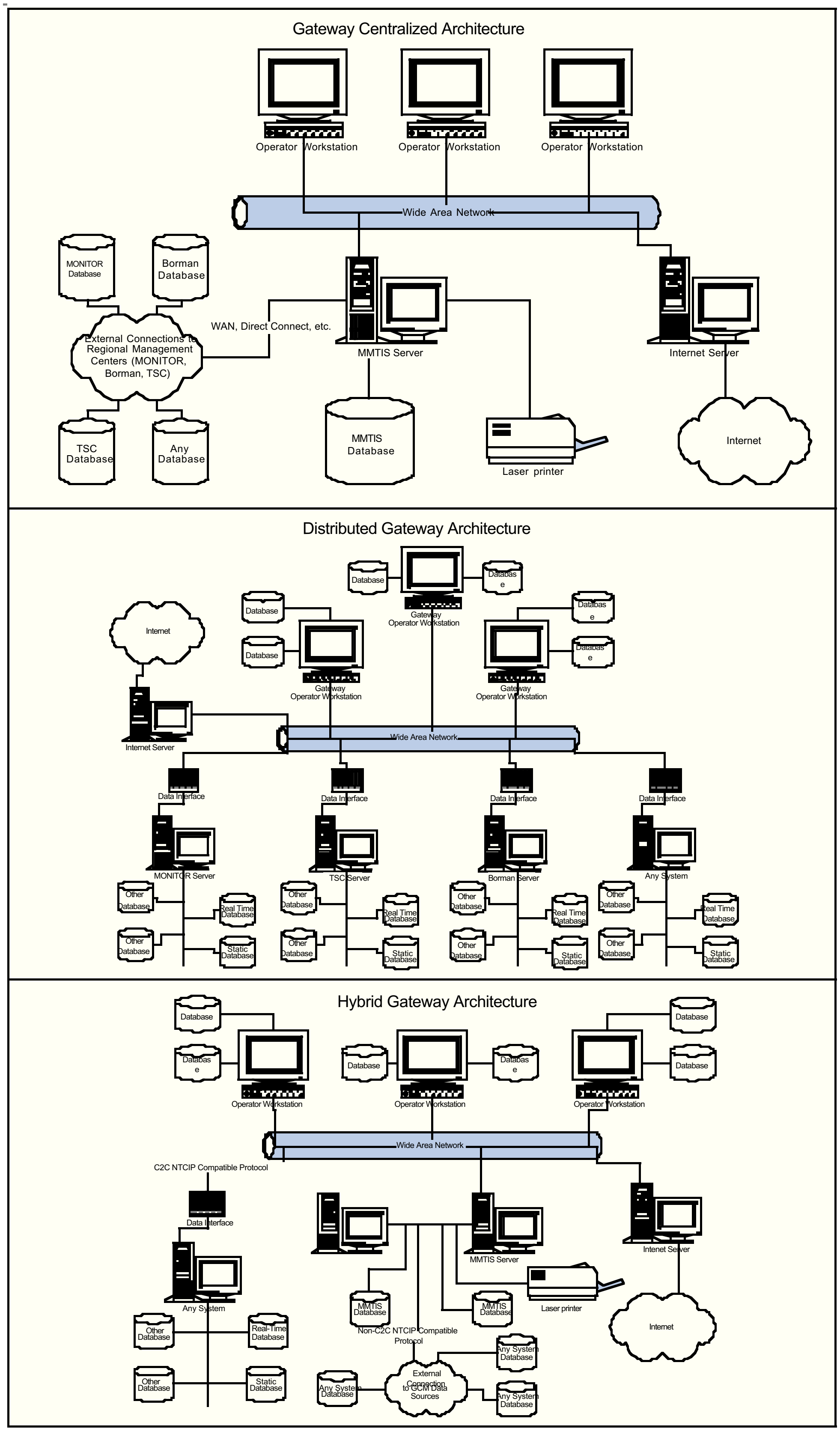

FIGURE 1 Gateway potential architectures.

SYSTEM OPTION DEVELOPMENT AND ANALYSIS

Following a systems inventory, design options were next developed. These included a review of centralized versus distributed systems and hybrids. The three potential architectures are as shown in Figure 1. Mapping of the alternate architectures to the National System Architecture was also performed. Data flows were identified and a list of potentially applicable interface standards was developed. Based upon feedback from these participants through meetings and workshops, an architecture was developed for the Corridor that was agreeable to the participants. This architecture is presented in the next section.

RECOMMENDED SYSTEM ARCHITECTURE

Overall System Architecture

The architecture proposed is based upon the MultiModal Traveler Information System (MMTIS) which is the name applied to the overall ITS scheme in the Corridor. MMTIS will be implemented using staged implementation. It is an information based system which will also facilitate joint monitoring and control of field devices such as variable message signs. The Gateway is the central element of the Corridor and will be the transportation information hub. The Gateway collects dynamic and static transportation data from the distributed transportation management systems throughout the corridor. This information will be presented directly to travelers, to transportation system operators and to information service providers. The Gateway will also serve as a "go-between" for various users who will eventually share control and monitoring of field devices such as Closed Circuit Television (CCTV). The Gateway has been designed as a distributed system as shown in Figure 2. Regional hubs in each state are responsible for collecting data within their respective states and providing it to the Gateway server. This server then distributes corridor wide data back to the regional hubs for their own distribution and use. Regional hubs also have the ability to distribute data they collect. Minimal data fusion will occur at the Gateway server as it is intended that the regional hubs will perform any necessary data fusion prior to transmission to the server.

Conformance With National Architecture

The developers of the National ITS Architecture envisioned that ITS systems would be deployed in an ad-hoc manner with a variety of combinations of subsystems and functionality. To ensure national

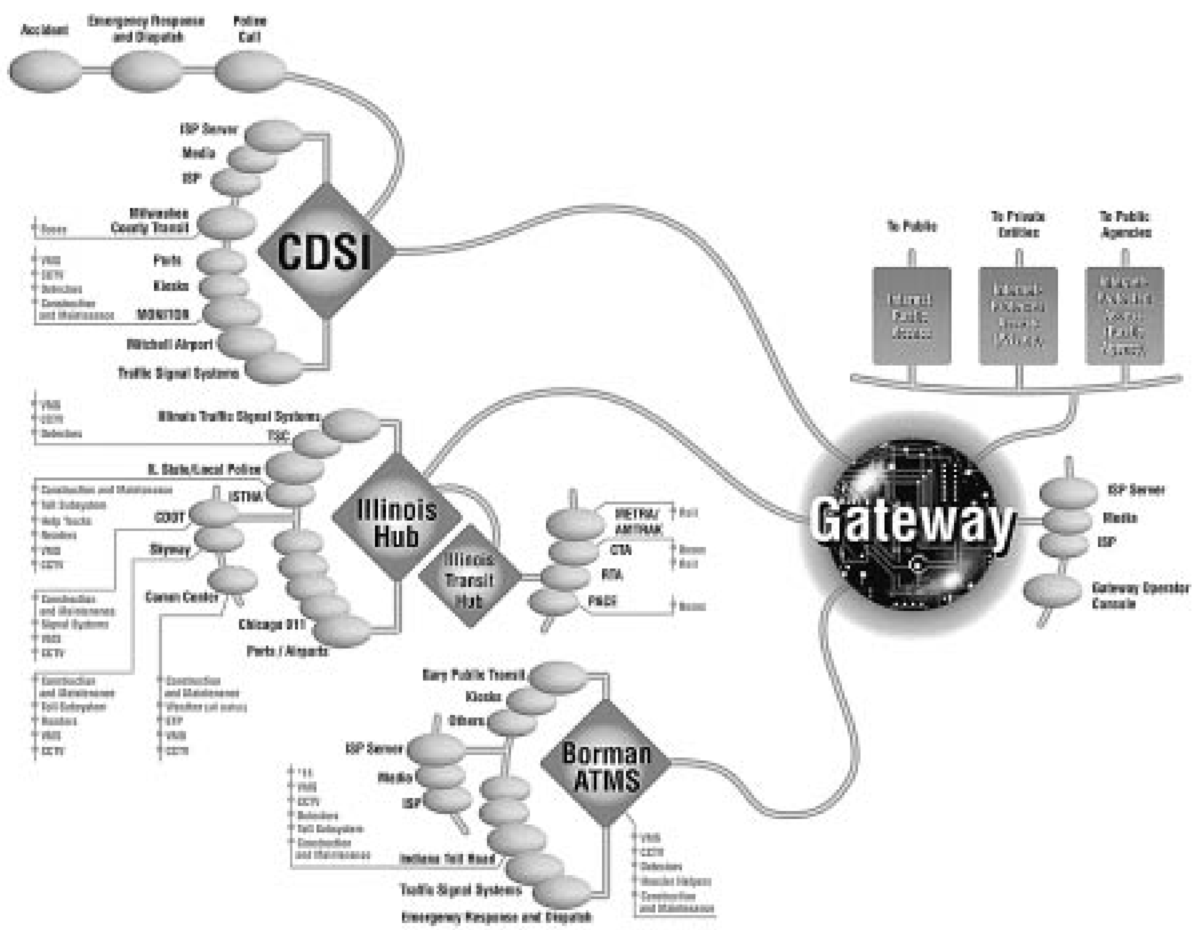

FIGURE 2 Gateway ultimate phase.

and regional system compatibility however, standards for the architecture's interfaces and data flows are necessary. Thus, the degree of compatibility of an architecture or a system with the National ITS Architecture can be measured by its use of national interface standards. National interoperability is specified for all interfaces to mobile subsystems:

- Information Service Providers (ISP) to Personal Information Access Subsystems
- Toll Collection Subsystems to Vehicle Subsystems
- Commercial Vehicle Subsystems to Traffic Management Subsystems.

Regional interoperability is specified when the coordination issues are regional rather than national in scope:

- Traffic Management Subsystems to Transit Management Subsystems
- Traffic Management Subsystems to Information Service Providers
- Traffic Management Subsystems to Traffic Management Subsystems.

As the Gateway and the MMTIS are regional in nature, they should conform to regional standards. However, there is nothing to prevent them from following a national standard (e.g. NTCIP) and there are very valid reasons for so doing. For example, by following a national standard for Traffic Management Subsystems to Information Service Providers, much wider distribution of transportation data will be obtained as it will be more attractive to national providers.

The principal national standard is the NTCIP. The primary objective of the NTCIP is to provide a communications standard that ensures the interoperability and interchangeability of traffic control and ITS devices. The NTCIP is actually a family of standard communications protocols used for data transmission within and between ITS. Work is under way on additional protocols for applications such as computer to computer or traffic management center (TMC) to TMC data exchange, which are directly applicable to the Gateway. Other efforts are involved in communications within transit management systems and communications with and between moving vehicles which are applicable to MMTIS.

Another other aspect dealing with the Gateway and MMTIS obtaining compliance with the National ITS Architecture involves the use of a modular approach. Software will be developed in modules that can be modified or removed as more details emerge on NTCIP and the National ITS Architecture.

Lastly, compliance with the National ITS Architecture will be obtained thru the use of the LRMS which is being developed under the auspices of the Federal Highway Administration. As previously noted, the agencies in the Corridor use a number of methods for referencing and tracking of geographic information. However, in order to be able to exchange data between agencies and also to allow for the interoperability required nationally, a point in space must be able to be identified by all parties which will be possible thru LRMS.

Thus, in summary, compliance with the National ITS Architecture will be obtained through the use of standard, nationally recognized interfaces (NTCIP); through the use of modular software to accommodate changes on the national scene; and through the use of a common referencing system (LRMS).

National ITS Architecture Implications and Gateway Current Status

The National ITS Architecture has been under development for several years. However, there remain many details/standards yet to be finalized. Efforts are underway in several committees such as those of SAE and ITE to develop these standards. As very little has been formally adoped, all current ITS activities are somewhat at risk since they may have to undergo change to meet future standards. This issue has been of great concern to the GCM partners and is an issue we have made great pains to address. The following paragraphs review the degree of compliance and status of various elements in the Corridor.

The Corridor Transportation Information Center (C-TIC) is being used as the prototype for both the operational test of the initial communications network and for the Gateway. At the time of this writing, it has been in operation for over 30 months. Linked color coded congestion maps of major roadways in the corridor between Milwaukee and Gary have been placed on the Internet. The addresses on the Internet are http://www.ai.eecs.uic.edu/GCM/GCM.html and http://www.GCM.travelinfo.org (after June 1, 1998).

The C-TIC is not in compliance with the National ITS Architecture and will be replaced by the Gateway. Current real time data inputs to the C-TIC include: *999, NorthWest Central Dispatch, the Illinois State Police, the IDOT Traffic System Center, and MONITOR. These links as described in the following sections will be retained during the Gateway implementation. Anecdotal data is also input manually to the C-TIC on a daily basis that relates to construction/maintenance operations undertaken by the three state DOTs, Chicago DOT and the Illinois State Toll Highway Authority. These manual inputs will be converted to electronic inputs during the Gateway implementation as the sources are automated.

*999 is the cellular phone based motorist aid system. Operators are able to answer a call, locate the incident on a NavTech electronic map database (with automatic assignment of a NavTech link ID), input the incident into a retrievable database and alert the primary response agency. Real time data from *999 is then electronically input into the C-TIC for broader distribution. The *999 system currently processes up to 300,000 reports annually for the Chicagoland area. This system is currently not in conformance with the National Architecture Standards and changes will need to be made to the referencing system.

Inputs to the C-TIC from NorthWest Central Dispatch (NWCD), an emergency dispatch operation for seven suburban communities in the outlying Chicago area, have also been automated, as has the connection to District 15 of the Illinois State Police. For these two systems, filtering occurs on a personal computer at the source to delete confidential/proprietary information as well as incidents that will not affect traffic flow. Upon arrival at the C-TIC, the location data within the incident message is deciphered via table lookup for use in the C-TIC database. To make this connection compliant with the National ITS Architecture, changes will need to be made to convert location data to the LRMS format.

Inputs from the Traffic Systems Center TSC), IDOT's freeway management system in the Chicago area, include speed, volume and occupancy data at five minute intervals for each of over 1500 loop detectors. The C-TIC converts this data to travel times and color coded congestion displays. The TSC is currently undergoing an upgrade and will use the LRMS format and NTCIP standards for TMC to TMC protocols.

The C-TIC also includes a connection to MONITOR, the freeway management system in Milwaukee. This system provides travel times from loop detectors on the Milwaukee freeway system as well as message displays from the Variable Message Signs (VMS) and Highway Advisory Radio (HAR) systems. CCTV images can also be viewed as snapshots. WisDOT is currently developing a regional hub for the Gateway which will convert the data from MONITOR to LRMS format and make it NTCIP TMC to TMC compatibile.

The Illinois Toll Highway Authority has an electronic toll system (I-Pass) and a prototype traffic management system has been successfully demonstrated that involves reading the toll cards in the vehicles at selected locations and developing travel times for links. As the traffic management center is developed, processed travel times and congestion levels will be sent to the C-TIC using LRMS format.

A connection is also under development to the Advanced Transportation Management System (ATMS) on the Borman Expressway in Indiana which will serve as the regional hub for Indiana. Data on travel times, incidents, HAR and VMS will be sent to the Gateway using the LRMS.

CONCLUSION

Multiple agencies in the three state area are continuing to cooperate under the GCM banner. The region has already created a regional architecture compatible with the National ITS Architecture. As the Gateway is created and the region integrated through this one system, the National ITS Architecture will be implemented and the existing legacy systems brought into conformance.

REFERENCES

Technical reference documents are on-line at http:www//gcmpic.ai.uic.edu/piclib.html

Organizing a High Accuracy GIS Prototype Using Diverse Existing Elements

ED KRUM, JOELLA GIVENS, AND DAVID P. PIEPER

Geographic Information Systems (GIS) and spatial data applications at the Minnesota Department of Transportation (Mn/DOT) are numerous and diverse. Like most large organizations, nearly every application and data set at Mn/DOT has been developed independently in an isolated environment. In the rapidly developing world of network accessibility and cross-platform applications, the need for a more accessible GIS has been recognized. The Metro Division of Mn/DOT has initiated a pilot project designed to study the feasibility of implementing a high accuracy GIS that is accessible and usable by everyone in the division. The Metro Division manages program delivery and operations for the seven-county (Twin Cities) area of the Minnesota Department of Transportation. The pilot is also designed to perform a cost-benefit analysis on collecting high accuracy basemap information. The body of this pilot focuses on the development of an implementation plan. Project scoping and methodologies are crucial in this endeavor. As the project evolves through the phases of conceptualizing data items and applications, the prototype will introduce logical object, data, and application models in a dimension that is new to Mn/DOT's Metro Division. There are many technical issues to manage as well. Several prominent GIS and data technologies will be explored. This project manifests a fascinating blend of business procedures, technical skill, and communication transactions. As each of these components becomes more important in Transportation GIS, it will be demonstrated that it is in fact possible to integrate all of these elements successfully and push the transportation industry into the next generation of information systems. Key words: enterprise, application, implementation, process, high-resolution.

INTRODUCTION

Geographic Information Systems (GIS) and spatial data applications at the Minnesota Department of Transportation (Mn/DOT) are numerous and diverse. Like most large organizations, nearly every application and data set at Mn/DOT has been developed independently in an isolated environment. In the rapidly developing world of network accessibility and cross-platform applications, the need for a more accessible GIS has been recognized.

Mn/Dot's Metro Division manages program delivery and operations for the seven-county (Twin Cities) area of the Minnesota Department of Transportation. The entire department employs about 5,000 people, and the Metro Division is about one third of the employee total.

The Metro Division of Mn/DOT has initiated a pilot project designed to study the feasibility of implementing a high accuracy GIS that will be accessible and usable by everyone in the division. Proposal and initiation of the pilot are a result of the increasing need for higher resolution and better communication in GIS mapping. There is a call for data sharing to become more common and easy to use in daily transactions. A two-year period has been established to develop the prototype and the metro-wide implementation plan.

The resulting prototype will make GIS data and applications centrally available with varying levels of access, and focus on creating a one-stop basemap that is created and maintained with the highest possible resolution. Since more and more data in the organization are being created with higher accuracy standards, the prototype GIS will provide centimeter accuracy for centerline, milepost, and section corner data, and sub-meter accuracy for all other data (except where greater accuracy is required on a per-project basis). Methods will also be developed to associate data relationships within Mn/DOT's systems of managing spatial data through a common database. The project identifies supplementary non-spatial and external data links that are usable in a GIS.

MISSION

The High Accuracy GIS Prototype will introduce a GIS model to Mn/DOT Information Systems that is easily accessible and usable across platforms and envelops the geo-spatial accuracy needed for business functions within the Metro Division.

GOALS

1. Model a GIS that interfaces existing and future Mn/DOT data
2. Evaluate accuracy solutions for basemap products
3. Examine efficiency enhancements for Metro Division databases
4. Develop an Implementation plan.

PURPOSE

Implementation and study of this prototype enables Mn/DOT to move further into the cutting edge of high technology with new tools and methodologies that enhance the existing system and better serve the business needs of the organization.

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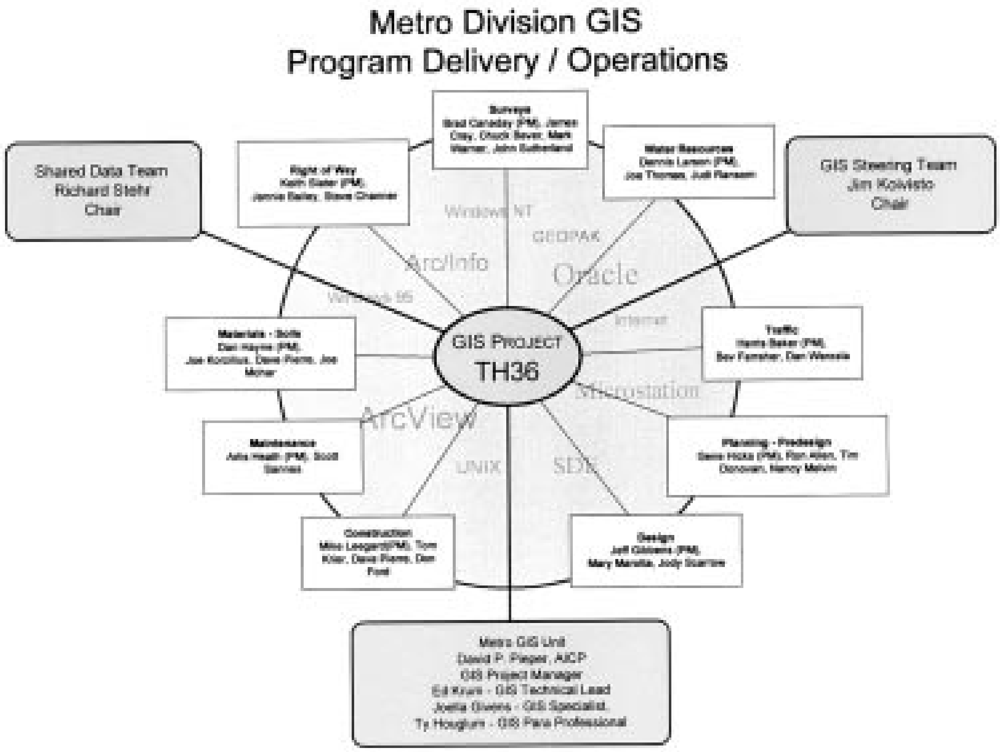

FIGURE 1 Organizational relationship model.

HISTORY

Historically, similar projects at Mn/DOT have not met implementation, or at best, were only partially implemented. A closer look at these projects shows that they might have failed because too much area (geographic and business) was covered in the initial plan, the scope of the project was not laid out clearly enough, or the technology was not yet in place to build an effective system. Communication with upper management also became a problem when attempting to secure funding or support necessary organizational changes.

In an effort to procure success of the current project, predecessors' "lessons learned" are being examined closely. A fundamental consideration in this effort is to limit the geographic extent during the period of study, but set up metrics to measure the scalability of implementing the plan across the metro area. Another extremely important factor is to explicitly define the number the components of the prototype well before modeling or application development begins. Extensive measures will be made to ensure quality and integrity of the proposed system and that there is a business need that warrants every prototype application that is created. Effective communication will certainly be a primary objective with all levels of Mn/DOT personnel.

In most cases, the technical activities required to carry out such a project in the past were not able to be coordinated within the organization. The type of data, systems, and applications being used were not mature enough to be used in this way, and technical flaws (however inconsistent or unpredictable) were considered normal. Now we have network accessibility and the ability to easily cross platforms in applications that never existed before. For example, data communication can be made between Windows and UNIX, GIS and CADD, and to the world via the Internet.

In addition to rapid technological advancements, methodologies and process management techniques are greatly improved in the GIS arena. As these higher technologies become more mature and better known to the GIS community and organizations such as Mn/DOT, the processes for developing them benefits from lessons of the past.

PROJECT TEAM AND RESOURCES

The Metro Division has organized a team lead by a core staff of three people to design the prototype models and applications, and devise the implementation plan (Figure 1). There is a project manager, technical lead, and project specialist. Student workers and graduate engineers are also working on the project.

The staff needs to reach out to many groups within Mn/DOT to design and create the prototype. A significant effort has been made with each of nine functional groups to ensure that the business needs of the metro division are met in this prototype. Functional group is the term used by Mn/DOT to describe a team or group of teams that has a specific purpose to the overall workings of the organization. For example, the Right-of-Way Group manages real estate for the organization. Within each functional group, three people are assigned to interface with the GIS prototype, each with a specific role. The Project Manager role is the point-of-contact for the group, the Technical Expert is the person who is most familiar with the group's data, and the Process Expert is responsible for identifying and communicating the business needs of the group.

The relationship of the TH36 project team with other significant factions at the Metro Division is depicted in Figure 1.

STUDY AREA DESCRIPTION

The study area for the model is the full corridor of Minnesota State Trunk Highway 36 (TH36). This highway is of special interest because of its wide variety of characteristics, including: entrance/exit ramps, traffic control signals, regulated entrance signals, rural and urban design, crossing of county and municipal boundaries, future development plans, and current availability of data.

PROCEDURES

Conceptualization

First efforts to define and tame the initial concept of the prototype were challenging. After the preliminary project goals were identified the project team was assembled. The next major task was to build consensus and identify key players in Mn/DOT Metro Division functional groups. Then the team began to build relationships with Mn/DOT contacts outside of the Metro Division, and disclose relationships with individuals and organizations outside of Mn/DOT who would have either input or interest in the efforts of the project.

Scope

After initial meetings and getting a feel for what the requirements for the project were, the team began to understand what was possible to do in two years and what was not. Since many useful ideas emerged that were clearly out of scope, a pool was created to collect ideas to be proposed as future projects (refer to the pool in Figure 2). Resources for the project were identified which included the creation of the core GIS staff, existing Mn/DOT staff and resources, and consultants.

Technologies that are applied in the prototype include networks, Windows (NT and 95), Microstation, ESRI products (corporate standard for GIS), Oracle (corporate database standard), MS Access (and other database engines), GEOPAK, Internet (WWW) Technology, and Spatial Database engines (SDE). Many other proprietary applications and data storage systems are currently in use.

The scope of the project is bound by the constraints of three major elements: construction of a GIS basemap, database design and architecture, and applications to demonstrate the prototype.

Basemap

High Accuracy Solutions will be examined for a metro-area basemap that has greater resolution than existing sources. The current Mn/DOT basemap being used for general business analysis is estimated to be accurate within 40 feet, with a resolution of 1:24,000. This resolution does not meet the business needs of functional groups that collect data with precision survey and GPS instruments. Many survey-quality maps exist in the CADD environment, but have limited geographic extents and are valid only through the life cycle of particular construction projects.

Through the use of new technologies in GIS mapping, the basemap for the prototype will strive for centimeter accuracy for centerline, milepost, and section corner data, and sub-meter accuracy for all other data that warrant a need for high accuracy. This will be achieved through the use of photogrammetics based on low level flights (1500 feet), video surveying from van and helicopter viewpoints, laser ranging, and Global Positioning Systems (GPS). Existing data derived from traditional surveying methods will also be utilized where appropriate.

Database

Modeling and building the objects and architecture of the Metro Division database are the most involved tasks that the project team faces. Most of the data sets that will be used in conjunction with the high accuracy GIS basemap already exist, but are held in a variety of formats. Many of the data sets are stored in isolated and proprietary formats. Some are generously distributed, but are engineered with redundant (de-normalized) and irregular structures. The feasibility of integrating a homogenous database structure into the prototype will be evaluated by incorporating the data object model in third normal form. More simply, all data for the prototype will be placed into one database - Oracle, and each table will be reduced to its simplest form. This will provide flexibility for query and transaction management of the data as well as provide a template for future attempts by Mn/DOT functions to enhance the efficiency of data sharing.

Applications

In our initial assessment it is clear that a single application will not meet the needs of every user. There are many users who will only have simple queries to the database. They should not be required to learn complicated interfaces such as ArcView, Arc/Info, or Oracle. Others will need to be able to manipulate fine details in the database. Figure 2 illustrates this concept.

The prototype application will be a proof-of-concept that business needs can be met by combining a homogeneous database structure with a high accuracy GIS basemap. A suite of applets will be employed to encompass the solutions to many problems. The aspects of managing different types of data (such as point, line, and polygon features) will each need to be sampled so other data elements can be incorporated into the model upon implementation. An array of data access will also need to be considered, so simple functions will be constructed to interact with data at many levels. These levels (referred to in Figure 2) correspond with applications that range from simple queries to full authority over data.

Project Plan

The project plan is based on a template supported by Mn/DOT's Office of Information Resource Management. Stages of the project unfold from conceptual to logical, and finally physical design and

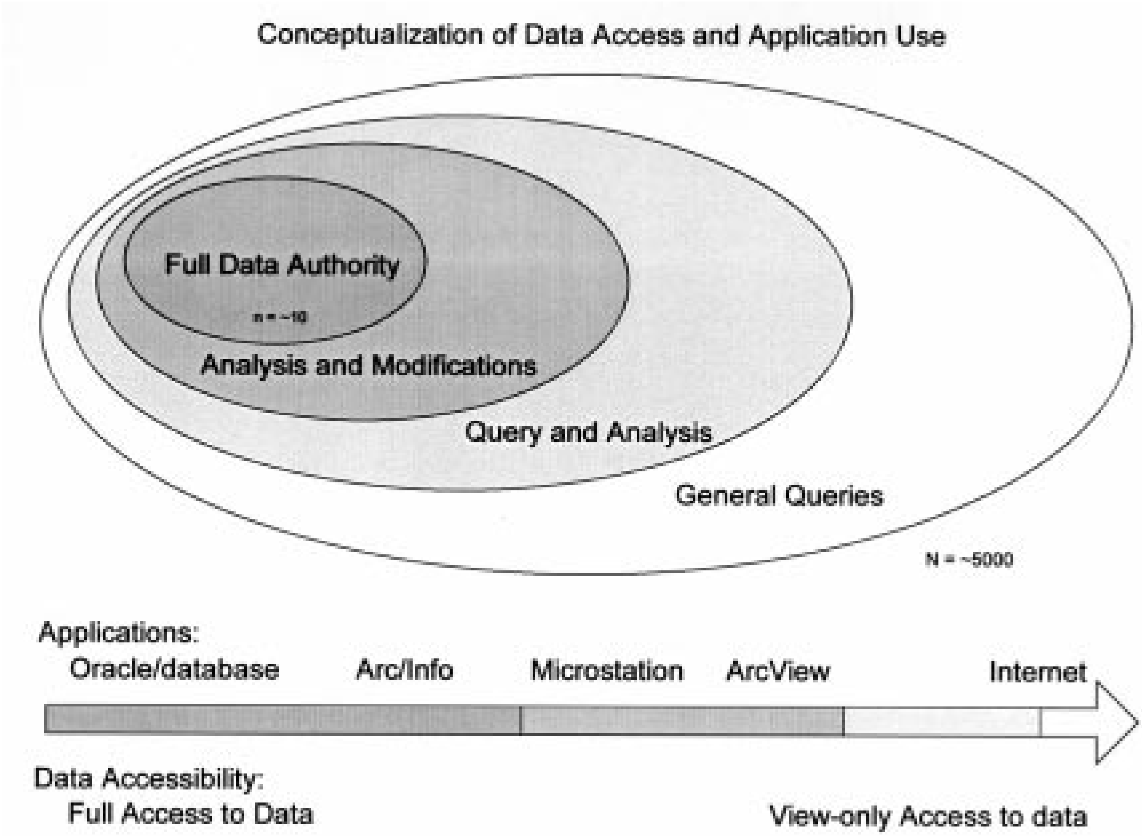

FIGURE 2 Database access and application model.

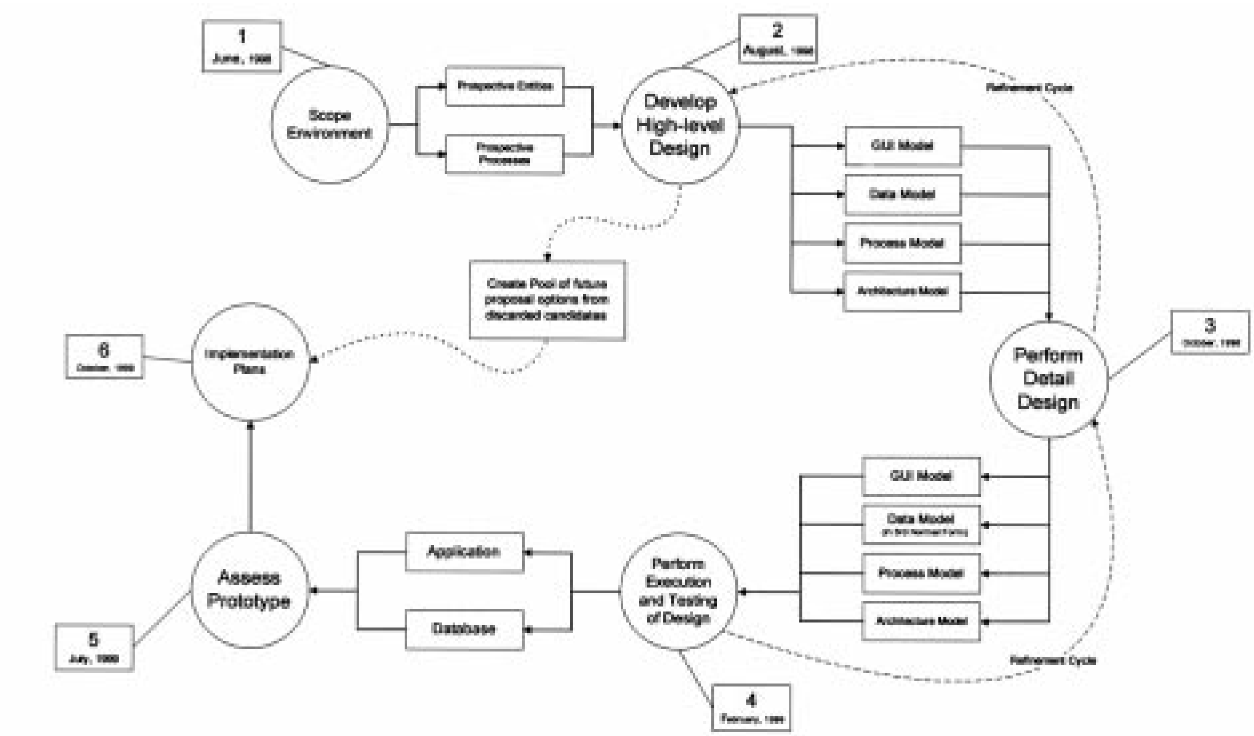

FIGURE 3 Project plan model.

construction of graphical user interfaces (GUI's), applications, databases, and architectures that will be the essence of the prototype. Final stages of the process will include user and stress testing of the prototype. When the quality and integrity of the prototype can be assured, an implementation plan will be presented with supporting demonstrations. Quality will be monitored throughout the process. The key elements of the plan and schedule are modeled in Figure 3.

ANTICIPATED HURDLES AND ISSUES

The first approach to a participating group or individual is very critical. People are used to doing work that gets a specific job done or meets their business needs only. Integrating into the bigger system is a lot of work. This is a technical challenge, and it also requires people to overcome the constraints of keeping data in the local arena and moving into a more global way of thinking and communicating. While Mn/DOT's business needs must be addressed as a whole, the functional groups will need to remain empowered within their own areas of expertise.

The issues of stewardship agreements, conflicting data needs, software licensing, data access, missing and sensitive data, and resource allocations will need to be scrutinized. These problems will be particularly pertinent when facing the complexity and scalability of the model.

The team is determined that a positive atmosphere of change can be promoted by anticipating these hurdles. The prototype can foster cooperation and trust by effectively demonstrating the benefits of interactive data sharing across functional groups.

IMPLEMENTATION PLAN

Working towards completion of the TH36 model, an implementation plan will be created for the Metro Division, which may or may not follow the prototype. The effective plan will engage changes from the prototype as well as successful components. While the models for applications, databases, architectures, and basemaps are not expected to be "throw-away" products, the core product of the project, the implementation plan, will specifically address how the high accuracy data sharing concept can be realized when promoting the scale to a metro-wide universe.

Core elements of the implementation plan are cost benefit analysis, time frame for Metro implementation, cost and resource estimates, process model, hardware/software needs, system and database administration, staffing issues, maintenance options, and others which are revealed during the prototyping process. Provisions will be made for enhancements to the methodologies as technologies and business needs change.

The plan will be designed as a multi-purpose tool that can be broken down into many parts. As a whole, it will be directed towards implementing an accessible, high-accuracy GIS in the Metro Division of Mn/DOT. Modules can be broken out of the plan that can be shared with others, including other Mn/DOT offices, other DOT's, and facilities management organizations. Reusable code from applications will be identified and documented. A "Lessons Learned" section will be quite conspicuous. Some of the "pooled" ideas that fall outside of the project scope will spawn proposals for future projects, and a summary of those proposals will be included.

CONCLUSION

The TH36 High Accuracy GIS Prototype manifests a fascinating blend of business procedures, technical skill, and communication transactions. As each of these components becomes more important in Transportation GIS, we would like to show that it is, in fact, possible to integrate all of these elements successfully and push the transportation industry into the next generation of information systems.

Iowa Department of Transportation Statewide Coordinated GIS

WILLIAM G. SCHUMAN, TIM STRAUSS, DAN GIESEMAN, AND REGINALD R. SOULEYRETTE

This paper details a project conducted by the Iowa Department of Transportation (DOT) and the Center for Transportation Research and Education (CTRE) to construct a statewide coordinated GIS. This effort was undertaken in response to increased pressures on transportation agencies to improve their efficiency and accountability through performance-based planning. The first step in response to this pressure was to better integrate the disparate data throughout the Iowa DOT. The project team conducted interviews of data collectors, managers, and users. They then identified key data elements, determined the system architecture needed to support and maintain the DOT's data, and created the necessary data conversion procedures to populate the GIS database. As most transportation data are spatial in nature, GIS was the logical choice for data integration. Implementation of the Statewide Coordinated GIS is progressing at the Iowa DOT using GIS, data warehousing, Local Area Network (LAN), and Wide Area Network (WAN) technologies. Internet and CDROM solutions will be used to distribute data to organizations external to the Iowa DOT. While these methods facilitate the sharing of data, it is very important that the DOT design a comprehensive data maintenance, metadata, and distribution strategy so that well documented data are provided on a regular cycle. A graphical user interface will provide a straightforward method of accessing the disparate data sources without requiring the end user to be familiar with the complexities of the underlying data sources. An interface to provide online metadata to the end user is also part of the system development. Key words: management systems, GIS, database, warehouse, metadata.

INTRODUCTION

The Iowa DOT has a wealth of information stored in many database systems throughout the organization. Some of these systems are well established and maintained, and others are smaller data sets collected by an individual for a specific office. In either case, while data are valuable in isolation, integrated data sources provide a much broader view of the information and the interrelationships of the data.

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Data integration is necessary for successful implementation of many national transportation initiatives. The Intermodal Surface Transportation Efficiency Act (ISTEA) initially mandated the creation of systems that would have required the development of integrated databases. While no longer mandated, the requirements of these systems heightened an awareness of the value of coordinated data to transportation professionals and management. In addition, Iowa's Blue Ribbon Task Force and the IowAccess initiatives have emphasized taking advantage of technology to provide more efficient methods of making decisions and interacting with Iowa's large and growing information infrastructure.

In most instances, the spatial component has been the element for facilitating the integration of these data. Many advances have made it more practical to provide integrated spatial data. The hard technologies, such as faster computer systems and better computer networks, have allowed the data to be made available on the user's desk. Easy to use software, such as Microsoft Windows GIS applications, have also played an important role in the ability of an organization to get the needed tools into the hands of the users. Even more important than these technological advancements may be the affordability of the technologies. GIS, until recently, was a tool only available for larger, well-funded organizations due to its high costs and learning curve. Lastly, the opening of proprietary data structures has made it easier to share data between systems, thus facilitating the sharing of data between or within organizations using different software products.

EXISTING SITUATION

The Iowa DOT has many offices that collect and manage data. While these data are important in the development of a Statewide Coordinated GIS, many other data sources exist outside the DOT that would aid in more efficient and better decision making. These internal and external data sources are described below.

Internal Data Structures

The data maintained by the DOT are related in many ways. The relationships can be direct, such as using a specific value from certain data entities, or indirect, such as a spatial relationship (e.g., a road crosses through a wetland). Figure 1 demonstrates the complex data relationships that exist at the DOT.

This situation is not unique to the Iowa DOT. Many DOTs in the United States have identified the need to better integrate their data and to identify the redundant data collection and maintenance

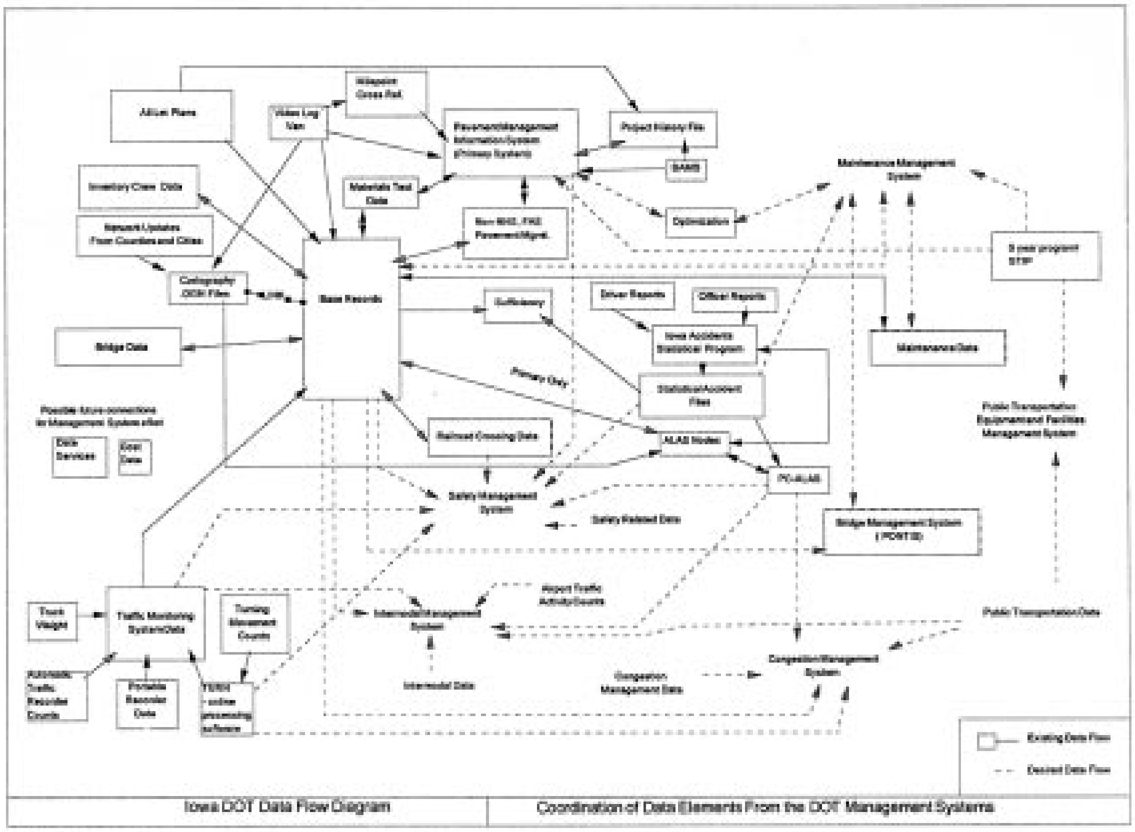

FIGURE 1 Iowa DOT data relationships.

problems within their organizations. Many DOTs, such as Maine, Texas, Virginia, Oregon, and Mississippi have data integration and GIS implementation projects currently underway.

Examples of Iowa DOT data include road, bridge, accident, and environmental data. These data are maintained in various mainframe and PC databases.

External Data Structures

In addition to the data collected and maintained by the DOT, several other data sources are available for use and are needed by the DOT users.

The Iowa Department of Natural Resources (DNR) has an established GIS with many useful layers of information. Examples of DNR data useful to the DOT include soil types, wetland information, location of underground storage tanks, and topography. The DNR's data are maintained in Environmental Systems Research Institute's (ESRI) PC Arc/Info software.

Several other organizations make data available to the public through Internet sites or by ordering the data on CDROM. The organizations that provide data useful to the DOT are numerous, but include the United States Geologic Survey, Census Bureau, National Wetlands Inventory, Bureau of Transportation Statistics, and several private organizations. These organizations may also provide aerial and satellite imagery that can be used by the DOT to enhance the accuracy of the existing data or to provide better information about the surface of the earth.

DATA INTEGRATION EFFORTS

Due to the large number of databases that store information at the DOT, it is impractical to consider translating all the existing data to one format. Additionally, numerous legacy systems were developed to maintain and query the data in those databases. Rewriting all the maintenance and query interfaces would be an insurmountable task for the DOT. For that reason, data warehousing concepts were identified as a means of integrating the data without having to merge all the data into one database management system for maintenance.

Data Warehouse

The data warehouse architecture allows data to reside in its current legacy system for maintenance, yet provides an environment where the data can be housed for better integration and used in GIS queries. Figure 2 illustrates the data warehouse integration process.

The data in the existing databases will be updated in the data warehouse based on user needs and maintenance cycles. For ex-

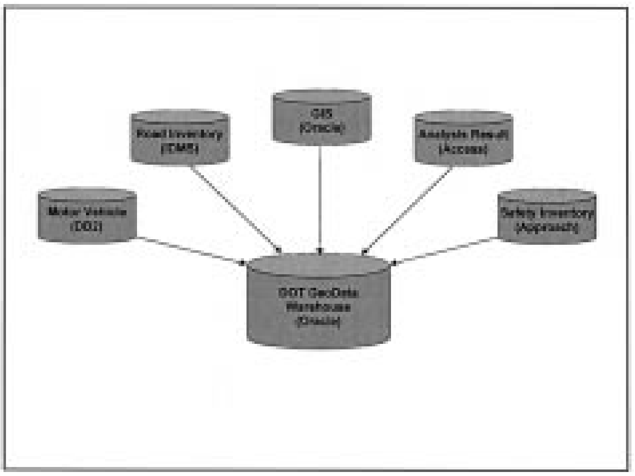

FIGURE 2 Data warehouse integration process.

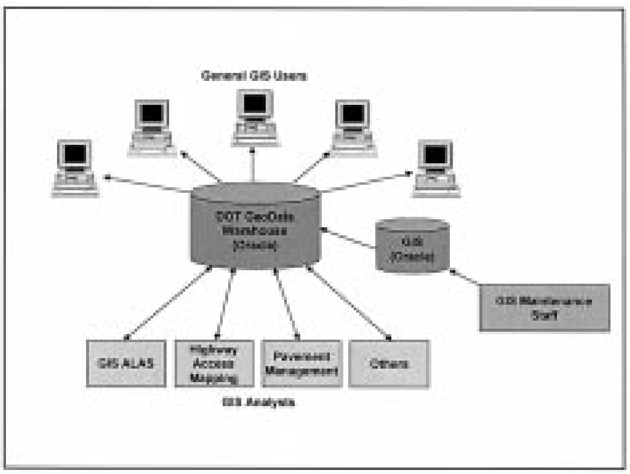

FIGURE 3 User access to the data warehouse.

ample, data updated once every year in December does not need to be updated in the warehouse on a nightly basis. Conversely, construction site status might need to be updated several times per day, if necessary for Intelligent Transportation System development. Real-time data replication processes can also be utilized if the need for dynamic data exists for certain systems (e.g., incident management applications). Storing the data in a warehouse creates a data storage environment that would accommodate all the GIS users at the DOT, whether they are highly skilled GIS analysts or occasional users of the data.

GIS Analysts (shown in Figure 3) have both read and write access to the data warehouse. They will use the data in the warehouse to perform their analysis, and may post the analysis results back to the warehouse for use by the rest of the DOT's users. These users will utilize whatever GIS software best suits their analysis needs.

General GIS Users will have read-only access to the data in the warehouse. These users will be provided a custom interface that allows them a method to find the data they need without having to be knowledgeable of the warehouse structure. This user group also has the ability to do analysis, but can not post their analysis results to the warehouse. The analysis results can be stored only in the user's personal storage spaces such as a local machine or personal network directory.

In order to minimize the customization needed to provide this functionality, the "off-the-shelf" software selected to access the warehouse has the ability to merge several GIS structures. This is a requirement due to the desire to read the warehouse in its "native state" without having to translate the internal and external data formats to a single GIS format, or transform all the maps to a single map projection. Intergraph's GeoMedia software was selected for the *General GIS Users* because it provides the capability to handle the multiple formats and map projections invisibly to the user. In subsequent development efforts this software will be more tightly intertwined with other systems, such as video logging and records management.

Metadata Development

When large amounts of data are provided in an environment such as a data warehouse, many questions arise about the source, accuracy and meaning of the data. To address those questions, metadata about the data in the warehouse will be maintained. Metadata development can be quite involved, but the DOT's initial efforts in metadata development will store the source, accuracy, date of collection and map projection of the data, in addition to a description of each field and the allowable values that may be found in that field. For example, the field may be called *light_cond* in the warehouse, but the metadata may describe that as, "The lighting conditions that existed at the time of the crash." The values stored in that field may be *1,2,3,..,9,* but the metadata will describe each value as, "1 - Sunlight, 2 – Twilight, 3 – Dawn, etc." This information will aid the users in their ability to build queries and understand the reliability of the results they get from a query. Future developments in the metadata will utilize the metadata standards that are defined by the Federal Geographic Data Committee.

IMPLEMENTATION ISSUES

In an effort as large as the departmental implementation of GIS and the development of a data warehouse, it is inevitable that implementation issues arise. Three key areas of concern are addressed here: training and resources, equipment acquisition, and institutionalization of the project.

Training and Resources

The training of the personnel involved in the use and support of the data warehouse and GIS is key to the success of the Statewide Coordinated GIS. It is often obvious that end users need training in the use of a new software tool, but as important is the training of the staff involved in support of the underlying architecture of the GIS and data warehouse. Since GIS is a relatively new tool for the DOT, it is necessary to train support staff in Oracle (the selected

FIGURE 4 Data warehouse interface.

relational database), the GIS, and the software used for customization of the GIS. The new training is often a difficult task to accomplish since GIS implementation is often added to the numerous responsibilities of the support staff. As a result, it is necessary for the DOT management to prioritize the need for GIS with the other efforts that are underway or needed. If different priorities are given to GIS in each Division of the DOT, Department-wide implementation becomes more difficult.

Equipment Acquisition

The acquisition of additional hardware and software to support the data warehouse and GIS is also key to the success of the Statewide Coordinated GIS. The project is on track for hardware and software needs due to effective planning, and the current network installation efforts. As users identify new data requirements, it is estimated that the storage capacity for the data warehouse may reach the five-hundred gigabyte range in the next two years. Thus, it is important to continuously keep management aware of the progress and the successes obtained to ensure future staff and financial support for the system.

Institutionalization

It continues to be important to consider the user's daily workflows when developing the system. The success of the Statewide Coordinated GIS will be measured mostly by its ability to aid the users in their business functions. For that reason, not only is it necessary to train the users in the use of the software, but in the use of the software to do their jobs more effectively. This is a difficult process because the trainer must be knowledgeable in not only new systems, but also in the users' workflows. For that reason the training process will utilize expertise from CTRE, who has been involved in the workflow analysis, and DOT personnel that have a basic understanding of the GIS technologies and data to help train additional DOT users. It is also important that the current procedures at the DOT be reviewed to include GIS where appropriate.

FUTURE PLANS

The current plans for the Statewide Coordinated GIS include the efforts to enhance the user interface of GeoMedia for the DOT's business, development of a single Department-wide Linear Referencing System (LRS), utilization of Global Positioning Systems (GPS) technologies, and integration of other systems being developed at the DOT.

Enhanced User Interface

While the interfaces provided by the GIS software are useful, it is more productive to display the data to the user in a format oriented toward the user's business. Improvements would include clear aliases for database field names and literal verbiage for the values in the field. For example, the new user interface would reformat the old display of "*county_num:85*" as "*County Name: Story.*" Another desirable enhancement would allow the user to enter values into two unrelated database tables and the custom software would automatically create the necessary spatial query to accommodate the request.

Linear Referencing Systems

The Statewide Coordinated GIS databases contain a large amount of linearly referenced data. To more efficiently utilize this data, a single LRS needs to be developed for the DOT that accommodates the many users and data collection efforts. The creation of this single LRS will alleviate the current method of "pre-segmenting" the DOT's Base Record road attributes based on the definition of a unique road segment. This would allow for simpler maintenance of the base road map and the road attribute data, and would allow users to utilize dynamic segmentation processes to query the data.

Utilization of Global Positioning Systems

GPS systems continue to become more accurate and require less expense and effort for a given accuracy. It is anticipated that this technology will allow the DOT's end users to add additional features to the base map for analysis and that the personnel maintaining the base map will be able to increase its overall accuracy. Standards and procedures need to be established for these processes to ensure the system integrity and to educate the users about the relative accuracy of the data they are using.

System Integration

Several information systems are currently under development or are being considered for development at the DOT. Three of these systems lend themselves to integration with the Statewide Coordinated GIS efforts: the Records Management System (RMS), the digital video logging system, and the Highway Closure and Restriction System (HCRS). Each of these systems utilizes attributes that are available in the Statewide Coordinated GIS. In an effort to minimize the work required of the end-user, these systems should be integrated to allow the end-user to process a query on only one of the systems and access data from the other systems. For example, a bridge engineer might query for the bridges over 20 years old with substandard deck inspections. After the results are displayed, it would be more efficient if the user could access the video log location of the bridge and the construction documents from the RMS without having to requery the other two systems again. This integration is more easily attained if the systems are designed with the integration in mind at the early stages.

CONCLUSIONS

It is evident that the integration of the data and the creation of the data warehouse will provide the end users with more data to use in their decision-making processes. The DOT continues to recognize the Statewide Coordinated GIS as an important part of improving the efficiency of data distribution at the DOT and additional resources are being devoted to the support of the project.

The current analysis processes depend too heavily on a small group of analysts that have access to the data. This system will allow the users to have more flexibility to apply “what-if” scenarios to the data and develop better solutions to problems they encounter.

The early stages of the project pointed out the need for a more tightly integrated maintenance and distribution process. The DOT continues to implement data warehousing and network technologies to make access to the data easier and provide “unprecedented” access to the Department’s data.

Roller Mountable Asphalt Pavement Quality Indicator

Edward J. Jaselskis, Hsiu Han, Lawrence Tan, and Jonas Grigas

Asphalt density measurements have been traditionally used as an indication of roadway pavement quality. These measurements, however, have not been available in real time to make in-process corrections to the paving operation since the existing techniques require on the order of minutes (nuclear density gauge) to hours (core samples) to produce accurate density measurements. Traditional ground penetrating radar (GPR) techniques could also have limitations due to the changing properties of the hot mix asphalt pavement while it is being compacted. This paper describes a novel approach to measure, for the first time, the density of asphalt in real time using a differential microwave signal approach. Two antennas, one in front of a roller and another behind it, will measure reflected signals from the asphalt; the change in signal characteristic from the front to back of the roller will show to the operator the optimal compaction and density of the pavement. This technique will minimize the need to quantify the hot mix asphalt properties that change during the compaction process. Field studies show that this approach has potential.

INTRODUCTION

In order to rebuild and pave existing highways that show signs of cracking and significant deterioration, it is important to effectively control the paving process. Several people are involved in producing a quality asphalt pavement (e.g., design mix specifier, hot mix plant operator, QA/QC inspector, asphalt laying operator, and roller compactor operator) — but it is the roller operator's skill that ultimately determines the final quality of the compacted mat where its density determines the effectiveness of compaction. Only a carefully planned rolling pattern gives the uniformity and desired density. An under compacted asphalt mat is permeable to air and water which shortens the pavement life, while unnecessary extra passes may lead to over compaction and a reduction in air void content that can cause significant permanent deformation (*1*). The only way to know the most efficient rolling pattern is for the roller operator to monitor the asphalt density in real time.

Existing techniques have drawbacks. For example, the nuclear density gauge, which uses a gamma ray back scattering technique, requires proper calibration and several minutes to obtain a density measurement making it difficult to implement in real time on a continuous paving operation. Ground penetrating radar units can operate in a continuous fashion but are more expensive and the correlation between signal reflection and density on a hot asphalt mat have not been precisely determined. This paper presents a new type of asphalt pavement density for a roller operator's real time control of paving density that will be simpler, cheaper, and safer than existing density measurement techniques and provide a continuous assessment of pavement quality.

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DESCRIPTION

This asphalt pavement density indicator will be developed based on the continuous comparison of two microwave signals reflected by the pavement, one in front of the roller and one behind it (Figure 1). Horn antennas will be used to transmit and receive the microwave signals which will be coupled via a microwave bridge in order to continuously monitor the variability of the two reflected signals. In previous field studies, the research team found a decreasing trend in microwave signal variability as the density increases. Once the pavement reaches the optimal compaction level, the variance suddenly jumps. It is this information that is being used to determine that the optimal compaction range has been reached. By using this "differential approach" on the two microwave reflections of the proper frequency, the influence of a large number of parameters involved on which reflectivity of a pavement depends (e.g., permittivity and loss) will be minimized.

This idea operates by transmitting an amplitude modulated microwave signal from a low-power microwave oscillator via a directional coupler and circulator to the antenna horn pointed perpendicular to the asphalt pavement (Figure 2). This setup with horn antennas in front and behind the roller pick up the reflected

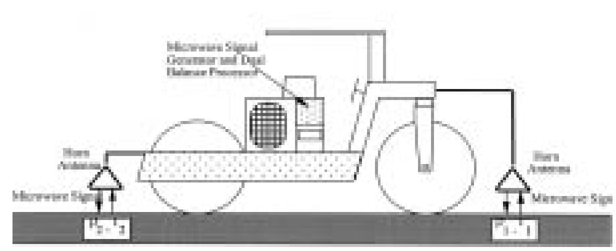

FIGURE 1 Real time microwave pavement sensor attached to a roller.

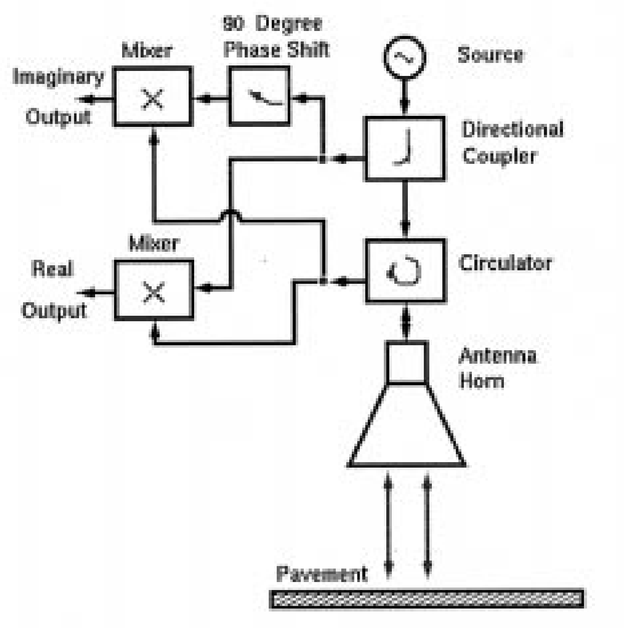

FIGURE 2 Schematic for real time microwave pavement sensor.

signals from the pavement, and transmit a real and imaginary output from each horn to a computer for processing.

PRELIMINARY FIELD TESTS

Field tests were conducted on two separate occasions to determine the feasibility of this idea. Both tests were funded by the Center for Advanced Technology and Development (CATD) at Iowa State University. The goal of this field testing was to be able to detect differences in microwave reflected signals for different compaction levels on hot asphalt. In the first field test conducted during May 1996, eleven different microwave signal frequencies were reflected off of a hot asphalt surface of varying density. Results from this test showed that a relationship was found between the reflected microwave signal and density. The variability of the reflected microwave signal was found to decrease as the density increased until the point of optimal compaction. At this point, there is a sharp increase in the variance of the reflected microwave signal. Results from the first set of field tests were verified during a second round of tests conducted in the fall of the same year.

PROTOTYPE DEVELOPMENT

The microwave horn antennas are mounted on carts (antenna platforms) attached to the breakdown roller with cantilevered arms. Each cart is equipped with a vibration dampening system to reduce the vibration from the roller. The set up is shown in Figure 3.

Many of the design constraints established for this project were based on what the research team found to work during the preliminary field testing stage. The design constraints used for the design of the prototype are as follows:

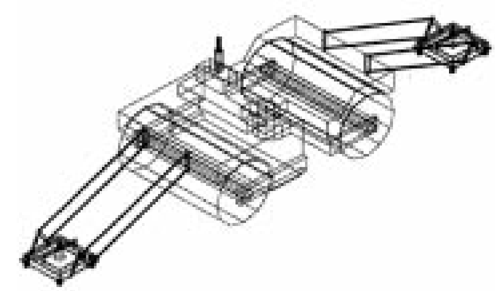

FIGURE 3 Equipment set-up.

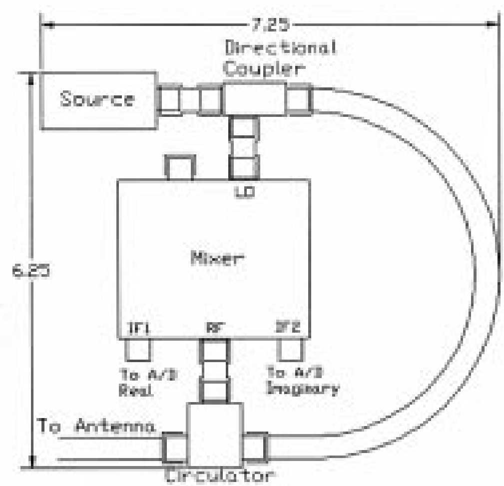

FIGURE 4 Microwave circuit (dimensions in inches).

- Antenna platform maximum weight = 30 lbs.
- Height of microwave antenna to be held constant at 15 cm.
- Microwave antenna should be 2 to 3m away from breakdown roller.
- Density readings to be taken at fixed distances of approximately 2 cm.

The prototype design consists of four components, listed below.

Microwave Signal Circuit

The final circuit design is shown in Figure 4.

Pneumatic System

Included in the design is a pneumatic system which will be used to remove water from the asphalt surface during compaction. Water used to lubricate the drums may pose a problem by distorting the return signal. It was decided to use a manual pneumatic spray approach for the initial tests whereby an air compressor and hose would be used to blow the water from the surface. In later versions, a spray attachments could be mounted on the carts. The research

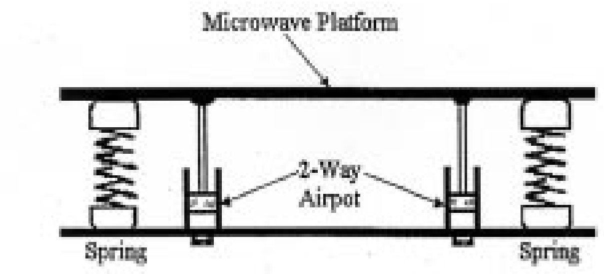

FIGURE 5 Mass-spring damper system.

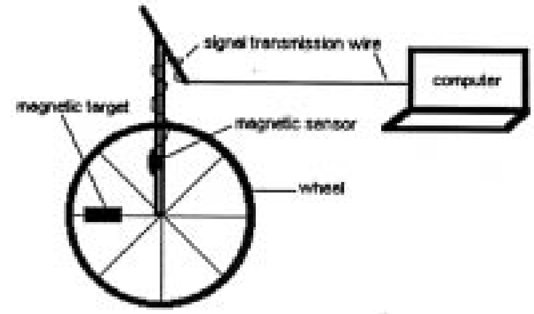

FIGURE 6 Magnetic field density method.

team conducted field tests that demonstrated that 100 psi is a suitable air pressure to remove water from the asphalt surface without damage.

Antenna Platform

The antenna platform has been designed using a two frame approach with vibration isolation devices connecting both frames. The upper frame holds the antenna and microwave equipment, while the lower platform rolls on the surface with special high temperature wheels and is connected to the roller. Vertical vibration dampening is achieved using a combination of springs and dashpots while a foam liner is used to dampen lateral vibration. A "Mass-Spring Damper System" as shown in Figure 5 provides a simple and low cost vibration control. Theoretical calculations as well as simulations were performed to determine the dampening coefficient needed on the springs and dashpots.

Signal Sampling Regulator

The signal sampling regulator is needed to trigger density samples measurements at discrete intervals. The design incorporates a "magnetic field density" approach as shown in Figure 6. The wheel, in contact with the rear roller wheel of the compactor, triggers a density reading each time the metal target is detected.

PROTOTYPE FIELD TEST RESULTS

The research team plans to conduct field tests on the prototype during the summer, 1998. These results are not available at this point in time but will be provided at the conference.

ANTICIPATED IMPACT TO PRACTICE

The novel approach described in this paper would have numerous applications. Asphalt paving contractors could deploy this device on their roller compactors and provide their operators with a better sense when optimal compaction in achieved. Transportation agencies will benefit from higher quality pavement and contractors will experience fewer penalties. It is anticipated that this asphalt pavement indicator will be simpler, cheaper, and safer than existing density measurement techniques and provide a continuous assessment of pavement quality.

CONCLUSIONS

This paper has discussed a novel approach to determine asphalt pavement density during the compaction process. This approach simultaneously compares the variability of microwave signals from the front and back of the breakdown roller. It was found in field tests that the variance of the microwave signal decreases as density increases and then sharply increases in the optimal compaction range. Results from the prototype field tests will be provided during the conference as they were not available at this time. It is anticipated that this method will become a widely used approach for determining asphalt pavement density.

ACKNOWLEDGMENTS

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Development of End-Result and Performance-Related Specifications for Asphalt Pavement Construction in Illinois

WILLIAM G. BUTTLAR AND MICHAEL HARRELL

Illinois has been engaged in research over the past decade to develop quality control/quality assurance (QC/QA) specifications that maximize pavement construction quality, while minimizing the amount of agency testing required to execute the system. This paper describes recent efforts to develop and implement end-result and performance-related specifications (ERS/PRS) in Illinois. In the summer of 1996, a pilot study was conducted during the construction of an eight-inch overlay placed over rubblized concrete on Interstate 57 near Edgewood, Illinois, to collect data to support the development of ERS/PRS. A high sampling and testing frequency was followed to aid in the determination of typical variances and to help determine the minimum amount of samples required for QC and QA in the future. Fundamental properties of plant produced mixtures were also measured, including: fatigue, permanent deformation, resilient modulus, and tensile strength. Furthermore, a unique "properties map" was developed, where all measured quality characteristics for each sublot of material were mapped by station and by pavement lift. This map will facilitate the development of future linkages between quality characteristics, engineering properties, and measured distresses (rutting, cracking, moisture damage, etc.), which is essential for the development of a comprehensive PRS. Ongoing and planned research efforts in this area are also described, including an upcoming project involving the development and use of percent-within-limits (PWL) based pay factors for field density control, which utilize contractor test results, and the evaluation of a rapid, non-nuclear pavement density gauge. Key words: quality control, quality assurance, performance-related specifications, end-result specifications, asphalt pavement.

INTRODUCTION

Quality assurance specifications are an important component of an organization's commitment to overall quality management, and consist of several activities, including: process control, acceptance, and sometimes, independent assurance of a product. Specifications for the construction of asphalt pavements can generally be classified into method-related specifications (MRS), end-result specifications (ERS), performance-related specifications (PRS), or combinations thereof (Figure 1). Method specifications give a set of procedures, or "recipe," that if followed by the contractor, will result in full payment for the constructed facility. This places a great deal of responsibility and testing burden on the agency rather than the contractor. End-result and performance-related specifications, as their names imply, require a contractor to achieve specified as-produced or as-constructed quality levels, which are ideally linked to the attainment of good future performance. These types of specifications shift most or all of the responsibility for producing a high quality product to the contractor, and should ideally offer the contractor complete freedom in the methods used to arrive at these quality levels.

Performance-related specifications are difficult to develop, but offer the ultimate means of compensation for a delivered product: variable payment (incentive/disincentive) can be assigned based upon expected performance and increase/reduction in life cycle value of the product relative to the design life cycle worth. As illustrated in Figure 2 (Shook [*1*]), the development of a PRS involves having links between quality characteristics (asphalt content, gradation, density, etc.), engineering properties (modulus, tensile strength, etc.), and performance (distresses, serviceability level). Obviously, the development of these links, particularly the secondary relationships between quality characteristics and engineering properties, can be very difficult since they are material dependent and many complicated interactions exist. Furthermore, one must also consider variability in measuring equipment, operator errors, and typical material variances during production (resolution of the inputs) when deciding on the required accuracy and therefore complexity of the secondary relationship models. As a result, it is most practical to develop a specification with the combination of ERS and PRS elements commensurate with existing technologies, local materials, and test equipment.

Overview

This goal of this paper is to:

1. Describe the philosophy for moving towards ERS and PRS in Illinois
2. Describe the Edgewood 1996 Pilot Project Conducted to Support Development of ERS/PRS in Illinois
3. Outline Upcoming Projects and New Technologies Being Evaluated for ERS/PRS Development

PHILOSOPHY FOR MOVING TOWARDS PERFORMANCE-RELATED SPECIFICATIONS

Because existing primary and secondary relationships preclude the direct movement to PRS, the following staged approach has been developed for gradual movement from the existing MRS/ERS sys-

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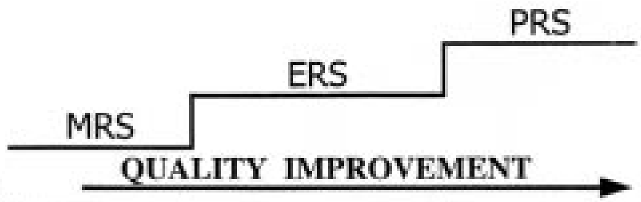

Characteristic			
Specification Variables	Indirectly or directly related to performance	Indirectly or directly related to performance	Only those directly related to performance
Raw Materials	Agency	Agency	Contractor
Mix Design	Agency	Contractor	Contractor
Process Control	Agency/Contractor	Contractor	Contractor
Lots	No	Yes, uniform construction run	Yes, same for all characteristics
Sampling	"Representative" or arbitrary, frequency based on opinion	Random locations, frequency based on sampling theory	Random locations, frequency based on sampling theory
Pay Adjustments	Usually not used	Often used. Based on judgement.[1]	Required. Based on performance and LCC analysis.[2]
Type of Acceptance Tests	May or may not be related to performance	May or may not be related to performance	Directly related to performance (prediction models)
Material Acceptance	By agency, often from a single sample	By agency, from samples obtained prior to placement or in-situ	By agency, from samples obtained in-situ
Equipment	Agency specifies	Agency allows wider range of equipment usage	Few proscriptions

[1]Considers variation in quality characteristics directly. Pay adjustment factors for individual quality characteristics, must combine into one pay factor.

[2]Considers variation in quality characteristics directly. Pay adjustment factors for quality of lot, not individual characteristics.

FIGURE 1 Evolution and advantages of end-result and performance-related specifications (after Patel [5]).

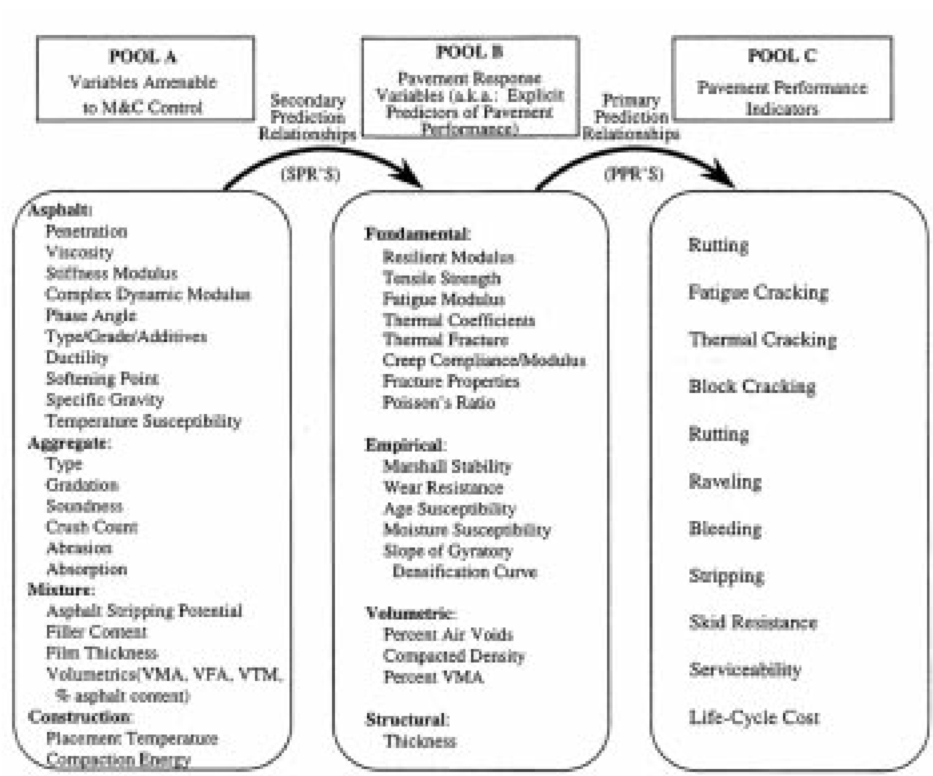

FIGURE 2 Connection among variables associated with an asphalt concrete pavement surface PRS (after Shook [1]).

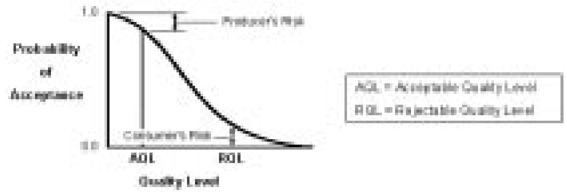

FIGURE 3 Operating characteristics curves (after Afferton et al. [*3*]).

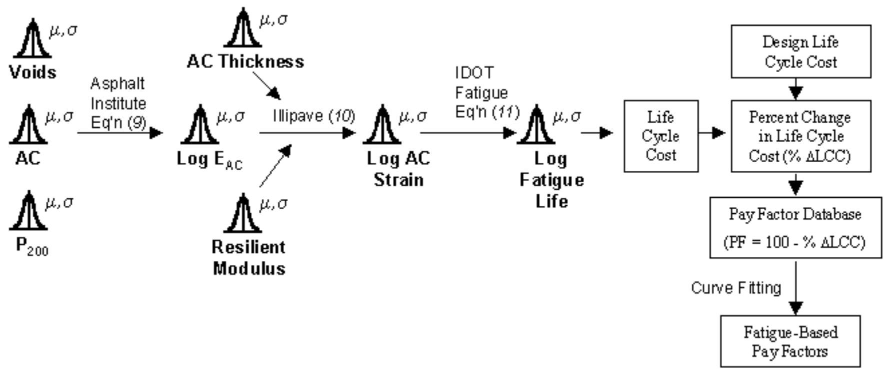

FIGURE 4 Framework of a performance-related pay factor for fatigue (after Patel [*5*]).

tem to a predominantly end-result specification. As primary and secondary relationships become available, the ERS will then be modified to contain as many PRS elements as possible. The key steps in this development have been identified as:

1. *Make an initial move to statistical QC/QA.* The AASHTO Quality Level Analysis (*2*), involving Percent-Within-Limits (PWL) based pay factors is now being used as a variable payment scheme for pavement thickness in Illinois. A demonstration project is also being developed to evaluate a PWL-based pay factor for field density for the summer of 1998, which is described later in this paper.
2. *Develop a comprehensive ERS to consider all relevant quality characteristics.* Develop pay factors for other quality characteristics that are likely to relate to performance (asphalt content, gradation, etc.). Use QC/QA and performance data from past projects along with engineering experience to establish rational specification limits and pay factors. Use operating characteristics curves (Afferton et al. [*3*]) to evaluate and adjust pay factors so that risks are balanced fairly between the contractor and agency (Figure 3).
3. *Monitor and foster development of primary and secondary prediction relationships.* A pilot field study was initiated in 1996 at Edgewood, IL (Buttlar et al. [*4*]), which will provide a comprehensive database for the development of these relationships. This project is described in detail later in this paper. The progress of other related studies, such as WesTrack and National Cooperative Highway Research Program (NCHRP) project 9-15, will be monitored and relevant findings will be incorporated to the extent possible.
4. *Develop performance-related pay factors.* A system was developed in IDOT ICHRP IHR-425 by Patel (*5*) to generate pay factors for full depth asphalt pavements based upon fatigue cracking (figure 4). The variable pay factor developed was based upon the change in life cycle costs associated with departures from design means and standard deviations of selected quality characteristics. The effects of these deviations on engineering properties were considered simultaneously using the Monte Carlo simulation model, which were in turn used to predict pavement response and fatigue life. However, additional pay factors must still be developed for other distresses, such as rutting, thermal cracking and moisture damage, and for other flexible pavement systems (conventional and overlays).
5. *Compare performance-related pay factors with ERS pay factors, which were developed based upon experience.* Choose the more conservative pay factor, if applicable, or develop a new pay factor that combines the most conservative elements of each of the two.
6. *Periodically repeat steps 3, 4, and 5 to move from ERS to PRS.* The move from ERS to PRS will take considerable time, and there will always be room to reevaluate and improve the accuracy and practicality of the system components. Therefore, periodic cycling through steps 3 through 5 will be necessary.

The following section will describe a pilot field study conducted in the summer of 1996 at Edgewood, Illinois, which was established to generate data to support the development of primary and

TABLE 1 Comparison of Pilot Study QC Test Program and IDOT's Superpave Demo Project QC Plan

Parameter	Frequency of Tests: IDOT Superpave QC	Frequency of Tests: Pilot Study	Test Method
Mixture Control Testing			
Aggregate Gradation Hot bins for batch and continuous plants. Individual cold-feeds or combined belt-feed for drier-drum plants (% passing 12.5-mm [1/2-in], 4.75-mm [No. 4], 2.36 mm [No. 8], 600-µm [No. 30], 75-µ [No. 200] sieves)	Two dry gradations per production day Washed gradations performed on every eighth test	Four dry gradations per productions day Four washed gradations per production day	Illinois Procedure (See currrent Department Aggregate Technician Course workbook.)
Asphalt Content by Ignition Oven	Two per production day	Four per production day	Illinois Modified AASHTO TP 53-pending
Air Voids Bulk Specific Gravity of Gyratory Sample Bulk Specific Gravity of Marshall Hammer Maximum Specific Gravity of Mixture	1 per half day of production for first 2 days and 1 per day thereafter (first sample of the day)	Four per production day	*Per the Department's "Superpave Field Control Course" Illinois-Modified AASHTO T 166 Illinois-Modified AASHTO T 209
Field VMA	1 per half day of production for first 2 days and 1 per day thereafter (first sample of the day	Four per production day	Per the Department's "Superpave Field Control Course"
Field Control Testing			
Field Density	800-m Lot Size: IDOT: 5 Transverse Measurements at One Location within Lot	800-m Lot Size: Stratified Random Measurements in Each of 4 Equal, 200-m Sublots (Random Longitudinal and Transverse Position within Sublot)	Per the Departments Illinois-Modified ASTM D 2950, Standard Test Method for Determination of Density of Bituminous Concrete In-Place by Nuclear Method"

*For each of the four daily sublot samples, two gyratory tests were performed: one to N_{des} gyrations and one to N_{max} gyrations.

secondary performance relationships for Illinois materials and construction (step 3 above).

THE 1996 PILOT FIELD STUDY AT EDGEWOOD, ILLINOIS

A pilot field study to collect data in support of ERS/PRS development was conducted in concert with rehabilitation of Interstate 57, in south-central Illinois, during the summer of 1996 (*4*). Some of the pertinent project features include:

- Location: IDOT District 7, about 12 miles south of the I-70 crossing at Effingham
- Project length: 4.3 miles, both northbound lanes of I-57
- Mainline paving: July 26-August 20, 1996; contractor: Howell Paving Co.
- 8-inch HMA overlay, 27,000 tons; batch plant: Effingham, IL., 2000 tons/day
- Overlay placed on 8-inches of rubblized CRCP (Antigo Construction, multiple-head breaker)

The goals of the pilot study, some of which are ongoing, were to:

- Foster end-result specification (ERS) development for ACP construction in Illinois, and explore sampling and testing schemes

suitable for statistically-based QC/QA, including: 1) clear LOT and SUBLOT definitions for as-produced and as-constructed asphalt mixtures; 2) random sampling protocols, and; 3) evaluation of a modified field density sampling procedure, which was more suitable for statistically-based QC/QA than the existing method
- Provide data for the evaluation of percent-within-limits (PWL) based pay factor concepts, including; 1) establishing baseline variability for new test procedures; 2) determining minimum sample sizes, and; 3) evaluating suitability of gyratory compactor and ignition oven for QC
- Establish possible links between QC measurements/ AC engineering properties/ and performance, by: 1) measuring fundamental properties on LOT samples; 2) monitoring pavement profile, structural layer properties, pavement distresses, and overall serviceability, and; 3) correlating quality characteristics, engineering properties, and performance for given LOTS of material.

QC/QA Testing Program at Edgewood

Part of the research effort was to study typical variances for new test devices and to determine optimum test frequencies for QC/QA in general. Therefore, additional tests were supplemented with the QC testing specified in IDOT's Special Provision for QC/QA of Class I Bituminous Concrete Mixtures (*6*), as shown in Table 1. In accordance with IDOT's QC testing plan for Superpave Demo Projects, air voids were monitored at the plant with a Troxler Superpave Gyratory Compactor. Asphalt content was obtained in conjunction using an NCAT binder ignition oven, and aggregate gradation was controlled through dry sieve analysis of ignition oven residue. Construction control testing was limited to the determination of field density using the nuclear gage. An extended testing program was implemented at Edgewood, which consisted of diametral creep tests, split tensile tests, resilient modulus tests, beam fatigue tests, and triaxial testing. Also conducted were Georgia loaded wheel tests and Superpave indirect tensile tests (IDT).

Sampling and Lot Definitions

Central to the development of a statistically-based QC/QA ERS/ PRS specification is having a sound random materials sampling and testing procedure. For the control of as-produced quality characteristics, A LOT was defined as one day of mixture production. Each LOT was further subdivided into four SUBLOTS. The exact sampling time for each sublot was found by multiplying separate randomly generated numbers by the length of time of each sublot and adding the resulting time to the beginning of each sublot.

For the control of as-constructed quality characteristics, a LOT was defined as 800 meters of paved roadway, which was then subdivided into four 200-meter SUBLOTS. Field density readings were collected using a nuclear density gage, following two sampling methods (Figure 5). The first method was IDOT's existing density sampling scheme consisting of five transversely-aligned density measurements taken at one randomly determined location in each designated LOT. Samples were taken at 0.61-m (2-ft) intervals across each lane slightly staggered from perfect alignment perpendicular to the direction of traffic. The second method consisted of a single density measurement taken at a random location in both the transverse and longitudinal directions within each SUBLOT (200-m section).

Properties Maps

Figure 6 is a "property map" of the northbound I-57 driving lane. Crosshatched sections represent a LOT of material with the LOT number and thickness and type of lift placed in the top left corner. Within each of those sections are four subsections representing the four SUBLOTS associated with each LOT. The locations of the SUBLOTS were determined using quantities found on daily plant reports, lift thicknesses, mixture density, and field notes indicating times and locations of the contractorís density QC measurements. The SUBLOTS are numbered sequentially in the order they were produced. Dashed lines denote the estimated SUBLOTS boundaries. Percent-Within-Limits (PWL) were computed for asphalt content, percent passing the #200 (0.075-mm) sieve, and plant-measured air voids (*4*). The results of these calculations are shown for each LOT of material.

Selected Results from Edgewood Pilot Study

A comprehensive description of the Edgewood project findings to date can be found in Buttlar et al. (*4*). Two of the most prominent findings from the study related to as-produced and as-constructed mixture properties are presented in Figures 7 and 8. Figure 7 illustrates that the modified density scheme (Figure 5) appears to give a more representative assessment of overall LOT density, as denoted by the lower standard deviation of the moving average and hence flatter curve. Figure 8 illustrates the differences observed between air voids back-calculated from N_{max} to N_{design} gyrations in the Superpave gyratory compactor to air voids measured directly at N_{design}. Errors in predicted air voids at the design compactive effort of over 1.5 percent were measured during mixture production.

Alternate procedures were developed by Buttlar et al. (*4*) and Vavrik and Carpenter (*5*) to arrive at more accurate estimates of air voids at N_{design}. Vavrik and Carpenter also suggested the evaluation of mixture compaction characteristics based upon the "locking point," or the point during compaction at which the mixture exhibits a marked increase in resistance to further densification. The locking point has been found to be related to compaction tendencies in the field and its relationship to field performance is currently under investigation. Compacting a mixture past the locking point generally results in aggregate degradation that is not representative of field compaction, and thus, the benefit of compacting mixtures to N_{max} is currently being reevaluated on a national scale.

Future Monitoring of Edgewood Project

Performance monitoring of the Edgewood Pilot Project will consist of periodic (at least annual) measurement of rut depth, roughness, effective layer moduli (from falling weight deflectometer), distress mapping, and overall serviceability assessment. To develop primary and secondary links, it was necessary to take several measurements in each segment of pavement corresponding to various LOT combinations through the depth of the pavement. The vertical arrangement of LOTS in different layers is more aligned in the driving lane (Figure 6) than the passing lane (not shown). A

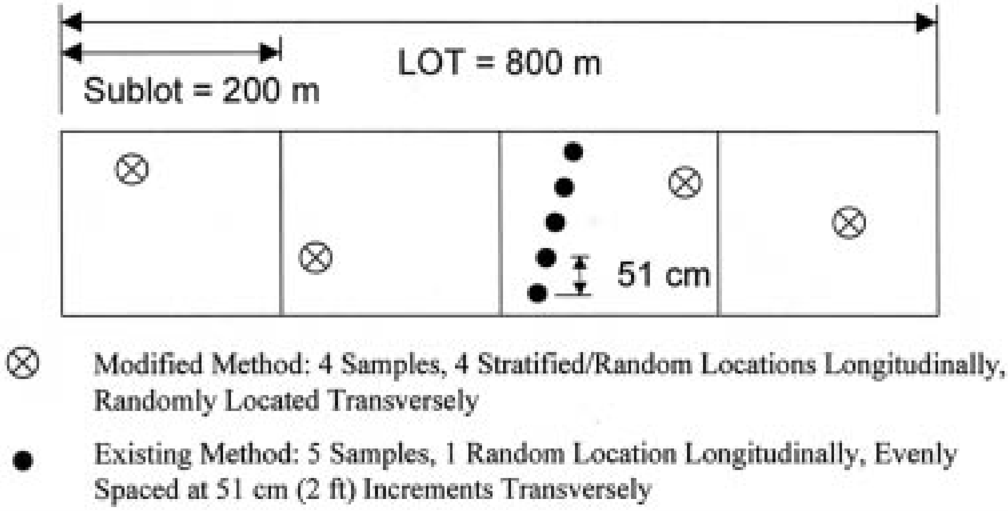

FIGURE 5 Comparison of density determination procedures: existing method versus modified method (from Buttlar et al. [*4*]).

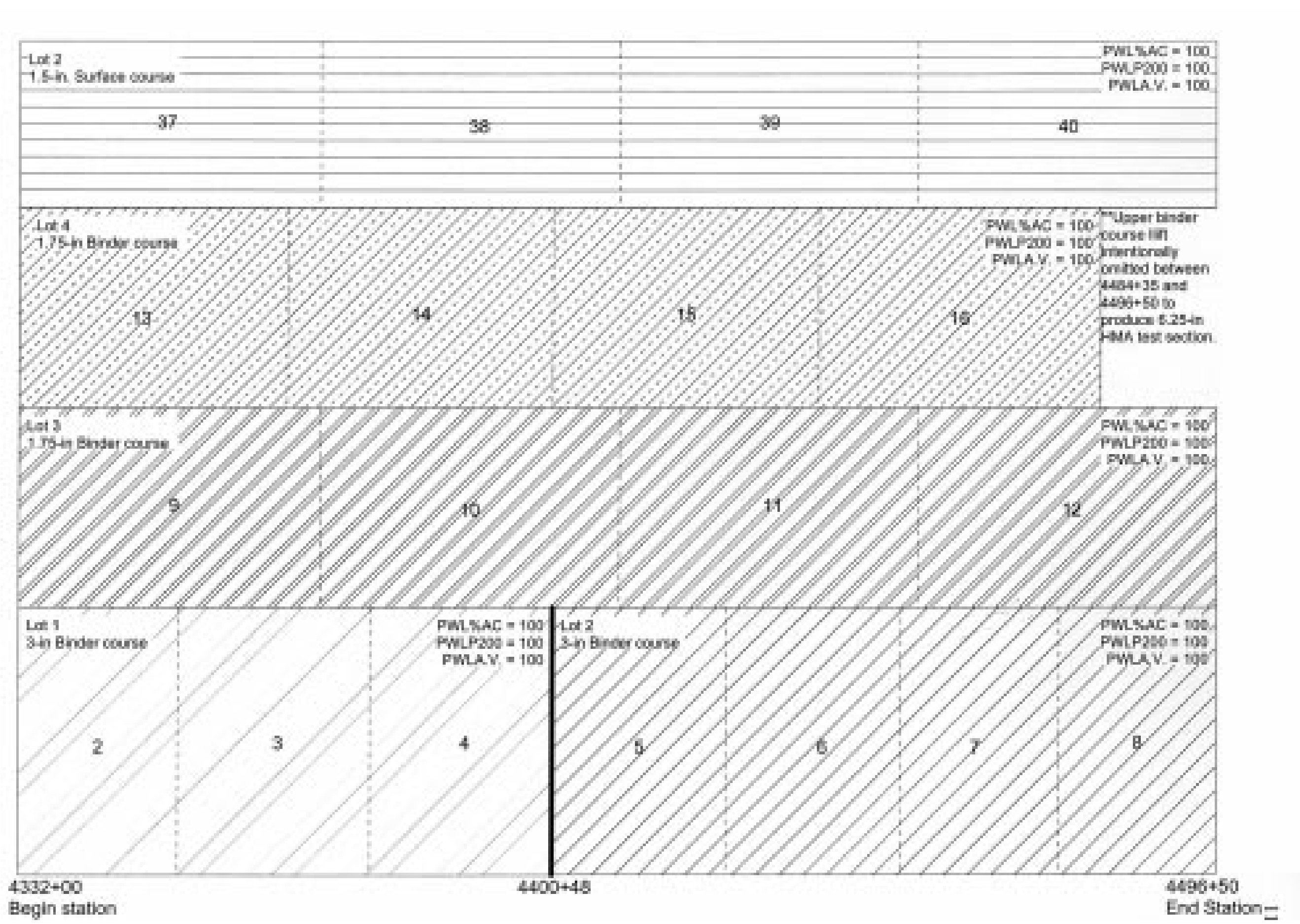

FIGURE 6 Property map for I-57 Edgewood pilot study: driving lane.

spacing of 200 feet between measurements was found to capture all possible LOT and most SUBLOT combinations.

Thus, the Edgewood pilot field study is one of the most comprehensive projects to date having plant, field, and engineering properties, a unique "properties map," and regular monitoring of performance. The study will provide valuable inputs for the development of ERS/PRS for asphalt pavement construction in Illinois.

UPCOMING PROJECTS AND NEW TECHNOLOGIES

A demonstration project is being developed for the summer of 1998 to:

- Showcase new ERS concepts for asphalt pavement construction in Illinois, including a PWL-based pay factor system
- Implement actual pay factors (PWL-based incentives/disincentives) to fully bring to light the practical implications of developing and implementing such as system

It was decided that actual pay adjustments would be limited to a single quality characteristic at first, namely field density. Other quality characteristics and pay factors will be monitored and tracked on a more casual basis to assess their feasibility for future revisions of the specification. It was felt that "real-time" information regarding PWL and pay factors should be readily available to contractors and agency personnel in the field. While the computation of PWL is not difficult, neither is it straightforward; it requires statistical tables and computations of standard deviations. This procedure is too cumbersome to be carried out by the quality inspector in the field, so the procedure was programmed into a Texas Instruments TI-85 calculator. Non-central t-distribution tables (*2*) had to be fit with polynomial prediction equations, since programming the calculator to interpolate the large tables would have been extremely cumbersome.

Other elements of the new ERS being developed for the demonstration project include:

- Use of contractor data as part of quality assurance and PWL-based pay factors, since the minimization of agency testing burden is a top priority in Illinois and most other agencies
- Dispute resolution and retest provisions, as these are perhaps as or more critical than the end-result property specification itself

Also to be investigated in the demonstration projects will be the following cutting-edge concepts and equipment:

- Monitoring of the "locking point" of the asphalt mixture during production.
- A triaxial-based QC/QA machine, developed by Industrial Process Controls (IPC), will be used to obtain fundamental properties of as-produced mixtures related to rutting and fatigue potential.
- The Superpave Indirect Tensile Test (IDT), developed by Roque and Buttlar (*8*) during the Strategic Highway Research Program, will be used to evaluate the change in predicted thermal cracking performance with respect to changes in quality characteristics measured during production.
- A non-nuclear pavement density gage, manufactured by TransTech, Inc., will be run parallel to the nuclear gage and compared to densities as determined by coring and lab testing. The gage is measures the dielectric constant of the mixture, which is correlated to pavement density through a simple calibration procedure. The principle advantages of the device is the speed of measurement (a few seconds per reading) and the elimination of nuclear licensing and detailed training needs.

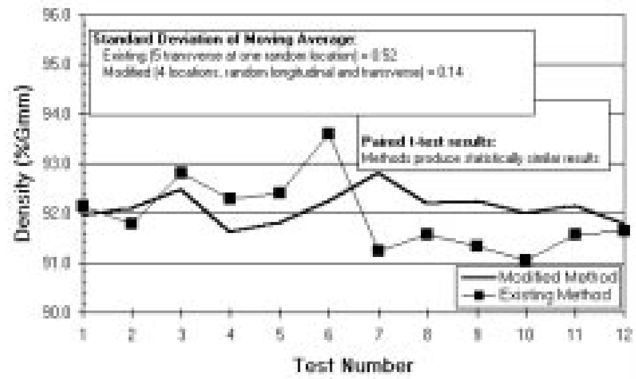

FIGURE 7 Comparison of density results at Edgewood: existing and modified methods.

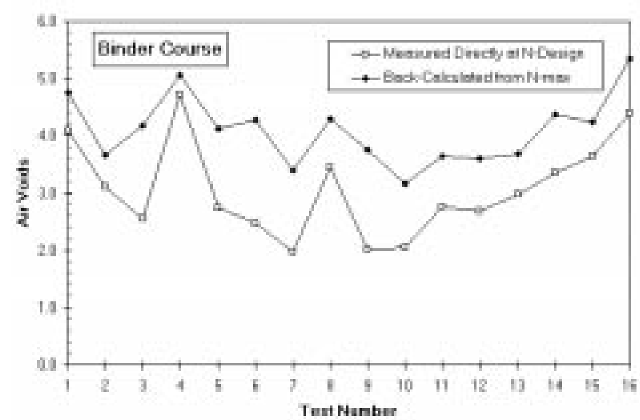

FIGURE 8 Difference between back-calculated voids and voids measured directly at N_{design}.

SUMMARY

The key points of this paper can be summarized as follows:

1. Performance-related specifications can be challenging to develop, but offer the ultimate means of compensation for a delivered product.
2. It is most practical to develop a specification with the combination of ERS and PRS elements commensurate with existing technologies, local materials, and test equipment. A staged approach for gradual transition to performance related specifications for asphalt pavement construction in Illinois was presented.
3. The Edgewood pilot field study is one of the most comprehensive projects to date having plant, field, and engineering properties, a unique "properties map," and regular monitoring of per-

formance. The study will provide valuable inputs for the development of ERS/PRS for asphalt pavement construction in Illinois.

ACKNOWLEDGMENT/DISCLAIMER

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The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Illinois Department of Transportation, nor does it constitute a standard, specification, or regulation.

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Incorporating Innovative Technologies into Traditional Pavement Management Activities

KATHRYN A. ZIMMERMAN

In recent years, computers have revolutionized the way in which business is conducted. In highway agencies, pavement management systems are now used for identifying, prioritizing, and selecting pavement maintenance and rehabilitation projects. The results of the pavement management analysis are traditionally reported to agency management, government officials, and the public in the form of reports, newsletters, and graphics. Today's interactive CD-ROM programs are greatly affecting the way some of these traditional pavement management activities are conducted, especially in the areas of distress identification training (for determining pavement conditions) and in reporting pavement-related information. This paper discusses some of the applications of this type of technology in a state highway agency, a commercial airport, and a state division of aviation. The first example illustrates the use of interactive training tools to develop a distress identification program for pavement condition raters. The program features photographs of distresses at different severity levels, audio and video clips explaining the inspection procedure, and an assessment program to evaluate the rater's ability to distinguish between distresses. The paper also illustrates using similar technology to distribute pavement-related information from a pavement management system. Using an interactive data display format, these airport agencies provide photographs of typical distresses and a summary of the condition information from the pavement management system for airport consultants and agency officials. This paper discusses the use of these new technologies and illustrates their effectiveness in the area of pavement management. Key words: pavement management, distress identification training, data access, institutional issues, technology transfer.

INTRODUCTION

Pavement management systems have been used by transportation agencies responsible for the maintenance and rehabilitation of their pavement network for almost twenty years. As pavement management evolved, a primary focus of pavement management activities concentrated on two primary areas: (1) the development of computerized modeling tools to predict pavement performance and identify and recommend rehabilitation needs, and (2) the implementation of these tools in transportation agencies. However, there are indications that many of the agencies with sophisticated pavement management tools are not utilizing these tools to the fullest extent. Agencies frequently cite Institutional Issues as the principal factors affecting the use of pavement management information in the project and treatment selection process.

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INSTITUTIONAL ISSUES IN PAVEMENT MANAGEMENT

Institutional Issues have become a focus in pavement management since the early 1990s, when the topic was first discussed as part of the Federal Highway Administration's (FHWA's) Advanced Course in Pavement Management (*1*). Many feel that these issues became a focal point because of the level of sophistication that had been achieved in the technical aspects of the programs. Prior to 1990, most conferences and developments in the pavement management area focused on the more traditional technical areas.

After closer investigation, it appears that there are a number of different reasons for the failure to use pavement management information in transportation agencies. These reasons can be compiled into three primary areas: lack of trained personnel, fear of change, and turf protection. Each of these areas is discussed in more detail in the following sections.

Lack of Trained Personnel

For the most part, pavement management is a new concept that has not been a part of a traditional civil engineering degree program until recent years. Even today, pavement management is taught in relatively few colleges and universities, when considering the total number of civil engineers graduating each year and entering the workplace. Because of the limited number of trained individuals in pavement management, pavement management departments frequently included only one or two individuals with a good understanding of the concepts and decision-making concept. Additionally, the individuals tasked with the implementation of a pavement management system were often assigned these duties as additional responsibilities in addition to the normal day-to-day activities routinely being performed. As a result, the individual typically had little time to spend on the maintenance and upkeep of the PMS once it was implemented. Further, the individual had little time to train anyone to assist him with pavement management so no new staff were trained in the operation of the system.

Complicating this issue further was the fact that for many years, pavement management engineers were promoted within transportation departments at an unusually accelerated pace. Whatever the reasons for these promotions, they had a dramatic impact on moving the experienced pavement management engineers away from the day-to-day operation of the PMS, leaving a void in many pavement management departments.

Although the amount of training in pavement management has expanded in recent years, the problem of keeping experienced pavement management practitioners in pavement management departments continues. Coupled with the continual advancements in pavement technology, the need for training in all aspects of pavement management continues to be a high priority.

Fear of Change

For many individuals, changes in procedures or processes are not welcome because of a natural hesitation to accept new ways of doing things. The implementation of a PMS frequently involves changes in some aspects of project and treatment selection or identification. These individuals may worry that the changes will expose inefficiencies or errors in the processes that had been used for years; somehow implying that they hadn't been doing a very good job in the past. This can be especially true if the pavement management system is perceived as being very complex and the automated project selection process is not well understood. Individuals naturally wonder how the changes will affect their role within the organization and worry that they may lose power, responsibility, or importance. Because of this, efforts to integrate the pavement management system may be blocked.

Turf Protection

Fear can often lead to another personal challenge: turf protection. Turf protection occurs when individuals feel that their importance within the organization is being threatened, so they become protective of their "turf." Turf protection has a negative impact because it tends to interfere with the formal and informal lines of authority and communication inside an agency. Individuals concerned with "protecting their turf" may view the pavement manager as gaining power due to the large amount of information available from the pavement management system and the pavement manager's interaction with others to communicate pavement management recommendations. This may worry people who perceive a loss of their power, leading to a breakdown in communication and cooperation.

Maintaining open communication paths is an important method to both fear and turf protection. Individuals responsible for the pavement management activities must remember that one of their responsibilities is the dissemination of information throughout the agency, not just the collection of information. Methods of disseminating information that are simple to use, current, and easily accessible will be more readily accepted than methods that are frequently outdated, hard to obtain, or slow in coming.

Innovative Tools to Address Institutional Issues

The likelihood of success of a pavement management system can be improved through the use of currently available technology to improve the skills of agency personnel and to better communicate pavement management information both inside and outside the agency. The use of CD-based programs as training tools and data access programs are being used to address the types of institutional issues discussed earlier.

CD-based technology also plays a role in enhancing the display of technical information included as part of a PMS. In this role as a data access program, the new technology provides visual reports that display distress types identified in the field, graphical representations of pavement construction histories, and other detailed information contained in the PMS database through an interactive medium. These CD-based programs have the potential to dramatically influence the way technical information is displayed for both training and reporting purposes.

THE DEVELOPMENT OF AN INTERACTIVE TRAINING TOOL

Interactive multimedia training programs provide opportunities for computer assisted learning through the use of video, audio, photographs, and text. There are a number of advantages to the use of interactive training programs as opposed to classroom instruction or video learning. For instance, computer-based training gears the training to the participants' learning needs. Through the use of hyperlinks to various portions of the training package, the participant determines the topics of interest and the length of time to spend on each topic. Testing capabilities also provide an opportunity to assess the knowledge of the participant so that the appropriate level of instruction can be provided. An incorrect answer on a quiz may trigger the computer to initiate a more remedial section that helps them find the correct answer. Other participants who understand the material would not be required to review the remedial material but would be able to review the information at a faster pace. This type of learning is referred to as interactive because the participant's response determines the sequence of training interactions. The programs are also available at any time, as long as a computer is available.

Some of the greatest advantages to this type of learning are that students learn faster, retain more, and find the learning fun. Studies have found that trainees may retain 30 percent more information using interactive computer training than through conventional methods (*2*). Other studies have shown that with multimedia programs, instruction is completed in one-third of the time of traditional instruction with 50 percent higher competency levels (*3*). The participant can repeat the information contained in the program until it is fully mastered and then move on to another topic. As a result, participants are free to learn at their own pace.

The Illinois Department of Transportation (IDOT) has used the results of its Condition Rating Survey (CRS) for planning and programming activities for over 20 years. In the mid 1990s, procedural changes in the CRS data collection procedures involved data collection using automated survey vehicles that videotaped the pavement surface, and a method of calculating the CRS values from distresses identified in each pavement section.

As a result of the changes to the CRS procedure, there was more of an emphasis on the correct identification of pavement distress type and severity than in previous years. With the new procedures, district personnel (typically technicians) viewed the videotapes of the district's pavement sections while seated at a workstation so a CRS could be calculated.

Several important issues were raised because of the changes to the CRS rating procedures. First, the districts were required to correctly identify distress types and severities that had not been required in the past. Second, the rater had to have a good understanding of CRS values to know whether the override button should be used on a pavement section. Additionally, the raters were less experienced than in previous years when highly trained individuals participated in CRS rating panels to conduct the subjective surveys.

In light of these issues, IDOT initiated a project to develop a multimedia CD-based training program that could be used as a reference tool for pavement distress identification at the workstation. Additional uses for the program became evident early into the project. The program is now being used as a tool at IDOT's annual CRS training session, as a refresher course immediately prior to conducting the CRS surveys at the workstation, and as a training program for raters not able to attend the annual training class.

Several features were incorporated into the design of the training package because of the anticipated use of the program. For example, under each of the two surface types included in the CRS procedure, the user has the option to select a particular distress type and access the distress description, photos of the distress at each severity level, and definitions for each severity level. Also, since the user may be unsure of the technical terms for the distresses showing on the video tape, the distress menu features graphical representations of each distress option and has buttons that link the user to the distress information, as shown in Figure 1.

Hypertext links and popup links were used to facilitate easy movement through the training program. Hypertext links provide a means of jumping from one portion of the program to another. Popup links, on the other hand, provide more in-depth information about a particular topic. For example, each distress screen features a photograph of the distress and Key Identifiers so a user can quickly determine whether the right distress has been accessed. Popup links are provided for access to the distress descriptions and severity levels for the selected distress. Additional popup links are used to display photographs of each severity level for the distress. The next higher and lower severity levels may be accessed from a distress severity photo so the user is able to easily compare one distress level to another. This is illustrated in Figure 2.

The last component of the training program includes an assessment of the user's understanding of the CRS distress identification procedures. The test was designed to simulate actual conditions at the workstation while the CRS surveys are being conducted. The user first views a videotape segment of a pavement section. The video may be viewed as many times as necessary for the user to comfortably begin identifying predominant distresses. When the user is ready, up to five predominant distress types and severity levels are entered into the assessment program. The entries are compared to a set submitted by the Central Office and the number of matched responses are determined. Any unmatched responses are explained to the user who may review the video again and re-enter his responses. This process is repeated until all of the distresses have been matched.

THE DEVELOPMENT OF A DATA ACCESS TOOL

In addition to being a training tool, multimedia programs provide the framework necessary to convey pavement management information in visually stimulating ways. These programs, which provide a form of data display, have been used by a number of agencies to present information to managers or as a reference tool to quickly identify specific characteristics of a particular pavement section. Two applications of these data displays are presented.

O'Hare International Airport

The City of Chicago has used a pavement management system for the identification and prioritization of its maintenance and rehabilitation needs for approximately eight years. Pavement condition index (PCI) surveys have been conducted over this period of time on all runways, taxiways, aprons, roads, and parking lots. Due to ongoing requests for airport information, the City of Chicago's

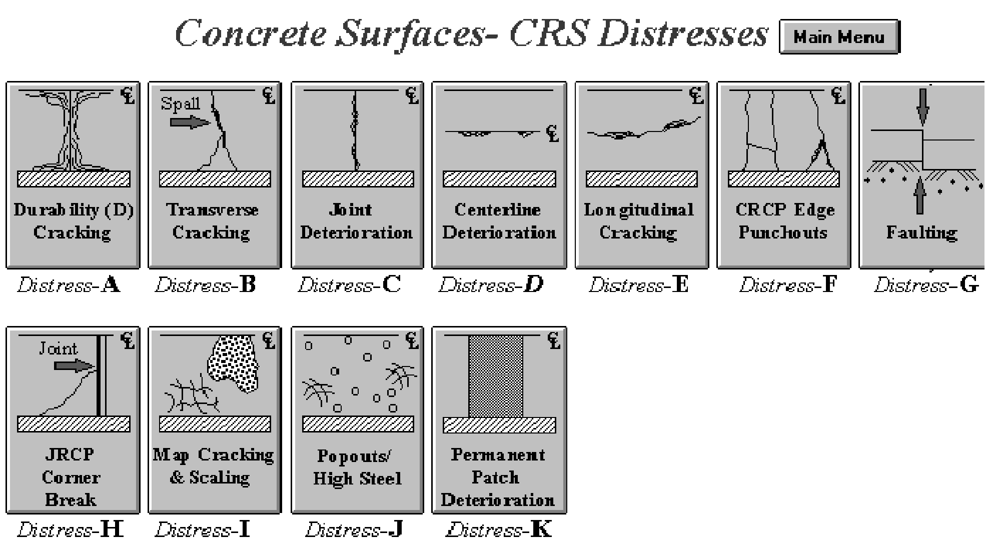

FIGURE 1 Distress selection menu.

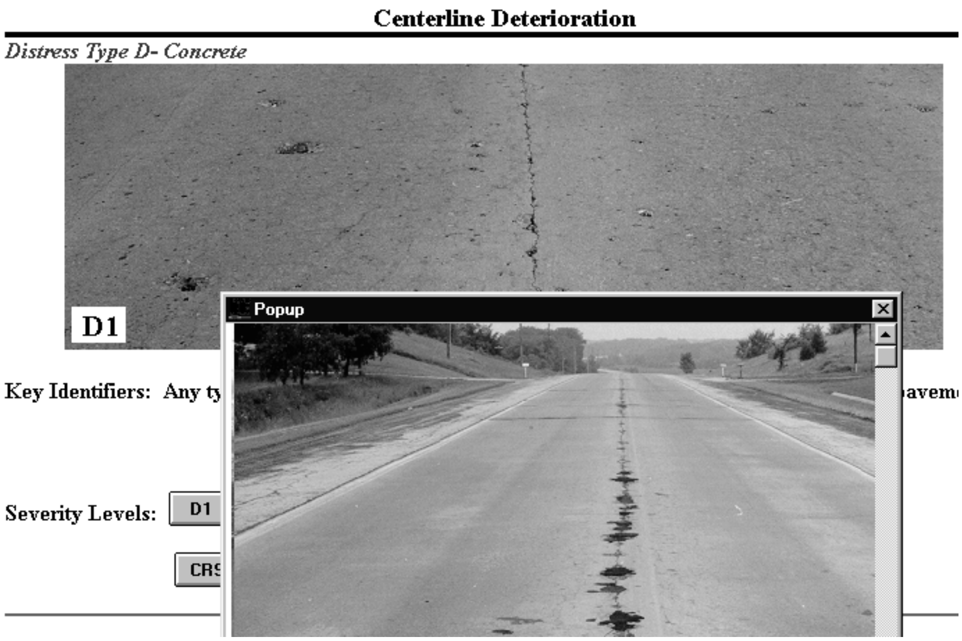

FIGURE 2 Popup links to illustrate different severity levels.

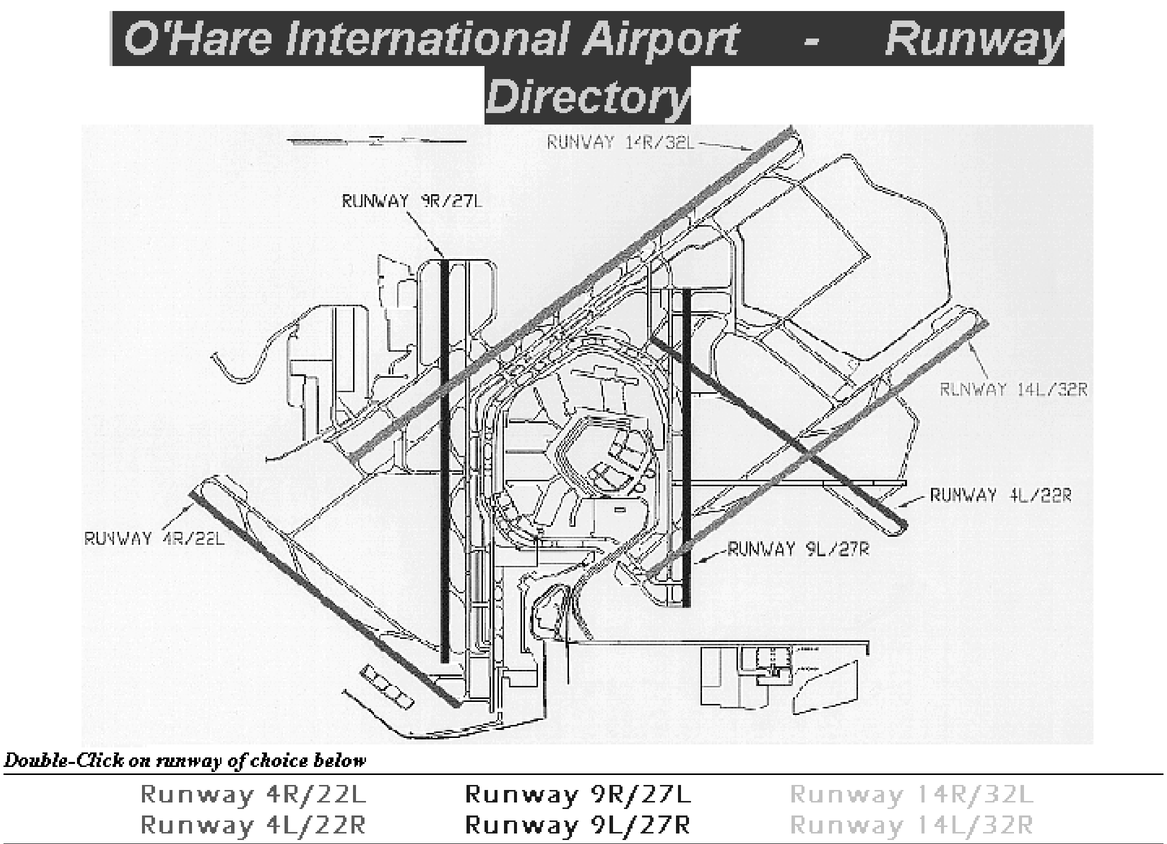

FIGURE 3 O'Hare opening menu.

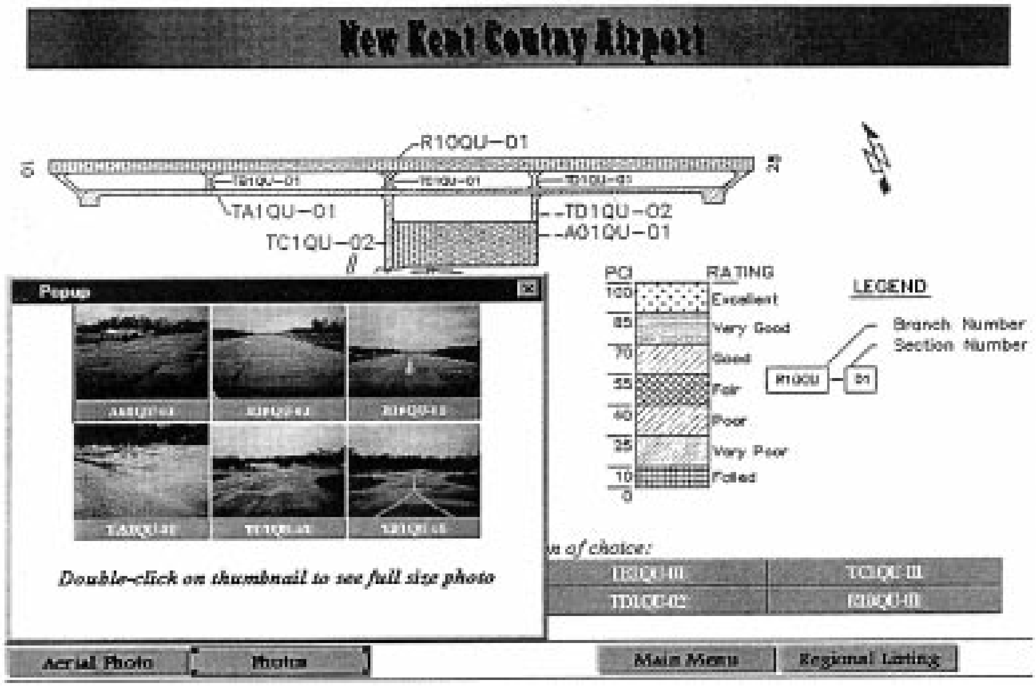

FIGURE 4 Virginia section screen.

Aviation Department began looking at new ways of conveying the information available from its pavement management system. A data display program was developed in 1997 for the six main runways at the airport to demonstrate the capabilities afforded through multimedia programs.

The data display developed for the City of Chicago makes use of graphical means of display as much as possible. For example, the opening screens for the program provide graphical representations of the runways at the airport so the City personnel can quickly locate the runway of interest without worrying about pavement section names, as shown in Figure 3. After the runway of interest is selected, another graphical representation is provided so that the pavement management section of interest can be selected.

As in the IDOT training program, popup links are used throughout the program as a means of accessing information of interest. In the O'Hare data display program, popup links access photographs of the pavement section selected to illustrate distress types located in the field or unusual conditions that were observed. The program also features a table to summarize pavement section information, construction history, and PCI survey results.

Virginia Department of Aviation

The Virginia Department of Aviation (VDOA) has been using a pavement management system for the identification of pavement-related projects at 60 of its public-use airports since 1990. Although the VDOA has information about the pavement network at most of its commercial, reliever, and general aviation airports, the pavement management program has not been installed at each of the 60 sites. Instead, the pavement management information is distributed through the VDOA to each of the participating airports, its consultants, and its sponsors after each inspection. Individual airport reports are normally distributed to each of the airports with a summary of the sectioning locations, the PCI survey results, and the maintenance recommendations. The information contained in the reports is used by airport sponsors to request funding for various projects from the VDOA. At the state level, the overall network information is used to prioritize all of the funding requests to ensure that the best network-wide decisions are made.

As part of the latest pavement management update for the Virginia airports, the project was expanded to include a CD-ROM browser that could be used as a mechanism for presenting pavement management data to airport staff and their consultants in the Commonwealth of Virginia. The program contains much of the information included in the individual airport reports that have become so popular, except that now the information will be provided on read-only CD-ROMs.

Because the VDOA network consists of a number of different airports, the data display format took on a somewhat different look than that used at O'Hare International Airport. In order to utilize graphics to the fullest extent possible, the opening menu of the program features a map of the Commonwealth of Virginia with the location of each airport included in the program identified on the map.

Once the airport of interest has been accessed, additional information is available to the user. This information features a map showing the section locations of each facility at the airport. By selecting the facility of interest, the user is provided information about the selected section, including photographs of the section, a summary of the PCI survey information, and the historical construction history. A portion of the screen for the section information is presented in Figure 4.

The VDOA is anticipating a positive reaction to the CD-ROM data display. Not only does it provide the airport and its consultants with information needed for project development, it also provides information in a digitized format that can be downloaded into word processing programs or spreadsheets for presentations, reports, and other uses. It is also expected to be used by managers to quickly assess the airport's condition or to provide general airport information to general queries.

SUMMARY

The use of interactive multimedia programs has unlimited potential in terms of the possible applications in the pavement management field. These tools show promise as an important training tool, providing a number of advantages over conventional training, including unlimited access (rather than scheduling courses), personalized training programs, and improved retention of the course material.

These programs also show potential as a means of displaying pavement management data. Consultants, commissioners, and other transportation officials find these data access programs to be an excellent way of conveying technical information without being forced to operate technical programs such as a PMS.

Other applications of these programs are expected to emerge in the next few years, as agencies become familiar with their capabilities. Through the use of interactive multimedia programs, agencies may find ways to address the institutional issues that frequently affect the success of pavement management for project and treatment selection.

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1998 TRANSPORTATION CONFERENCE PROCEEDINGS

A Methodology for Studying Crash Dependence on Demographic and Socioeconomic Data

MICHAEL D. PAWLOVICH, REGINALD R. SOULEYRETTE, AND TIM STRAUSS

Many agencies use traffic crash data to identify problems, establish goals and performance measures, measure progress of specific programs, and support development and evaluation of highway and vehicle safety countermeasures. Traditionally, efforts have considered only crash data and roadway network attributes and have not taken adjacent demographics, socioeconomics, land use, and other non-roadway variables into consideration. The evaluation of non-roadway variables may support two related types of safety management efforts: identification of additional causal factors for roadway crashes and identification of empirical relationships between crashes and non-roadway factors. The second may provide improved estimates of the impact of future changes in land use, demographics, and socioeconomics. Recent efforts use Geographic Information Systems (GIS) or non-spatial relational databases to combine crash and other data to assess correlation and causation. The variety of data available, both within the traditional approach and with the addition of demographic, socioeconomic, and land use data, creates a complex analytical environment. The complexity of these analyses warrants development of a typology to structure an assessment of the best approach in a given situation. This paper presents a concept typology to organize the use of GIS, along with statistical techniques, to explore the relationship between crash incidence and underlying demographic, socioeconomic, and land use data.

INTRODUCTION

Many agencies use traffic crash data to identify problems, establish goals and performance measures, measure progress of specific programs, and support development and evaluation of highway and vehicle safety countermeasures. Traditionally, efforts have considered only crash data and roadway network attributes and have not taken adjacent demographics, socioeconomics, land use, and other non-roadway variables into consideration.

Engineers have long studied relationships between traffic crashes and potential causal factors, traditionally focusing on roadway geometrics. The studies have generally not considered characteristics of demographics, socioeconomics, and land use in the area proximate to crashes. Efforts on a microscopic (intersection or corridor) level have been made to determine crash causality based on socioeconomic and demographic features, but little has been done to expand this to a macroscopic (network or citywide) level.

Many studies have focused on determining causal factors for traffic crashes (*1,2,3,4*). Other studies utilize traffic crash data to determine cost effectiveness of improvements (*5,6*). Still others focus on factors to reduce crash frequency, fatalities and injuries, and response time. Few sources mention land use, demographics, or socioeconomics in relation to traffic accidents. Of these, two older sources (1965 and 1969) focus on demographics of persons involved in crashes (*7,8*), one mentions land development and traffic influences on road accidents (*9*), and another analyzes a variety of subjects in addition to land use (*10*). No articles found considered demographic, socioeconomic, or land use data in relation to traffic crashes on a macroscopic level.

The evaluation of non-roadway variables may support two related types of safety management efforts. First, it may identify additional causal factors for roadway crashes. Once all such factors have been identified and analyzed, better-informed decisions can be made to remediate existing or potential hazardous locations. Identified non-roadway causal factors will enable engineers and planners to design and plan safer roadways and neighborhoods by providing a clearer picture of contributing factors in certain crashes or crash types. Changes in crash numbers would more clearly be linked to actual causes.

Second, short of causality, the identification of empirical relationships between crashes and non-roadway factors may provide better estimates of the impact of future changes in land use, demographics, and socioeconomics. Such empirical relationships would be useful to guide the allocation of emergency response (e.g., ambulance and police) resources necessary to respond to the potential additional demands presented by new residential and economic developments located in specific locations, and by changing demographic and socioeconomic patterns. However, as few studies have considered the relationship of demographics, socioeconomics, or land use to crashes or crash rates, these variables are not available for design, planning, or analysis.

Recent efforts use Geographic Information Systems (GIS) or non-spatial relational databases to combine crash and other data to assess correlation and causation (*11,12,13*). GISs provide excellent tools to analyze location specific crash data. Multiple layers

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can be viewed and analyzed at once. In addition, GISs enable development of a methodology to consider non-roadway variables.

Currently, the Center for Transportation Research and Education (CTRE) is developing a GIS-based accident location and analysis system for the state of Iowa that facilitates spatial analyses of crash incidence. Iowa is fortunate 1) to have a comprehensive location–based database covering 10 years of all traffic crashes on all road systems, and 2) to have developed one of the better systems for analyses, Personal Computer-based Accident Location and Analysis System (PC-ALAS). Many approaches are available to analyze crash data in a GIS environment. The variety of data available, both within the traditional approach and with the addition of demographic, socioeconomic, and land use data, creates a complex analytical environment. The complexity of these analyses warrants development of a typology to structure an assessment of the best approach in a given situation.

A topological (i.e., based on feature class) division of crash rates includes three types of geographic representations: point, line, and polygon. Points can either represent the location of a single crash or the location of a point where multiple crashes have occurred. Lines can denote a segment or corridor with multiple crashes or a network made up of a series of lines, combining the crashes on each line to develop the crash rate. Polygon representations combine crashes within an area in order to develop an areawide crash rate. Polygons can be further divided, representing regions using an arbitrary grid or block groups, depending on the data available and the desired analysis.

Utilizing the topological representation of crash rates as the dependent (Y) variable, various independent (X) variables, which also can be represented with varying topology, can be used to determine potential causal relationships. As the analysis of these topological representations can become quite complex, a classification scheme (typology) to structure the analyses is helpful. This paper presents a concept typology to organize the use of GIS, along with statistical techniques, to explore the relationship between crash incidence and underlying demographic, socioeconomic, and land use data.

TYPOLOGY

The typology, as shown in Figure 1, consists of two topological dimensions: dependent variable (crash rate) and independent variable (here, demographic, socioeconomic, and land use). Prior to representation with varying topology, the dependent variable must be created from a spatial combination of crash incidence and exposure (traffic levels). In this study, crash locations are represented as points. Three methods are presented here to develop crash rates (point on line and point on polygon, including arbitrary grid and census block), creating three sets of dependent variables for subsequent analyses. In addition, the three sets of dependent variables are statistically related to three independent variables, creating nine or more possible types of analyses.

Each dependent variable/independent variable pair can be utilized to assess the impact of different features of the independent variables on crash rates as shown in Figure 2. For example, the arbitrary grid/economics combination could be utilized to determine impact of business point locations on an areal (grid) crash rate. Employment densities within grids could be related to crash rates within grids. In addition, given an accident location, all businesses within a grid of an accident location could be determined.

crashes (point) overlaid on:			
arbitrary grid (poly)	impact of business point locations on areal (grid) crash rate	impact of census tract median income on areal (grid) crash rate	impact of land use zoning on areal (grid) crash rate
block group (poly)	impact of business point locations on areal (census) crash rate	impact of census tract average age range on areal (census) crash rate	impact of land use zoning on areal (census) crash rate
roadway (line)	impact of business point locations on segment based (roadway) crash rate	impact of census tract average vehicles per household on segment based (roadway) crash rate	impact of land use zoning on segment based (roadway) crash rate
vs.	economics (point)	socioeconomic/ demographics by block group (poly)	land use by block group (poly)

FIGURE 1 Crash rate typology.

	traffic/ geometrics	demographics/ socioeconomics/ land use	employment
points	ALAS file fields related to roadway crash characteristics	sex, age, DWI, ALAS file	# employees, point locations of employers
line	Base Records (AADT, lane width, VMT)	access points buffer	access points buffer
polygon	VMT	census data, zoning	densities

FIGURE 2 Causal factors matrix.

METHODOLOGY

The methodology to develop dependent and independent variables consists of six main steps, as shown in Figure 3. The first three steps are used to develop independent variable data, the fourth to develop dependent variables, the fifth to develop independent variables themselves, and the sixth to apply statistical techniques to determine significant causal relationships. In this paper, the methodology is demonstrated at the block group level.

Step 1

Data available include census data, crash data, infrastructure data (mainly roadway data), and employment data. An enormous amount of data is available, resulting in computational problems for standard statistical packages; therefore, the variable list was narrowed to those viewed as most promising by the authors. Initially, we arbitrarily selected variables that seemed most likely to affect crash rates.

Census data contains over 3,400 data elements that can all be referenced to geographic regions. These elements are divided into several main headings and subheadings. To provide more manageable census data for the purposes of this paper, data under certain headings, unlikely to relate to crash rates, were discarded from consideration. The remaining data elements were contained in 261 main headings and subheadings. From these we arbitrarily selected a portion for subsequent analyses. The selection left us with a large, but much more manageable number of total variables (225 variables under 25 main headings) to consider.

Infrastructure data obtained from the Iowa Department of Transportation (DOT) include several background GIS coverages such as hydrology, rail lines, secondary roads, primary roads, and municipal roads. The latter three of these were of significant interest for this paper. Included as attributes for the road coverages are AADT and lane length. The AADT and lane length were used to calculate Vehicle Miles Traveled (VMT) for each roadway segment. The VMT, combined with total crashes along each roadway segment, were used to construct the dependent variable, crash rate, for each roadway element.

Crash data also obtained from the Iowa DOT includes many data elements related to the crash, the vehicles and drivers, and the injured persons. These data elements are explained in a recently published report detailing current efforts to expand and improve Iowa's accident location and analysis system (*14*). For the purposes of this paper, relevant information is the incidence and location of crashes. Combining these data with VMT results in crash rate (crashes/100 million VMT). Future efforts may consider specific crash attributes.

Socioeconomic data contains: business name, address, city, state, zipcode, Standard Industrial Code (SIC), and number of employees. The data were obtained from the Iowa Department of Workforce Development (DWD) and are confidential. The SIC code is the most important element, though number of employees may be of interest. The other variables were used to create a point location map of businesses using the geocode function of a GIS.

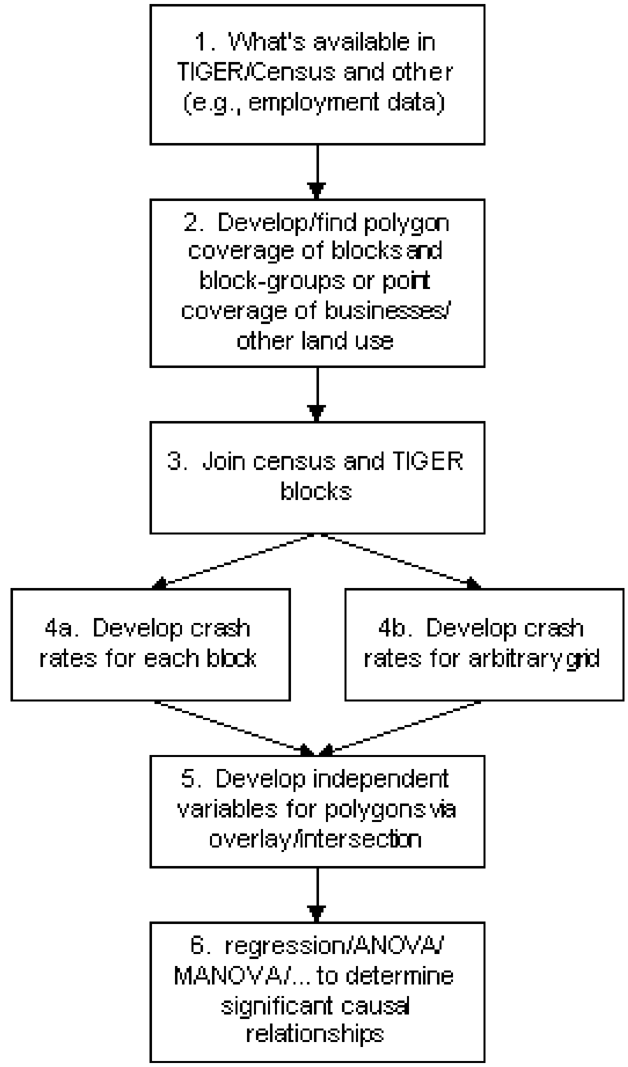

FIGURE 3 Equation development process.

Step 2

The second step was to develop or locate polygon coverages of block groups and point coverages of businesses and other land use data. Polygon coverages of block groups, illustrated in Figure 4, were obtained from a commercial product and exported to GIS format. The point coverages of businesses, displayed in Figure 5, were developed from DWD data. Utilizing the address code of the DWD data and a commercial street address database, point coverages were developed using the geocode function of a desktop GIS.

Step 3

The third step was to join census data and Topologically Integrated Geographic Encoding and Referencing (TIGER) blocks (i.e., polygons for block groups). However, the census data and TIGER block join was found to be commercially available; therefore, no work was entailed in this step other than querying for desired census information and exporting it into GIS format.

Step 4

The fourth step was to develop crash rates, the dependent variable, for each independent variable type: block group, arbitrary grid, and linear system. For this paper, only the first, crash rates for block groups, was completed. The latter two independent variable types will be considered in future efforts.

To develop crash rates for block groups, first the Iowa DOT roadway coverages were spatially joined to crash data, as shown in Figure 6. The resultant table was then summarized to produce a table of crashes by using the roadway coverage index fields. The summarization table and roadway coverages were then spatially joined, creating roadway coverages with total crashes along each roadway segment.

The VMT for each block group was calculated by first spatially joining each block group to the road coverages, as shown in Figure 7. The resultant road coverages with the block group table was then exported to dBase format and imported into Microsoft Access. Within Access, the crashes, average annual daily traffic (AADT), and lane length (meters) were grouped by area identifier using a summation for each. The grouping results were saved and imported into GIS and then spatially joined to the block group coverage. Each block group now includes total number of crashes, total AADT, and total lane length as attributes. Additional fields, VMT and crash rate, were created and their values calculated for each block group using the following formulas:

- VMT = total AADT * 365 days/year * lane length (meters) / 1.609 meters/mile; and
- Crash rate = total crashes/(1000000*VMT).

After generation of the grid, development of crash rates for an arbitrary grid would proceed similarly. Linear system crash rates would involve the spatial joining of the roadway and crash data and the subsequent calculation of VMT and crash rate.

Step 5

The fifth step was the development of the independent variables, for polygons via overlay/intersection. This was accomplished by joining the census data to the crash rate data. An overlay of block groups, crashes, and business locations is shown in Figure 8.

Step 6

The sixth step uses the independent and dependent variables developed previously to examine the data for significant causal relationships. Various statistical techniques may be utilized, including linear regression, analysis of variance (ANOVA), multiple regression ANOVA (MANOVA), factor analysis, bivariate regression, time series, and spatial regression. Within the selected GIS environment, few rigorous statistical techniques were available; however, a simple bivariate regression script is available for analyzing data. In addition, exporting the database to a spreadsheet allowed for more involved multiple regression. For this paper, various variables were tested against the block group crash rate for a variety of regions to ascertain whether there were any obvious causal factors. This was done using a desktop statistical software package once the data had been exported in delimited text format from the GIS. Future efforts will involve statistical packages allowing the more robust analyses desired.

FIGURE 4 Block groups.

FIGURE 5 Socioeconomic point locations.

FIGURE 6 Iowa DOT roadway and crash coverages.

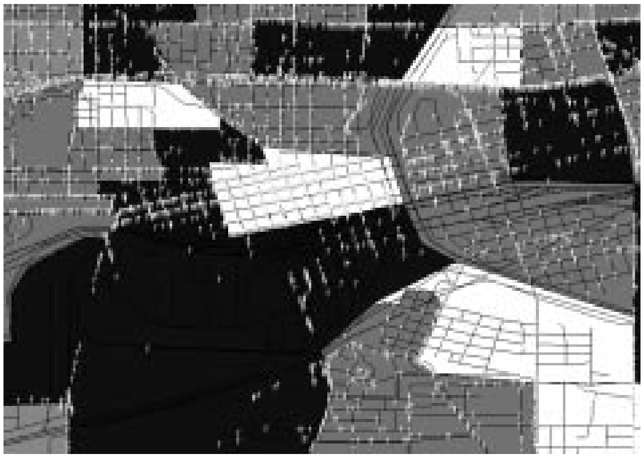

FIGURE 7 Roadway, crash and block group coverages.

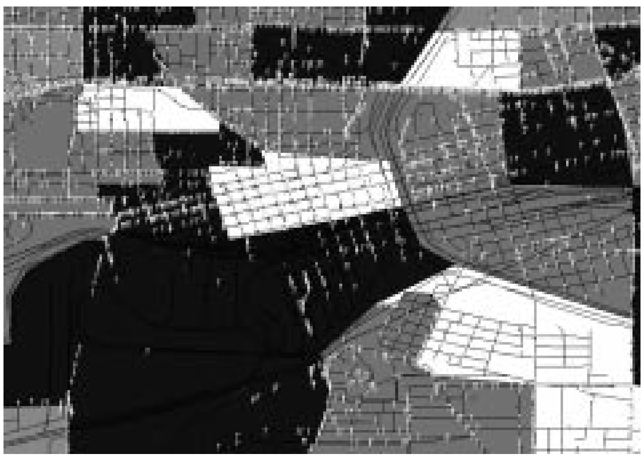

FIGURE 8 Block groups, crashes, and business location coverages.

	Coefficient	'P'
S_PrpSch	-12.210	0.000
L_WlkHom	6.533	0.044
OcSales	5.726	0.000
OcFarming	23.837	0.002
InWhoTrd	-8.441	0.001
InBusiness	7.702	0.001
InPerson	-10.363	0.002
InEntert	-9.790	0.024
InPubadm	5.250	0.027
L_SlfEmp	5.028	0.039
Age6	14.813	0.000
Age16	-8.108	0.032
Age35_39	3.006	0.022
Age62_64	6.896	0.001
Age70_74	-5.135	0.003
MC_ChU3	-8.386	0.002
MnW_Ch13	27.187	0.031
DrvWkHom	-7.529	0.030
Tim10_14	-4.668	0.000
Tim35_39	29.098	0.001
Tim40_44	25.684	0.000
Tim60_89	9.317	0.104
Ch5P2FWk	-111596.094	0.104
Ch5P2MWk	2582.544	0.000
Ch17P2FW	-80085.038	0.003
Ch17MaNW	-7775.235	0.014
MedFami	-206287.871	0.031

FIGURE 9 Regression results.

ANALYSIS

The initial analysis effort utilized a subset of the available data. Future efforts will analyze the data more comprehensively. However, a variety of factors contributed to the limited analyses performed at this time.

Using a metropolitan region for the analyses, the data fields were pared to an arbitrary set of variables of interest. These variables were then exported to delimited text format from the GIS and imported into the desktop statistical package for analyses. A backward, stepwise, linear regression was performed, using entry and exit probabilities of 0.15. The correlation coefficient (R = 0.677) and the coefficient of determination (R-squared = 0.458) indicate that the equation is moderately successful in predicting crash rates at the block group level (see Figure 9). Several independent variables remained in the estimated equation, 27 in all. Of these, 10 were statistically significant at the 0.001 level, 15 at the 0.01 level, and 25 at the 0.05 level. Independent variables with a calculated statistical significance of "0.000" included:

- Persons 3 years and over enrolled in preprimary school (S_PrpSch)
- Employed persons 16 years and over who are in sales occupations (OcSales)

	Factor								
	1	2	3	4	5	6	7	8	9
Tim10_14	0.805	-0.167	-0.085	-0.023	0.115	0.184	0.098	0.045	0.04
OcSales	0.757	0.339	0.032	-0.092	0.129	0.081	0.072	-0.003	0.102
Age35_39	0.742	0.028	0.038	0.015	0.328	0.174	0.133	0.109	0.081
InBusiness	0.674	0.302	0.081	-0.146	0.059	0.058	-0.079	0.007	-0.016
MC_ChU3	0.664	-0.019	0.151	0.099	0.363	0.072	-0.024	0.117	0.314
InWhoTrd	0.656	-0.195	0.226	0.102	0.211	0.139	0.087	-0.256	0.06
InPubadm	0.625	-0.228	0.04	0.117	0.176	0.114	0.192	0.061	0.07
InPerson	0.617	0.257	0.069	-0.266	-0.009	-0.031	0.037	0.055	-0.029
Age16	0.595	0.152	-0.081	0.192	0.106	0.149	-0.007	-0.144	0.036
Ch17MaNW	0.035	0.794	-0.126	0.017	0.154	0.054	0.039	-0.04	-0.027
MedFami	0.177	-0.653	-0.094	-0.119	0.255	0.184	0.099	-0.089	0.091
Crash_Rate	-0.171	-0.079	0.753	-0.145	-0.004	-0.018	0.175	0.061	-0.119
Tim40_44	0.263	0.006	0.703	0.065	0.09	0.049	-0.135	-0.1	0.123
L_WlkHom	-0.125	-0.016	0.048	-0.733	-0.183	-0.108	0.021	0.071	-0.197
DrvWkHom	0.325	-0.166	-0.023	-0.626	0.27	0.212	-0.088	-0.054	0.274
Ch17P2FW	0.093	-0.179	-0.004	0.044	0.761	0.106	0.044	-0.088	-0.084
Ch5P2FWk	0.332	0.084	0.101	0.012	0.567	-0.032	-0.191	0.123	0.219
Age6	0.242	0.388	0.155	0.114	0.553	0.049	0.303	0.12	-0.036
S_PrpSch	0.421	0.135	0.026	-0.024	0.552	0.134	0.304	0.206	0.163
Age70_74	0.238	-0.088	-0.169	0.031	0.055	0.773	0.029	0.063	-0.169
Age62_64	0.174	0.022	0.247	0	0.072	0.755	-0.011	-0.059	0.051
Tim35_39	0.257	-0.009	0.001	-0.001	0.077	-0.029	0.821	-0.096	0.018
Ch5P2MWk	0.113	0.191	0.239	-0.027	0.115	0.193	0.016	0.691	0.276
OcFarming	0.067	0.152	0.29	0.021	0.009	0.211	0.137	-0.69	0.267
InEntert	0.213	-0.081	-0.033	0.068	0.048	-0.087	0.024	0.015	0.747
MnW_Ch13	0.311	0.128	-0.065	-0.05	0.4	-0.214	-0.369	-0.019	-0.331
Tim60_89	0.297	0.076	0.345	0.231	0.052	0.168	-0.11	0.08	-0.301
L_SlfEmp	0.393	-0.223	-0.006	-0.326	0.403	0.351	-0.024	-0.109	0.253

FIGURE 10 Factor analysis results.

- Persons aged 6 years old (Age6)
- Workers 16 years and over, not working at home, whose travel time to work was 10-14 minutes (Tim10_14)
- Workers 16 years and over, not working at home, whose travel time to work was 40-44 minutes (Tim14_44)
- Families with two parents and children under 6 years with only the mother in the labor force (Ch5P2MWk)

At first glance, the results seem promising. A few caveats are necessary, however. First, one must be careful about assigning causality to the above relationships. The results should not be interpreted as indicating that crashes are caused by 6-year-old children, and their preprimary aged siblings, with commuting salesman fathers and stay-at-home mothers. The results might indicate, however, that locations with families having these characteristics, may have above-average block-group-based crash rates (controlling for a limited set of other factors). Second, regression diagnostics indicated that the above results should be viewed cautiously. In particular, extreme values may be skewing the results. A series of scatterplots with the dependent variable and selected independent variables indicated that this is the case. A few block groups with extremely high values greatly affected the results. Normally, these observations would be discarded, but in crash analysis they are usually the ones of most interest. Third, although a surprising number of interrelated independent variables entered the final equation, a high degree of multicollinearity, typical with socioeconomic data, can make regression parameters unstable and substantive interpretation difficult.

To assess the third point, a principal components analysis (PCA) was performed on the 27 independent variables to identify highly correlated groups of variables (minimum eigenvalue = 1.0, varimax rotation). Nine factors resulted from this and are presented in Figure 10. Analysis of the results requires a bit of interpretation. For instance, variables with high loadings on the first component include:

- Workers 16 years and over, not working at home, whose travel time to work was 10-14 minutes (Tim10_14)
- Employed persons 16 years and over who are in sales occupations (OcSales)
- Persons aged 35-39
- Employed persons 16 years and over who are in public administration industries (InPubadm).

This component can be interpreted as representing neighborhoods with thirtysomething professional service industry workers with average commute times. Other components can be similarly interpreted.

To assess the potential impact of these groups of variables on block-group-based crash rates, the components can, in turn, be used as independent variables in a regression analysis. The results were less useful than the original regression, however, since the R (0.276) and R^2 (.076) were lower (worse for empirical prediction), and the difficult interpretation of the components that entered the equation makes it difficult to derive much meaning from the results that we could use to establish causal factors. Additionally, only factor 1 (0.054) and factor 6 (0.000), had p-values of statistical significance.

CONCLUSIONS

The development of the above typology is a promising approach, but mainly for empirical prediction (e.g., to estimate impact of changes on number of crashes in a given development, city, or emergency response district). Statistical associations and patterns discovered through using this typology can be examined for possible causal significance, which would then be assessed via more detailed studies.

Though regression using individual variables gave good results for prediction, the results are hard to interpret substantively. In addition, use of PCA made the equation worse and did not aid interpretation. The next step would be mapping to find block groups with high values for the two statistically significant components.

Some data issues made the made the analysis more difficult. Extreme values/outliers make it difficult to get meaningful results from regression. Additionally, these extreme values/outliers skew the analysis. However, these values are more interesting in from a safety perspective.

Additionally, the typology might be developed further for two different types of analyses. Examining immediate corridor proximity would facilitate causal analysis and engineering countermeasures. Examining broader areas (e.g., block groups) is better for planning applications such as police and fire response or broader estimates of changes in crash statistics and patterns.

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Collision Diagram Software Compatability with Iowa Accident Database

DUANE E. SMITH, JEFF GERKEN, AND PHIL MESCHER

The Iowa DOT was interested in automated collision diagram products. The Center for Transportation Research and Education (CTRE), an Iowa State University center, completed an evaluation. This paper presents the findings. An automated collision diagram program quickly and accurately generates a graphic of intersection accident history. Limited human resources can concentrate on the safety analysis and not on manually generating collision diagrams resulting in a more efficient safety analysis program. The Iowa DOT was interested in software packages that were currently available, used by other DOTs, and how packages compared to the requirements. Fourteen packages were initially identified by CTRE. After the first evaluation step, nine packages were considered for evaluation. A decision matrix was developed that provided a "go" "no go" to the individual programs. From this, four programs were obtained by CTRE for further analysis, and an investigation of each was conducted. From this analysis, a final selection was made. Comparing to the requirements, Intersection Magic, distributed by Pd' Programming, was the program that the Iowa DOT selected for their collision diagram package. The software displays accident history in graphical formats and use filters to segregate graphics for specific inquiries. This allows the evaluator the opportunity to look at different types of accidents and see if there are trends that warrant further evaluation. The Iowa DOT is in the process of comparing the results from Intersection Magic with previously generated diagrams and developing a program for implementation in field offices. Key words: collision, diagram, Intersection Magic.

INTRODUCTION

The Iowa DOT was interested in implementing an automated collision diagram product because of the wide usage of this evaluation tool at all levels of government. The Center for Transportation Research and Education (CTRE), a center of Iowa State University, completed an evaluation of the potential software packages for the Iowa DOT and this paper presents the findings.

State, county, and city engineers and planners are responsible for analyzing traffic crash data as a part of their duties. They analyze crash data for the purposes of developing a list of locations where crashes have occurred, ranking locations according to crash numbers and rates, and developing reports for each location selected for analysis. They create a collision diagram that graphically displays crash trends, and for the purpose of preparing reports for public information meetings, budget preparation, or funding requests. Generating a collision diagram generally concludes the data collection and analysis process and allows planners and engineers the opportunity to focus on specific initiatives that are directed toward specific crash trends. It is desirable to automate the collision diagram development process and integrate the software application with the existing accident database at the Iowa Department of Transportation.

The underlying purpose of an automated collision diagram program is to have the ability to quickly and accurately generate a visual description of the accident history for a specific location. This also means that limited human resources can concentrate on safety analysis where it is most needed. Resources that were previously spent on the generation of collision diagrams by manual applications can now be channeled into more in-depth analysis, resulting in a more efficient and better safety analysis program.

Currently, the Iowa DOT's Traffic Safety staff produce collision diagrams that are hand drawn. Technicians must first research accident records by accessing the Iowa PC-ALAS (Personal Computer-Accident Location and Analysis System) database and then generate a collision diagram manually which visually displays the accident history. A summary of data, collected from PC-ALAS, is attached to the collision diagram and groups the accident reports by type of accident (corresponding to the visual display). Because of reduced staffing at the Iowa DOT, collision diagrams are currently drawn only for Hazardous Elimination System (HES) projects and major problem intersections.

The Iowa electronic accident record database has existed since 1977. Although the format has changed from a mainframe system to a personal computer system, the database has managed the same basic accident record information over the years. The current version, called PC-ALAS, utilizes flat files that are in ASCII text format arranged into "A," "B," and "C" records. The "A" record contains the general information about the accident, the "B" record contains driver and vehicle specific information, and the "C" record contains injury information. Every accident record will contain an "A" record and at least one "B" record, but the presence of additional "B" or "C" records varies with each accident. Upon request for accident information, the Iowa DOT can query PC-ALAS for a 3-5 year period (urban locations) or up to a 10 year period (rural locations) for accident data. The accident location system is a link-node system utilizing eight-digit node numbers assigned to intersections and other roadway features on a quasi-coordinate system.

The Iowa accident system contains 70,000-75,000 accidents for each year. The magnitude of this system mandates that a collision diagram software package be sophisticated enough to handle all accident records, including an expanded database for future years. The Iowa DOT is in the process of converting the PC-ALAS data-

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base to Microsoft Access. They are also planning to interface the ALAS database with a Geographical Information System (GIS) platform and create a GIS-ALAS application. All of these issues including GIS-ALAS development were considered during the collision diagram software evaluation.

Evaluation Requirements

The Iowa DOT identified specific data fields they required be included in a collision diagram software package. The data fields were prioritized by level of importance into primary, secondary, and tertiary data fields. These data fields are shown in Table 1. The primary data fields were required to be a part of any collision diagram software package the Iowa DOT would consider. The secondary and tertiary data fields were not as important to the Iowa DOT but would be weighed in the evaluation. These fields are derived from the officer's accident report and are used in the PC-ALAS database. These data fields are used to conduct filters or queries that help engineers and technicians conduct traffic accident analysis.

In addition, other evaluation requirements were established:

- Utilizes a PC with a 386 processor or higher, MS Windows 3.1 or later, 4 MB of RAM, VGA or higher resolution monitor, 6 MB of hard disk space, and any compatible printer
- PC-ALAS data format compatibility
- Color plotting
- Adaptable to a GIS platform
- Calculate accident rates
- Accident record edit functions
- Quality presentation graphics
- Apply filters and complete queries
- Statistical reports of analysis
- Retrieve accident record by clicking on accident icon
- Alter intersection alignment and move accidents around within intersection
- Level of support from software developer.

FIRST STEP EVALUATION

The Iowa DOT was interested in reviewing collision diagram software packages that were currently available, if any other Department of Transportation was using a collision diagram software package, and how the packages compared to the Iowa DOT requirements.

The following list of fourteen includes all of the software packages that CTRE explored for compatibility with the Iowa DOT requirements. Some of these packages are commercially available and others are developed in house by a transportation agency.

- Intersection Magic (Pd' Programming)
- Accident Information Management System: Geographic Information System (AIMS:GIS)
- Collision Database System (Crossroads Software)
- Accident Surveillance & Analysis Program (ASAP) (Hank Mohle & Associates)
- Collision Plot Program (Illinois DOT)
- Collision Diagram for Windows 2.1, COLLDIAG (Ohio Department of Public Safety)
- TRACPLOT (Ohio DOT)
- Collision Diagrams (Wyoming Highway Department)

Table 1 PC-ALAS Data Fields

	PC-ALAS Data Fields	
Primary Fields	Secondary Fields	Tertiary Fields
Case: year, prefix, and number	Accident severity code: Fatal, injury, property damage	Report type
Date of accident	Total killed	City
County number	Total injured by severity level (major, minor, unknown)	Intersection class
Intersection identifier (node-based system)	Total vehicles	Locality
Reference node	Total property damage	Special use: police, fire, taxi, etc.
Distance indicator	Day of week	Number of occupants
Direction node	Time of day	Vehicle defects
Collision type	Route number	Type of surface
Initial direction of travel	Road class: interstate, US or state highway, county road, city street	Location of fixed object
Vehicle action (prior to accident)	Type of accident	Drivers sex
	Character of roadway	Position of injured pedestrian
	Roadway geometrics	Protective devices
	Light conditions	Sobriety of pedestrian
	Weather conditions	
	Location: on roadway, shoulder, median, etc.	
	Vehicle type	
	Point of initial contact	
	Contributing circumstances	
	Traffic controls	
	Type of roadway: # of lanes, ramp, etc.	
	Traffic flow: 1 way, 2 way	
	Fixed object struck	
	Surface conditions	
	Driver age	
	Driver charged	
	Sobriety test given and results	
	Driver condition	
	Driver/vehicle contributing factors	
	Vision obscured	
	Injury severity (pedestrian)	
	Protective device	
	Pedestrian action	

- Intersection Collision Plot Diagram (Texas Safety and Traffic Operations)
- Traffic Operations System Software TOSS (University of Kansas)
- AutoCAD
- Small Computer Accident Records System SCARS (University of Florida)

Table 2 State Department of Transportation Inquiry

State	Software
New York	No current collision software
Ohio	COLLDIAG for Windows
Missouri	No current collision software
Kansas	TOSS and Intersection Magic
Wyoming	FORTRAN PROGRAM
Pennsylvania	No current collision software
Connecticut	No current collision software
Texas	Mainframe program
California	No current collision software

- Small Computer Collision Diagram SCCOLD (University of Florida)
- Accident Records, Summary and Diagrams (ACCISUM) (University of Kansas)

Several states were using or were considering the use of collision diagram software packages and each of them was contacted to evaluate their particular application, if any, and the correlation to the Iowa DOT requirements. This information is located in Table 1 and was current at the end of 1996.

Five of the software programs were out of date and were typically based on a main frame system. Others obviously were not compatible with PC-ALAS format and could not be made compatible without extensive reprogramming. Some programs were simply not sent to CTRE for evaluation.

SECOND STEP EVALUATION

Nine software packages were considered for further evaluation and the are listed below.

- Intersection Magic (Pd' Programming)
- Accident Information Management System: GIS, AIMS:GIS (JMW Engineering, Inc.)
- Collision Database System (Crossroads Software)
- ASAP Accident Surveillance & Analysis Program (Hank Mohle and Assoc.)
- Collision Plot Program (Illinois DOT)
- COLLDIAG for Windows (Ohio DOT)
- Traffic Operations System Software, TOSS (University of Kansas)
- Small Computer Collision Diagram, SCCOLD (University of Florida)
- Accident Records, Summary and Diagrams, ACCISUM (University of Kansas)

A decision matrix was developed that gave a "go" "no go" to the individual software programs that were evaluated at this second level. Table 2 indicates the various software package's individual capabilities along with a notation as to whether the packages were obtained for further testing. The software was evaluated against the functional criteria. The packages that fit the most criteria and offered the most interest by the developer were studied further. Many of the programs were not windows based and were dismissed.

Table 3 Decision Matrix for Continued Evaluation

Software	PC-ALAS compatible	System Requirements	Graphics quality	Obtained for further analysis
Intersection Magic	x	x	x	x
AIMS:GIS	x	x	x	x
Collision Database	x	x	x	x
ASAP	x		x	
Collision Plot Program		x	x	
COLLDIAG	x	x	x	x
TOSS	x		x	
SCCOLD		x	x	
ACCISUM	x	x		

Table 4 Final Evaluation Matrix

Software	Editing capability	GIS compatible	Graphics quality	Query capabilities
Intersection Magic	x	x	x	x
AIMS:GIS	x	x		x
Collision Database	x	x		x
COLLDIAG	x	x		x

THIRD STEP EVALUATION

The characteristics of the collision diagram software programs that received the most consideration were compatibility with the existing PC-ALAS structure, system requirements such as associated software and hardware, and quality of the collision diagram graphics.

Based on these characteristics, the following software packages were obtained for further investigation.

Four software packages were obtained for further analysis. The research team at CTRE acquired the collision diagram programs from individual suppliers or from government agencies. They tried to integrate them with the ALAS data base and evaluated them against the requirements established for the project. The four that were acquired include Intersection Magic (Pd' Programming), AIMS:GIS Accident Software (JMW Engineering), Collision Database System (Crossroads Software), and COLLDIAG for Windows (Ohio DOT).

Demonstration software packages were obtained and investigations of each software package were conducted. Introductory meetings with the Iowa DOT narrowed the focus to two software packages through the use of initial demonstrations. The software packages chosen for in-depth investigation were Intersection Magic and AIMS:GIS. The other two programs were dismissed because they were not compatible with a GIS platform. CTRE obtained 1993 Jasper County accident records taken from PC-ALAS data files and sent them to Pd' Programming and JMW Engineering. CTRE wanted to test the compatibility of the data files with these

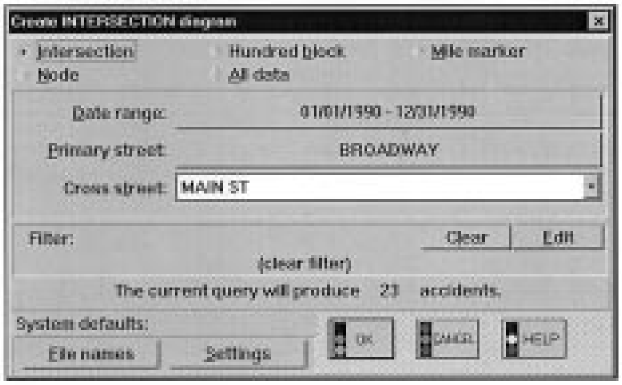

FIGURE 1 Intersection menu.

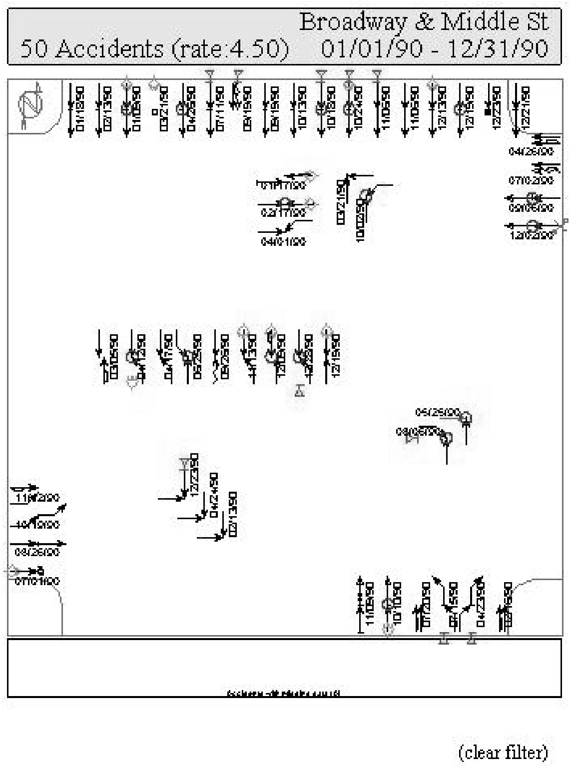

FIGURE 2 Total accidents.

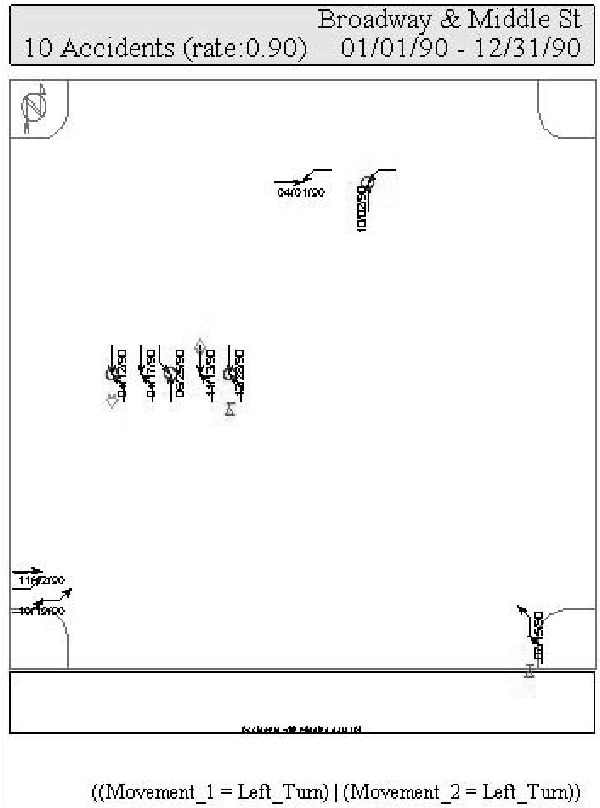

FIGURE 3 Left turn accidents.

two programs. It was a good test of the service the Iowa DOT could anticipate from the software distributor in the future. After some minor problems with formatting, both Pd' Programming and JMW Engineering sent us working copies of their respective software packages. Testing and evaluations were completed within several weeks and a working session with Iowa DOT safety analysis technicians was conducted. A meeting with the Iowa DOT safety engineering staff engineers took place soon after the working session.

To further verify the conclusion reached by CTRE, state department of transportation representatives from other states were interviewed to see if the Intersection Magic program they were using gave the same results that CTRE experienced in the test and evaluation process. All of the responses were positive and supported the conclusion that CTRE derived. The list of states and contacts are shown in Table 4.

RESULTS

There were many good collision diagram software programs that CTRE evaluated. Compared to the requirements established by the Iowa DOT, Intersection Magic, distributed by Pd' Programming, would be the program that the Iowa DOT would use for their collision diagram requirements. The Iowa DOT made this decision because Intersection Magic met all of the requirements that had been established along with the following:

- GIS compatible
- Editable database and intersection geometry
- Variety of symbols for accident types
- Color plots
- Query and filter capabilities
- Statistical reports are generated
- Graphics were superior
- Superior software support
- Superior data displays and graphics
- Ability to print a variety of reports
- Internal scripting language for repetitive tasks was very sophisticated.

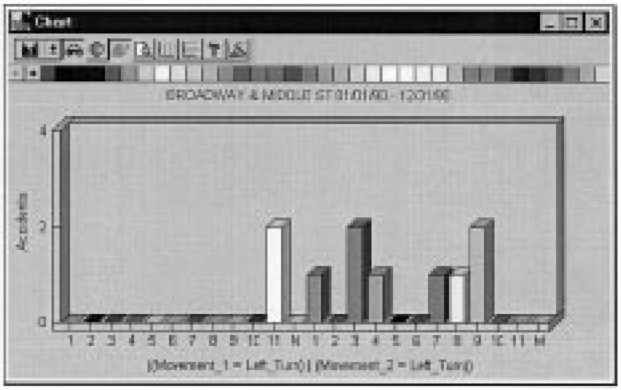

FIGURE 4 Time of day left turn accidents occurred.

Table 5 DOT Contacts Using Intersection Magic Collision Diagram Software

State	Name	Telephone Number
Kentucky	Boyd Sigler	(502) 564-3020
Idaho	Gary Sanderson	(208) 334-8487
South Dakota	Larry Dean	(605) 773-3869
Alaska	Ron Martindale	(907) 266-1593
Utah	Eric Chang	(801) 965-4284
Minnesota	Mike Schadegg	(612) 797-3126
Connecticut	John Vivari	(860) 594-2712

An example of the capabilities of this software program, four figures are presented that illustrate how it may be used in an analysis. Figure 1 illustrates the intersection menu and allows the user to select the type of identifier, the date range desired, the primary and cross streets, and add any filters that may be desired.

Figure 2 is a graphical plot of all of the accidents that occurred at the intersection identified in figure one. The accidents are for the date range specified and are located on the leg of the intersection where they occurred. If there is a wrong data element that located a given accident in the wrong location, the user can click and drag the specific accident to the proper location. Each type of accident has a separate symbol.

A filter has been applied to the accident history show in Figure 2. All of the left turn accidents for the date range specified in figure one are shown. The location within the intersection is representative of the direction the vehicles were traveling at the time of the accident.

A question that is likely to be asked would be “What time of the day do these left turn accident occur?” Figure 4 shows a graphical representation of the time they occurred. The transportation official that is evaluating this intersection can now start to concentrate on the time period from 11:00 AM until 4:00 PM to see if there are reasons for the pattern shown.

IOWA DOT RECENT ACTIVITIES

The Iowa DOT has been in the process of implementing the Intersection Magic software program into their analysis process. DOT staff has used the program to generate collision diagrams and comparing to hand drawn diagrams for the same locations. The staff has found that the accuracy is about 95% when comparing Intersection Magic to the hand drawn diagrams. The next step for the Iowa DOT is to provide this product to field offices for evaluation and for implementation. The final step will be for local governments to use the software for the evaluations required when an application for safety funds is generated.

ACKNOWLEDGMENTS

CTRE would like to thank all the participating vendors and developers for supplying their collision diagram packages for evaluation. In addition, we want to thank the following Iowa DOT employees who gave of their time and efforts to participate in this study and provide guidance: Ian MacGillivray, Fred Walker, John Nervig, Bill Bielefeldt, and Susan Fultz.

1998 TRANSPORTATION CONFERENCE PROCEEDINGS

An Assessment of Emergency Response Vehicle Pre-Deployment Using GIS Identification of High-Accident Density Locations

BRADLEY M. ESTOCHEN, TIM STRAUSS, AND REGINALD R. SOULEYRETTE

On average, over 15,000 crashes occur daily in the United States, most of which involve only damage to property. However, for crashes that involve injury, response time is critical. Ideally, if a specific accident location could be accurately predicted beforehand, an emergency vehicle could be dispatched before the accident occurs. Although this is not possible, the identification of high-accident locations using historical crash trends might allow the positioning of response vehicles so as to minimize the expected travel time to incidents. This paper summarizes research to identify the potential benefits of emergency vehicle pre-deployment. This research uses point location data from Iowa's Accident Location Analysis System (ALAS) for the period 1990-1995 to generate maps of high accident locations for Des Moines, Iowa. The emergency medical service facilities will be used in conjunction with the roadway network to determine the service areas of the existing facilities. The recommended location/allocation of emergency response vehicles will be determined using the power of a Geographic Information System (GIS). The network analysis capabilities of GIS will be used to estimate the response times of strategically placed emergency vehicles; this will be compared to actual response times. Key words: response time, crash analysis, GIS, EMS.

INTRODUCTION

It is well known that EMS response time is critical in traffic crashes involving injury. Emergency medical service planning involves decisions from both strategic and tactical viewpoints. Strategic decisions involve the location and number of vehicles to attain overall system goals. Tactical decisions involve responses to situations that arise given a fixed number of vehicles (*1*).

Response time is crucial to the survival of many traffic accident victims. In a potentially fatal accident, the time of starting an intravenous drip (IV) is often imperative to the survival of the victim (*2*). Additional basic life support may also be needed soon after the crash to increase the chance of survival (*3*). All factors that might increase response time are matter of concern. Some variables related to response time include land variables, such as differences in travel time and terrain between rural and urban settings, and road variables, such as variations in traffic flow related to time of day, weather, and congestion (*3*).

Various government policies have encouraged the combination of hospital accident and emergency departments into centralized units, responsible for large geographic areas. While this provides an improved quality of medical care, travel time to a crash scene is often compromised for those not near these centralized locations (*4*). According to Brown, there is a positive association between ambulance delay and the ratio of fatal to serious injuries. This study found an increased mortality rate in counties that had a low population density, further suggesting a link between elevated response time and prognosis (*5*). Numerous other studies have also demonstrated the relationship between decreases in response time and corresponding decreases in mortality (*1*).

RURAL AND URBAN RESPONSES

Emergency calls generate different approaches to providing service for individuals in need. This is evident in analyzing the difference in response hierarchy in rural and urban settings. Response characteristics vary by geographic area. Rural areas provide service to a large geographic area with limited resources (*6*). Optimal strategic location of facilities is a key element in meeting rural emergency response needs (*6*).

With higher population densities, urban areas may consolidate services from a limited number of locations. Simultaneous requests for service are also more commonplace in urban settings. Therefore, the positioning of such facilities should provide efficient service to a diverse area.

TYPICAL EMS SYSTEM

The typical EMS service response is shown in Figure 1. A communication center operator receives a request for service, usually by phone or two-way radio. The operator makes an initial screening to determine if an ambulance should be dispatched and what particular response code should be used (if needed). Next, the dispatcher assesses the geographical location and the availability of the fleet based on the particular assignment hierarchy established by the management. It is a generally accepted rule to send the closest unit to the incident. At this point the appropriate crew is

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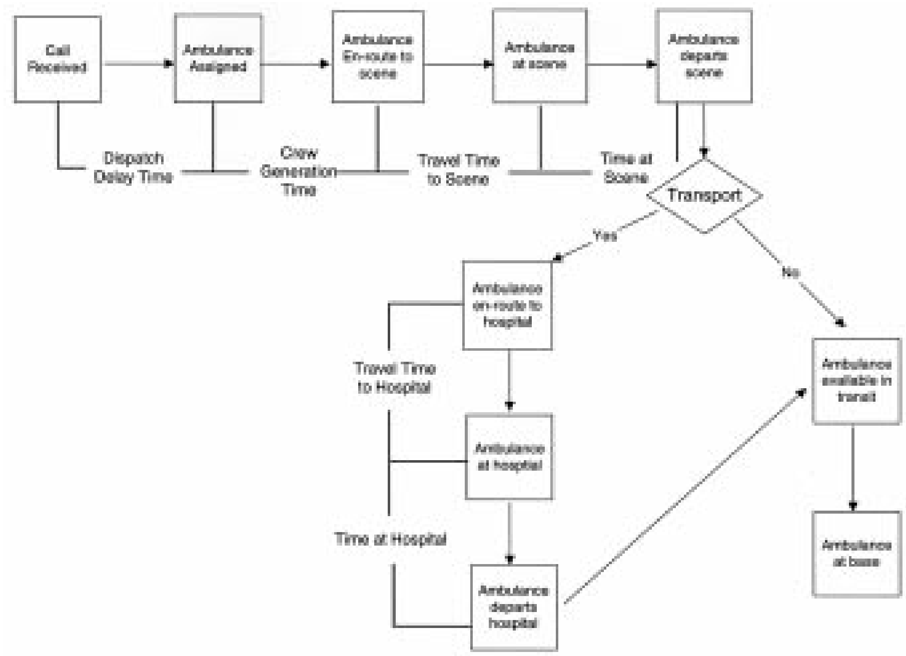

FIGURE 1 The EMS process.

assigned to respond. The time elapsed during this phase is referred to as the dispatch delay.

The response unit gathers any necessary equipment that may not be resident on the vehicle and proceeds to the specified location. The interval between the time the response unit receives a call to the time the response vehicle is in motion is called crew generation time (however, some operations include this as part of the dispatch delay).

The travel time is the time elapsed from the initial movement of the vehicle until the dispatcher is notified of the arrival. The total system response time is represented by the time interval between the call notification and the arrival on the scene. However, it is common for EMS systems to view response time delay without the dispatch delay, not taking into consideration the factors external to the mobilization process.

Dispatchers usually decide if an ambulance needs to be deployed. This does not occur in all cases. At times the necessity of an ambulance cannot be determined until notification from police or other responding units arrive at the scene. Often a short time is spent on the scene because the service is not needed or the victim refuses medical attention or transportation. When this occurs the crew departs and heads back to the base. During the time in transit back to base the unit is available to respond to another call if needed; however this does not occur often and when it does it is usually in large cities during peak hours (7).

In cases where medical attention is deemed necessary, the crew determines the appropriate medical facility that can best address the needs of the individual. After arriving at the hospital, the patient is transferred to the hospital staff. Before returning to duty, the EMS crew spends additional time completing reports and cleaning and resupplying the ambulance unit. After these steps have been completed the crew returns to their base location. The total service time is the time elapsed from the reception of the initial call to the unit's departure from the hospital.

SERVICE GOALS OF EMS SYSTEMS

EMS service has several goals in urban areas; the level of service sought by EMS planners is to have 95% of the daily demand for service be within 10 minutes (*8*). However, this level of service is not usually attained. Louisville, Kentucky, for example, responded to only 84% of the calls within the 10-minute specification (1). In general a reachable goal of an EMS station is responding to 90% of all calls in less than 10 minutes (*9*).

The level of service in a given region depends on the spatial distribution of EMS facilities as compared to the spatial pattern of the demand. EMS facilities located near areas with high demand, like crash densities, while not neglecting other areas can provide lower response times and improved levels of service and final outcomes. A geographic information system (GIS) can be used to identify existing EMS service areas, to compare these areas with traffic crash patterns, and to generate strategies to improve EMS services. The next section uses EMS facility and traffic crash data for the City of Des Moines and Polk County to illustrate the use of GIS in assessing existing EMS response patterns and the potential impacts of alternative locations of EMS facilities.

GIS-ALAS

The Iowa Department of Transportation, with assistance from the Center for Transportation Research and Education at Iowa State University, has developed a Geographic Information System Acci-

FIGURE 2 Graphical representation of crash locations.

dent Location and Analysis System (GIS-ALAS). The system, an extension of Iowa's DOS based PC-ALAS, includes the location of all crashes on all roads in the state for the last ten years, approximately 700,000 accidents. It provides spatial displays of accidents and allows the database to be queried and analyzed.

This can provide information on the locations of high-density accidents. Each accident contains up to three files (accident, driver, and injury) that provide information about the accidents that occurred. The database contains injury severity and time-of-day information. Data is also available about the roadway, including average daily traffic (ADT), lane width, length of each segment, and speed limit. These files have been created allowing a GIS to provide a spatial graphic of crash locations.

Historical trends can provide information on high accident density locations. Historical information can be beneficial to a number of users including engineers, planners, and law enforcement and emergency medical service personnel. Emergency medical services can use this historical accident information to determine the characteristics relevant to vehicle crashes involving injuries. The remainder of this paper will examine high-density locations in Des Moines, Iowa and compare response service areas from static facilities, exploring the potential for pre-dispatching in areas of need for locations that do not have adequate service.

LOCATING HIGH ACCIDENT AREAS

In GIS-ALAS, the roadway is represented as a link-node system. There are approximately 226,000 node locations throughout the State. Each accident is located with respect to two nodes, the reference and direction nodes, and the corresponding distance from the reference node. This system provides easy representation of accident density by visually displaying the accident locations with respect to the nodes.

The database was queried to determine the number of accidents associated with each reference node. The total number of accidents associated with each reference node was calculated and incorporated with the accident database. This allows the number of accidents at each reference node to be visually displayed within the GIS, the larger the dot the increased frequency of accidents. (Figure 2).

Inspection of the Des Moines metropolitan area identified accident locations based on five years of crash data (1991-1995). The objective of locating these crashes was to determine if pre-dispatching emergency vehicles would provide benefits to crash victims by reducing EMS response time. The time intervals were used to determine the peak period of crash occurrence for the metropolitan areas. These crashes were then examined based upon time of day. The a.m. peak period for crashes was determined to be from 6:30 – 9:00, while the afternoon peak period was 3:30 – 6:30. These intervals had 13% and 26%, respectively, of the total crashes within the metropolitan area for the five years of data provided.

The determination of the high crash locations provides useful information regarding the geographic characteristics of the crashes. However, other information can also be obtained including the number of injuries and fatalities. As stated, at times the necessity of an ambulance unit cannot be determined until an EMS unit is present at the scene (*7*); therefore, accidents that involve non-injuries are also important to EMS providers, as well as the crash victims.

SERVICE AREAS

Using the analytical capabilities of the GIS, the response times from each EMS location can be computed and graphically displayed to determine if service to particular areas is satisfactory. The locations of the EMS facilities were added to the GIS using data from the Iowa Department of Public Health (Figure 3). The roadway

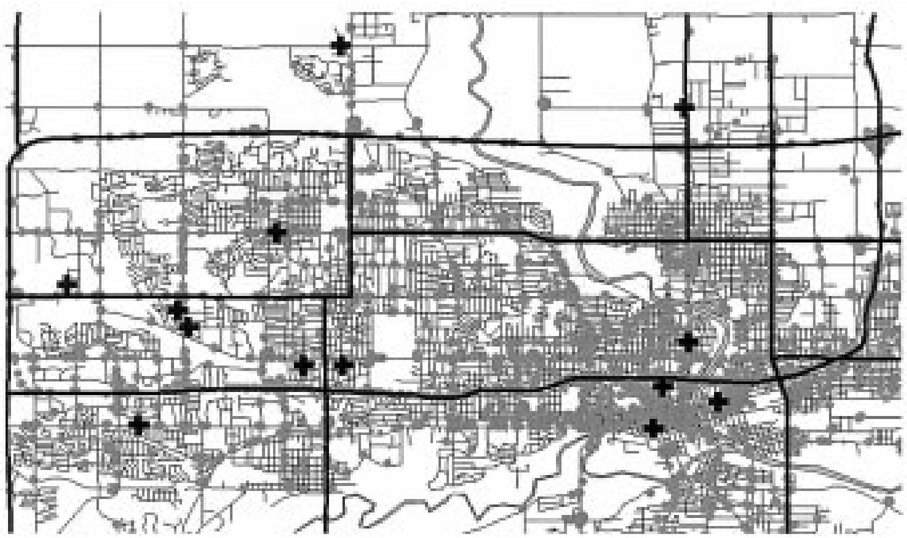

FIGURE 3 EMS locations with respect to traffic crashes within the Des Moines metro area.

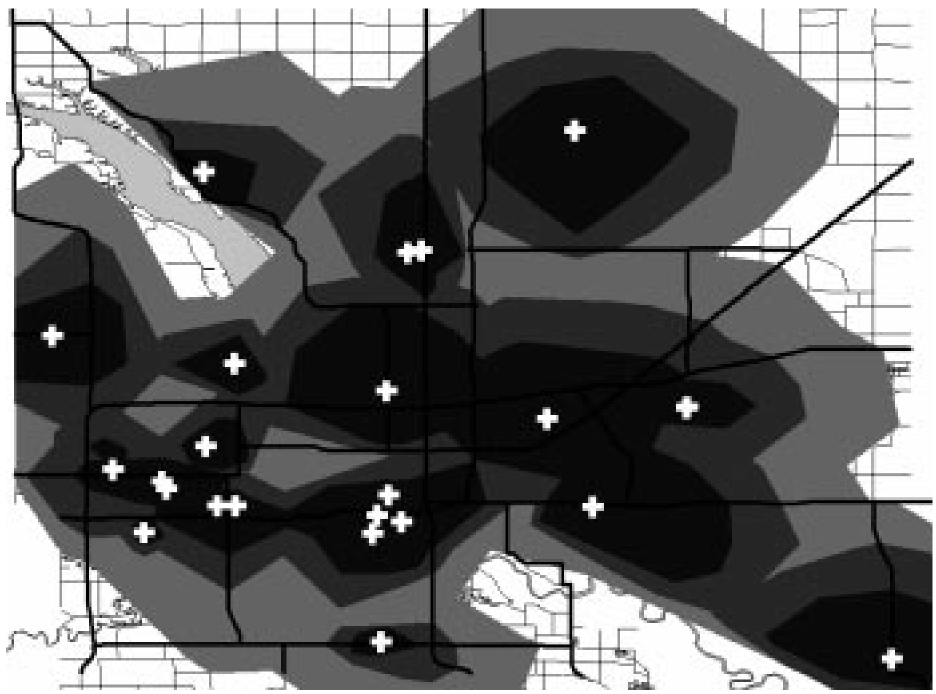

FIGURE 4 EMS travel ranges (5, 7, and 10 minutes).

files were obtained from the Iowa Department of Transportation Office of Cartography; attribute information was also used to enhance the data set.

Segment length and speed limit are used to compute a travel time for road links. This provides each individual segment with a travel time value; however, this value is only an approximation. This value does not take into consideration time delays associated with traffic control and traffic congestion.

RESULTS

Travel time areas from each of the facilities were computed and are displayed in Figure 4. The rings around the facilities indicate the areas, and accidents, that can be reached within 5, 7, and 10 minutes. Several facilities are located in clusters in close proximity to each other, especially in downtown Des Moines and in the western suburbs. Neighborhoods near these clusters are located within over-

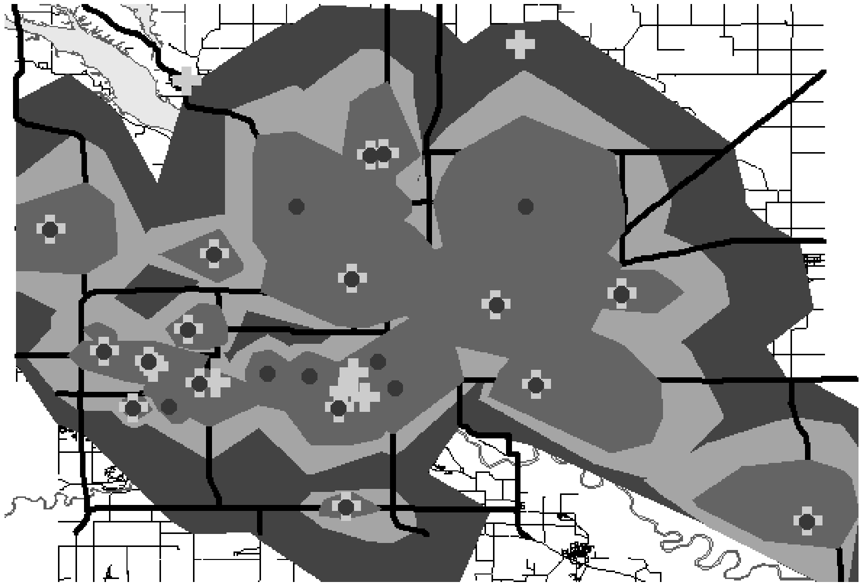

FIGURE 5 EMS pre-dispatched locations and the service areas.

Table 1 Response During AM Peak Period (Current EMS Locations)

	Total Crashes	%	Fatal Crashes	%	Injury Crashes	%	Number Injured	%
5 Minutes	4356	56.4	4	33.3	1429	55.3	1902	54.9
7 Minutes	6330	81.9	9	75	2076	80.3	2776	80.1
10 Min.	7320	94.7	11	91.7	2411	93.2	3241	93.5
Total	7729		12		2586		3467	

Table 2 Response During PM Peak Periods (Current EMS Locations)

	Total Crashes	%	Fatal Crashes	%	Injury Crashes	%	Number Injured	%
5 Minutes	9245	58.7	18	50	3186	53.7	5027	57.9
7 Minutes	13124	83.3	24	66.7	4932	83.2	7132	82.2
10 Min.	15131	96.1	33	91.7	5714	96.4	8329	96
Total	15752		36		5928		8679	

lapping 5-minute service areas of several facilities. In contrast, areas in the county's periphery tend to be in the 10-minute service area or beyond. (The results may be partially affected by data limitations related to the connectivity and characteristics of the road network.) Service area maps like Figure 4 can be used to provide an initial indication of countywide EMS coverage and identification of potentially underserved areas.

These response areas were broken further down into one-minute intervals to estimate the number of crashes occurring during each response-time interval. This was then used to determine the average response time for the entire region. The estimated average response time, given the locations of 1991-1995 crashes and EMS facilities, for the a.m. and p.m. peak periods is 4.91 and 4.92 minutes, respectively, for the Des Moines metropolitan area.

The locations of the facilities were moved in GIS to simulate possible changes in EMS activities, such as pre-dispatching vehicles to areas with high crash densities. These areas were based upon historical crash patterns identified using GIS-ALAS. (The differences in the facility locations can be found in Figure 5.) The resulting changes in service areas and response times were computed and compared to the current situation.

The overall response time for the pre-dispatched vehicles was 0.4 minutes (24 seconds) lower for both the a.m. and p.m. periods. In practice, this difference may be insignificant for most crash outcomes but critical for others. The change in facility location also resulted in an increased percentage of crashes reached within the 5-minute threshold — 56.4% before the change vs. 63.2% after for a.m. crashes, and 58.7% vs. 65.8% for p.m. crashes, about a 7% improvement for both time periods (see Tables 1-4). This benefit decreases at the 7-minute threshold (about a 2% improvement), and at the 10-minute threshold the percentage of crashes reached is roughly the same for both sets of EMS facility locations. Figure 6 further illustrates the shift toward shorter response times. Data for injury crashes and the number injured show similar patterns. (The

Table 3 Response During AM Peak Periods (With Pre-dispatched Locations)

	Total Crashes	%	Fatal Crashes	%	Injury Crashes	%	Number Injured	%
5 Minutes	4884	63.2	4	33.3	1604	62	2138	61.7
7 Minutes	6482	83.8	8	66.7	2111	81.6	2823	81.4
10 Min.	7302	94.5	11	91.7	2412	93.3	3243	93.5
Total	7729		12		2586		3467	

Table 4 Response During PM Peak Periods (With Pre-dispatched Locations)

	Total Crashes	%	Fatal Crashes	%	Injury Crashes	%	Number Injured	%
5 Minutes	10360	65.8	22	61.1	3901	65.8	5557	64
7 Minutes	13421	85.2	24	75	5040	85.2	7312	84.2
10 Min.	15129	96	32	88.9	5712	96.3	8315	95.8
Total	15752		36		5928		8679	

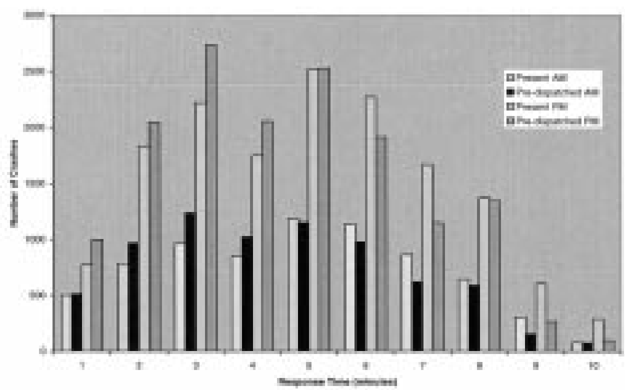

FIGURE 6 Changes in AM response times due to pre-dispatching units.

number of fatalities was too small to provide significant comparisons.)

CONCLUSIONS

The ability of EMS personnel to quickly reach crash sites is a critical determinant of final crash outcome. Response times are closely linked to the locations of EMS facilities in relation to the locations of crashes. GIS can be used to assess existing service areas, to identify potentially underserved areas, and to evaluate the implications of potential changes in EMS systems. The case study presented here illustrates that changing the location of EMS services, such as through the pre-deployment of vehicles, can result in improved response times. The benefits, in terms of improved outcomes, and costs associated with this strategy is an issue for future research.

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Economic Growth, Property Valuation Change, and Transportation Investments

DAVID SWENSON, LIESL EATHINGTON, AND DANIEL OTTO

Transportation related infrastructure investment is closely linked to economic performance of a region. In assessing the role of transportation investment in economic growth, it is important to refine the spatial detail of analysis and to use effective data analysis techniques. As an alternative to county level data, property tax data that is collected annually can be used to evaluate economic growth patterns at a detailed level. Because of overlapping jurisdictions 1,500 separate parcels can be identified within the study region. This paper uses GIS techniques to examine patterns of economic change in a nine-county central Iowa region between 1987 and 1995 and to relate these changes to investments in transportation infrastructure during the same time period. Specifically, the economic change will include property valuation change for residential, commercial, industrial and agricultural classes of property. A mapping of valuation changes in this region depicts the major shifts in capital wealth. Overall, a growing metro core city is ringed by high growth urbanized clusters at the fringe. Nonurbanized cities located within 20 miles of the metro core also grew strongly. Residential values declined in the core city while growing dramatically in the suburban fringes. Commercial values grew strongly in the metro core and even more so in the western suburbs. Transportation infrastructure investments are shown to coincide with areas rapid population and valuation growth. Key words: GIS, property valuation, transportation, spatial.

INTRODUCTION

Transportation related infrastructure investment is closely linked to economic performance of a region. Planners and economic development professionals debate whether these transportation investments occur in response to growth pressures or whether the investments lead or stimulate new economic growth. As local economies change and grow, additional land is typically required for housing and for commercial purposes. As a result, property values increase and a conversion of land to higher valued uses occurs. The change in demand for land and properties becomes capitalized into property values and becomes a reliable indicator of regional economic performance. Transportation investments can also affect these values by providing greater access for commuters and commercial activities.

This report demonstrates a measure of economic change within a metropolitan region and its extended labor and trade market area involving disaggregation at levels finer than the traditional county level reporting that is common of most U.S. and state secondary data sources. This disaggregation helps us to better understand some of the spatial dynamics of economic growth and decline in a Midwestern metropolitan region and compare these dynamics to modern urban development concepts. Instead of relying on the annual counts of employment and income (or population) in a county, our data set is comprised of net growth in capital values within specific taxing districts or fractions of taxing districts as they are enumerated.

When studying economic change, aggregation at the county level can be relatively sufficient for most purposes. Elemental mapping programs can be used to track and display gross changes and patterns of change over time. While useful, the county level of analysis (or subcounty level every ten years) does not allow us to get some of the more intriguing intra-regional transformations that may be occurring as a result of central place dynamics, structural changes in an area's economy, or major infrastructure investments. Comparisons of different patterns of growth for a large sample of central places are another method of isolating the potential range of responses to change and the spatial distribution of those responses *(1)*. These, too, are limited to census year comparisons.

The proper documentation of economic or social change at subcounty levels and the accounting of the change using GIS techniques and standard urban hierarchy designations can lead to meaningful analytic outcomes. There are over 8,200 unique taxing districts in Iowa for which property data are collected annually. These districts include cities, counties, townships, school districts, community colleges, and special purpose districts. Some boundaries coincide, others do not. Almost all of the cities and all of the townships are contained within the confines of county boundaries. School districts, community college districts, and special districts do not coincide with county or township boundaries. It is because of these overlaps in boundaries that we get 8,200 jurisdictional fragments out of the state's 1,500 local units of government.

Figure 1 displays a nine-county region of central Iowa, the locus of our study. Within the counties we can see the township boundaries and the municipal boundaries (gray-shaded). The large cluster in the center is the Des Moines MSA. This level of analysis gives us a lot of spatial detail provided we know information at the place (city) level or the township level (remainder of county subdivision). In all, there are 263 distinct spatial/governmental units measurable within the region. We have also overlaid the school district boundaries that serve the region. Each intersection or intrusion produces jurisdictional chunks of space that are geographically identifiable. These parcels are irregularly shaped and now number 744. These parcels are relatively standard over time and change only when school districts consolidate or municipalities annex unincorporated territory.

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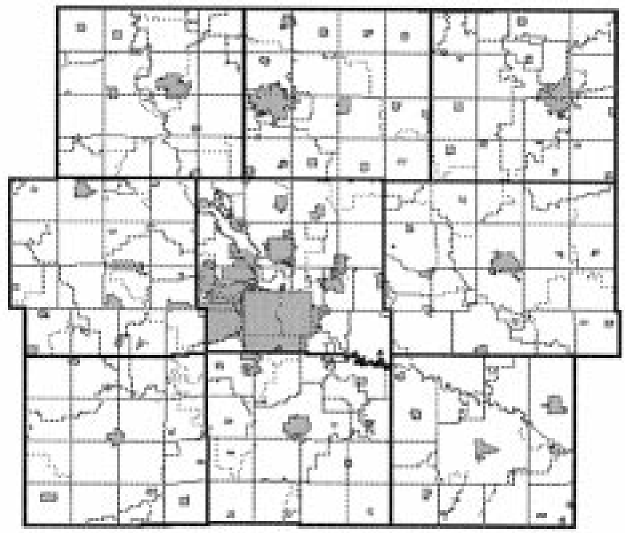

FIGURE 1 Study area: school districts, municipalities, townships and counties.

These data were originally compiled for three purposes: the first was to demonstrate the potential of analyzing economic information at levels that were more refined than the typical county-level analysis (Salge, J. *unpublished data*). The second original purpose of the data was for the purpose of isolating relative changes in area property tax capacities as they relate to taxable property value changes *(2)*. Another reason for compiling the data this finely was for the purpose of testing whether a jurisdictional model of capital valuation change can be linked to infrastructure levels and social characteristics within the jurisdictions measured (Swenson, D. and L. Eathington. *unpublished data).* In particular, we were looking for a reasonable alternative to hedonic price approaches to measuring the likely demand for and relationship of infrastructure investment to capitalized regional growth.

REGIONAL CHANGE AND THE URBAN HIERARCHY

The nine-county region that we study contains a core metropolitan county (Polk County – Des Moines City) along with two adjacent metropolitan counties that comprise the entire Polk County MSA. The remaining counties that are adjacent to Polk County are urban counties whose populations range from 13,000 to 75,000. The region is roughly in the center of the state, and Polk County/Des Moines is the intersection of two interstate highways. Economic growth in the region has been strong over the past decade. Since 1984, nonfarm employment has grown by 50,000 jobs. Overall, net job growth in the state has accrued disproportionately to the state's eight MSAs and six other regional trade centers. The state's remaining 85 counties have experienced only minor levels of job growth and economic expansion. The unevenness of growth is such that nearly a half of the state's non-metropolitan counties continued to lose population since 1990 while all of the metropolitan and metro-adjacent counties have grown. Recognizing that job growth in these areas nearly always outstrips population growth rates, we are left with the conclusion that greater and greater numbers of these jobs appear to be filled by incommuters from surrounding counties.

In all, we can arrive at a set of relatively safe conclusions about the nature and character of economic growth in Iowa when we compare our counties by level of urbanization and adjacency to metropolitan areas as would be the case if we used USDA ERS rural typology measures (often called "Beale" or rural-urban continuum codes). The growth is, of course, limited to county summaries and tells us nothing about the nature and character of change at levels finer than the county level.

For our purposes, the dynamics of change within the counties in the nine-county region are more interesting and, perhaps, much more telling. Although GIS helps us to identify the patterns of change, it is also instructive to reliably classify the jurisdictions under study in order to look for meaningful average experiences. Here we have a relatively large metropolitan core city that is surrounded by several adjacent urbanized cities. Just outside of the urbanized core are the remaining cities within the three counties that comprise the MSA. Lying outside of the metro county cities but still within the metro counties are the unincorporated areas represented as townships or township remainders that either are adjacent to the metro core city or are not. Farther out we have the remaining cities in the counties adjacent to the metro counties and the unincorporated remaining space in those counties. Accordingly, we have an urban hierarchy beginning at the urban core and moving outward by level of incorporation, physical adjacency to the core metro city, and presence within the metropolitan counties.

We next calculated net shifts in property values in each of the parcels comparing 1987 with 1993. Shifts for each class of property in each jurisdictional fragment were calculated using shift-share methods. The actual competitive share or change in value for any jurisdiction in the state is for each class of property is:

Competitive Property Share =
Property Value in 1987 *
(Jurisdictional Percentage Change – Statewide Percentage Change)

Though this approach is usually used for employment change over time, it works well for other measures of economic activity in a jurisdiction *(2,3).* The shift-share or net of shares method allows us to net out the statewide growth characteristics of each property class to identify the potential shifts in capital values geographically in the state. We are, in effect, measuring the relative changes or position each jurisdiction finds itself regarding valuation change over the time periods. When we use this method, the sum of all changes for all jurisdictions statewide equals zero. All net growth is the amount in excess of the expected value (the state rate of growth). Those that did not grow at the state rate lost ground to other places. Using this method within a coherent economic region allows us to calculate the net capital flows or competitive positions among regions and to isolate, in our case, changes within our region.

Table 1 is a compilation of the shifts in property values on a per square mile basis. Using per square mile allows us to standardize the change; we get an intuitively more clear idea of the rate of wealth generated in the measured areas. The rate of residential capital wealth growth per square mile was over twice as great as the per square mile losses in the metro core. The very same pattern emerged in the non-urbanized communities: their per square mile gains were nearly twice as great as the per square mile losses suffered in the unincorporated adjacent areas.

TABLE 1 Net Shifts in Property Values Per Square Mile by Urbanization Level, 1987-1993

	Residential	Commercial	Industrial	Total
Metro Core	(5,245,470)	1,919,086	(168,391)	2,540,723
Urbanized Adjacent	11,208,754	5,926,829	(454,175)	26,015,628
Urban Non-Adjacent	3,427,962	1,591,499	(194,021)	6,780,076
Non-Urbanized Adjacent	(1,846,823)	(901,116)	(490,022)	(2,181,692)
Non-Urbanized Non-Adjacent	118,450	(5,618)	2,067	61,058
Other Urban (nonmetro)	(22,358)	(339,234)	218,804	3,657,134
Other Non-Urban (nonmetro)	16,717	(3,927)	(2,014)	(78,187)
9 County Total	74,077	71,313	(16,408)	351,400

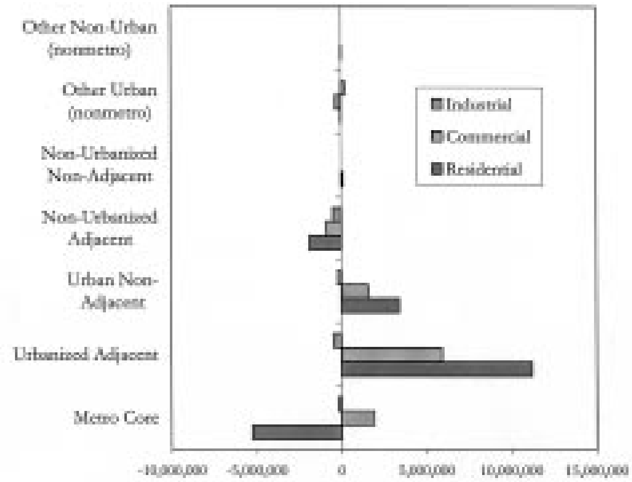

FIGURE 2 Shift in capital values per square mile, 1987 to 1993.

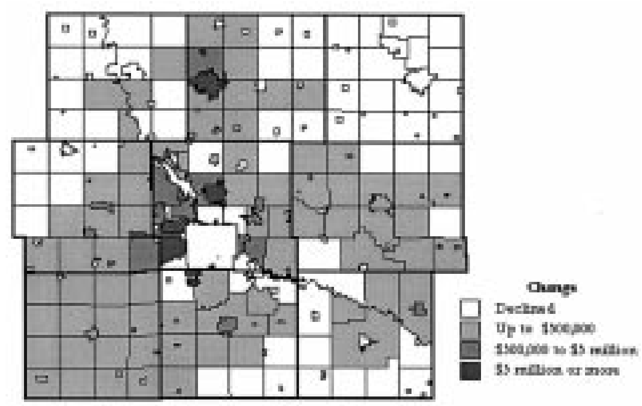

FIGURE 3 Net shift in residential values per square mile, 1987-1993.

Strong commercial property value per square mile gains are also evident and demonstrate that, on a per square mile basis, commercial growth in the urbanized adjacent cities was over three times greater than in the metro core and more than three and a half times greater than in the urban non-adjacent metro cities. It is interesting to note that the $.9 million in commercial decline per square mile in the non-urbanized adjacent territories continues the relative losses that those properties posted, especially when compared to neighboring incorporated places. Industrial losses per square mile indicate that the average gain per square mile in the nonmetro urban areas are meaningful in that their gains are in excess of the metro core or the urban non-adjacent declines. Their gains per square mile, however, are at a rate much lower than the rate of erosion posted by the urbanized adjacent and non-urbanized adjacent parcels in the region.

On a total valuation basis, the rate of growth in the urbanized adjacent communities was over ten times greater than in the metro core and almost four times greater than in the urban nonadjacent communities. The differences reflect not only the total dollar amount invested but also greater investment densities in the suburban fringe.

Figure 2 displays the weighted net shifts per square mile of territory in the jurisdictional parcels for residential, commercial, and industrial properties. These properties most adequately represent the net demand and valuation of housing along with changes and concentrations of business and industrial activity.

Ideas of central place assume capital concentration in these growth centers. We also know that there are both centralization and decentralization forces at work, especially in cities of some size. Suburban flight is a direct response to the disamenities and diseconomies of urbanization. Commercial concentrations, on the other hand, continue to accrue to growth centers, and significant portions of urban vitality are centered in service, retail, and wholesale trade expansions. For much of the Midwest in recent years there has been an increase in manufacturing employment, especially relative to the nation's overall decline in manufacturing jobs *(4)*. These jobs have located in areas usually outside of metropolitan places. If we move from the graph (Figure 2) back to Table 2 we note that total property shares (per square mile) were significantly greater in the non-metro cities than in the metro core. Whether this is an indication of stability at the fringe or just the overwhelming influence of a handful of central places at the fringe remains to be seen.

DISPERSION OF PROPERTY SHIFTS

Figures 3 through 6 depict the major shifts in capital wealth in the region on a per square mile basis. In Figure 3 we see that residential declines were evident in the core metro city and in the northwest, northeast, and southeast corners of the region. Residential growth in the southwestern portion is attributable to the area's rolling hills and river valleys giving rise to the development of pricey country estates. Otherwise, communities with the greatest growth rate are either urbanized communities or the remaining non-adjacent communities and townships within the three metro counties. Six of these areas posted housing valuation growth in excess of $5 million per square mile. This same high level of growth is also

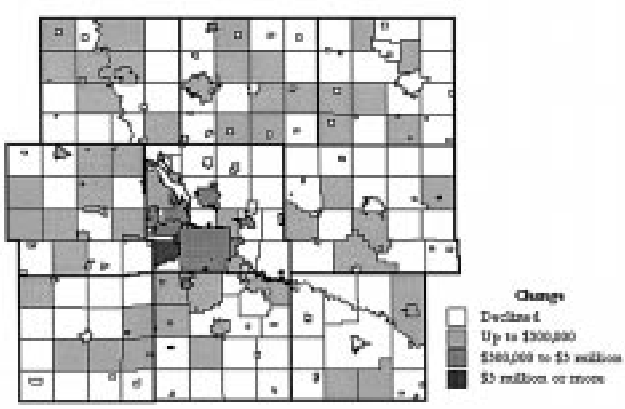

FIGURE 4 Net shift in commercial values per square mile, 1987-1993.

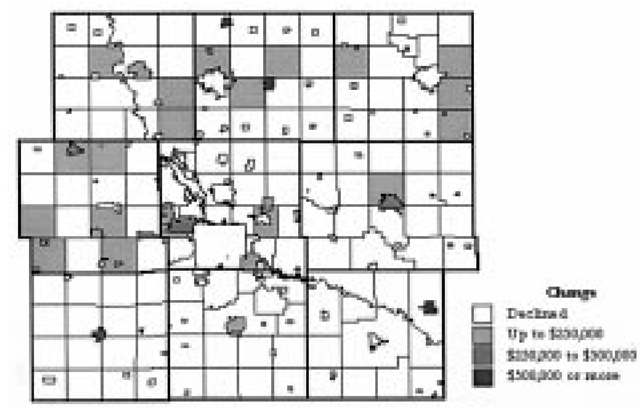

FIGURE 5 Net shift in industrial values per square mile, 1987-1993.

noted in a nonmetro community (Ames) north of the metro, which also serves as a significant central place in its own right. Strong growth is also evident in many of its surrounding townships, especially those bordering major highways. Despite their growth, the modal experience of nonmetro cities was a slight decline. These declines are most notable as distance from the metro core increases and as size of the community declines.

Figure 4 demonstrates the pattern of commercial valuation change in the region. Here we see definite concentrations of growth within the metro core, and even more so in the city's western suburbs. We also see that where residential growth was generally widespread, the growth in commercial values is more associated with population densities and at list fringe access to the metro core. The vast majority of nonmetro county cities posted erosions in commercial wealth with the exception of the two large cities in the counties to the north and northwest of the metro. Growth exceeded $5 million in commercial value per square mile in one western suburb, and from $.5 million to $5 million in the remaining metro urbanized and most of the nonurbanized communities.

Figure 5 shows the powerful decentralizing forces at work in manufacturing sectors. Rotations outward from the metro core are evident with minor growth in urbanized cities and outlying communities, and even stronger rates of growth in many very small and quite distant communities. Some of this growth is also evident in the township remainders as small firms opt to locate outside of small communities where possible. (It is important to note that the scale of change is different in this figure.)

In Figure 6 we see the distribution of gross valuation changes. After sorting out the losses attributable to farmland value erosions along with widespread declines in the value of utility properties (telecommunications and railroads, primarily), we can ascertain the regions of strong growth. A growing metro core city is ringed by high growth urbanized centers. Nonurbanized cities located within 20 miles of the metro core also grew strongly. Much lower incidences of growth are evident in the remaining incorporated and unincorporated territories. Except for the central communities in the northeast and the southeast counties, the only outlying growth activity (including the two other metropolitan counties) is in their larger city/county seat.

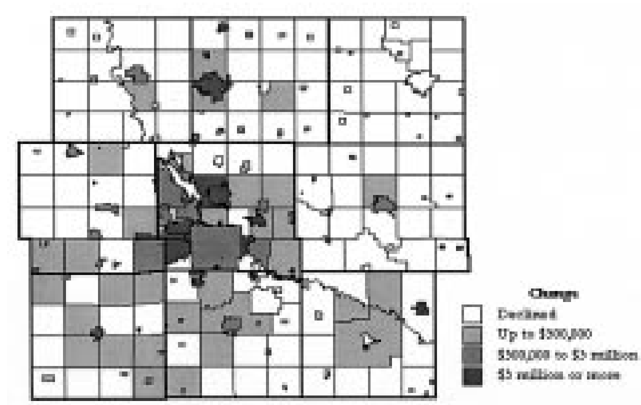

FIGURE 6 Net shift in gross values per square mile, 1987-1993.

CORRELATES WITH TRANSPORTATION INVESTMENT

One of the expected uses of this data set was to identify the spatial coincidence of transportation with residential and commercial growth with an eye toward developing simulation models that would help to predict expected property growth given roadway growth. With the use of TIGER/File data on roads in Iowa we were able to isolate all of the road segments within our study region and calculate their lengths for 1987 and 1993 by our jurisdictional units of analysis. Knowing the lengths also allowed us to calculate additions in roads, as measured by lengths, and changes in the number of interchange segments, as measured by access nodes to limited access highways. We excluded gravel and dirt roadways as unimproved roads that would in the main be negatively correlated with improved roads.

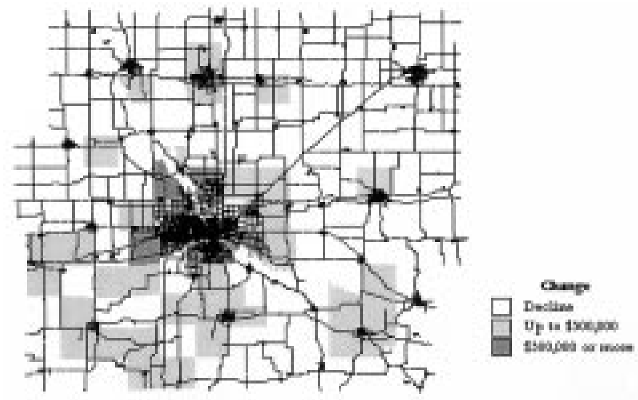

FIGURE 7 Transportation infrastructure and gross valuation shifts per square mile, 1987-1993.

For each jurisdiction, we were able to construct measures of total roadway length, changes in length, density of roadway development, along with the number and change in the number of interchanges. We could then be able to compare these infrastructure measures to the amount of final residential and commercial property values along with the changes in values measured by net shifts per square mile.

Figure 7 displays the improved roadway grid for our region of study. This roadway has been laid over a map of the change in gross values per square mile. One notices, of course, the overall density and concentration of roads and interchanges in the metro core. These overall densities are fairly indicative of the region's population densities. There is, of course, strong correlation between population and population density with total roads and roadway densities along with total property values and growth per square mile in values.

CONCLUSIONS

Our analysis confirmed several patterns of change indicative of usual central place dynamics and others that warrant additional research. There is a definite erosion of core city residential worth in clear favor of fringe, urbanized are investment. Commercial growth is strong and total capital shifts accruing to the metro core, however, are very small in relationship to those accruing to the adjacent urbanized fringe. Outlying communities within the metro counties and beyond are also posting gains, but we must carefully scrutinize the extent to which those gains are part of regional growth dynamics or local factors. Infrastructure investments are shown to be coincident with areas of rapid economic growth. More detailed information is needed to address the issues of timing of impacts from these infrastructure investments.

Future research also needs to collect more detail on other capital investments made by state and local government and to refine the property valuation system to the parcel level based on market transactions. Other applications could examine valuation patterns to other applications such as telecommunications investments.

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Transportation and Urban Form: A Case Study of the Des Moines Metropolitan Area

CHUNLIN CAO, TIM STRAUSS, REGINALD R. SOULEYRETTE, AND R. DUANE SHINN

It is well known that urban form is highly correlated with the evolution of transportation systems. In order to develop planning tools that are responsive to the complicated interaction between transportation and land use, it is helpful to identify the typical characteristics of the development of urban form. The relationship between transportation, land use and urban form is complicated by feedback relationships. Understanding the three-way dynamics between land supply, urban development and travel demand is a first step toward understanding these aspects in the transportation planning process. Using the Des Moines metropolitan area as a case study, this paper begins to examine and quantify some of these relationships. The purpose of this paper is to examine how urban form accommodates transportation systems and vice versa at a conceptual level. This is accomplished through a review of past studies on urban form and transportation from both design and transportation planning literature. The case study demonstrates the relationships between urban population density, travel pattern, residential and commercial distribution, using an interface between MapInfo and Tranplan. Following the characterization provided in the case study, suggestions for further strengthening of the relationship between land use and transportation in travel planning models are recommended. Key words: transportation, urban form, the Des Moines metropolitan area.

INTRODUCTION

It is well known that city development patterns are highly correlated with the evolution of transportation systems (*1*). In order to develop planning tools that are more responsive to the complicated interaction between transportation and land use, it is first necessary to identify the typical characteristics of the development of urban form (*2*). The relationship between transportation, land use and urban form is complicated by the fact that change in any one of these aspects will also result in changes in the other two. The ability to predict and display the three-way dynamics between the level of land supply, urban development and travel demand would be helpful to decision-makers (*3*). In this paper, a case study of the Des Moines metropolitan area is presented to demonstrate and shed light on some of these relationships.

The purpose of this paper is to examine how urban form accommodates transportation systems and vice versa at a conceptual level, and to contribute some additional understanding of the transportation and urban form literature. The historical development of the Des Moines area is reviewed to see how urban form is accommodated by transportation evolution, and the conventional transportation modeling process is reviewed to see how urban form is implied in the transportation modeling process. Seven spatial measurements are used to quantify urban form in Des Moines and its existing transportation network.

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TRANSPORTATION AND URBAN FORM

The Historical Development of the Des Moines Area

A review of the historical development of Des Moines area is given to provide a pictorial description of how transportation and urban form have accommodated each other (*4*). Table 1 summarizes the different phases of Des Moines' development, its corresponding transportation systems and transportation eras. Des Moines, like other cities, benefited from both transportation modal evolution (from ferry to automobiles) and transportation network evolution. The transportation system can be considered an expression of urban spatial pattern during the historical development of the city.

The Conventional Transportation Modeling Process

Transportation models are computerized procedures for estimating changes in travel patterns in response to changes in development. The development of an open space into a shopping center, or changing demographics often require changes in the transportation network (*8*). Table 2 summarizes how urban form is implied in the conventional, sequential transportation modeling process of trip generation, trip distribution, modal split and traffic assignment (*9*).

TABLE 1 Des Moines' Development Stages and Transportation Systems

Phases	Urban Development Characteristics	Transportation Systems (*4*,*5*)	Transportation Era (*6*,*7*)
1673 - 1838	No political entity; prairie land	Raft, ferry, horseback	Walking-Horsecar Era
1839 - 1857	More residential housing, predominant farm production	Stage (horse and wagon) or river	Walking-Horsecar Era
1858 - 1881	More social life, political byplay, increased population and housing	Horse-car, railroad, steamboats, dirt roads	Walking-Horsecar Era
1882 - 1910	Substantial urban development with increased population and housing, manufacturing, business and open space	Electric-trolley, railroads, recreation automobiles, paved roads	Walking-Horsecar Era Electric Streetcar Era
1911 - 1937	Substantial city sprawl, focusing on real estate, finance	Electric-trolley, railroads, more automobiles, better roads	Electric Streetcar Era Recreation Automobiles
1938 - 1947	Economic indicators went up, heavy investment on highway	Trackless-trolley, automobiles, bridges, highways	Recreation Automobiles
1948 - 1967	Urban renewal, more affordable and better housing, congestion	Diesel motor coach for transit, automobile, bridges, interstate highways, one-way streets	Freeway Era
1968 - 1978	City government closer to people, start comprehensive planning	Transit service deteriorating, increased automobiles use, airport, completion of highway system	Freeway Era
1979 - present	Continued urban sprawl, urban form becomes an issue with current growth scenario	I-235 becomes a priority; sustainable transportation system becomes an issue	Freeway Era

TABLE 2 Urban Form Is Implied in the Transportation Modeling Process

Transportation Modeling Process	Elements Implied in Modeling Process (*10*)	System Components in Urban Spatial Structure	Criteria for Urban Spatial Form (*11*)
Trip Generation	Land Use, Socio-economic, Demographic	Land use	Density pattern, homogeneity, concentricity
Trip Distribution	Travel time impedance, Personal preference, Socio-economic	Land use	Connectivity, density patterns and density gradient
Modal Split	Transportation policy, Auto ownership, Residential density, Income, Distance from CBD, Service	Land use, principles of urban structure, external determinants, the geographic extent and limits of the urban area	Density pattern, density gradient, sectorality
Traffic Assignment	Geometrics, Transportation network, Capacity of the roadway	Geographic extent and limits, the transportation network and its capacity	Directionality, connectivity

Seven Selected Spatial Measurements

Seven spatial criteria are presented here: homogeneity, directionality, connectivity, density pattern, density gradient, concentricity and sectorality (*11*) to measure the urban form of the Des Moines metropolitan area.

Homogeneity measures diversity of land uses or social areas. Density pattern reflects the shape and intensity of population density. Connectivity is used to demonstrate the roadway network.

Density gradient measures change in population density from the CBD of the City of Des Moines. Concentricity is defined as the degree to which uses and activities are organized zonally around the center. Sectorality is defined as the degree to which uses and activities are organized sectorally around the city center. In the Des Moines metropolitan area, population, housing and trips are highly concentrated at the central part of the metropolitan area. Center displacement and "the standard ellipse" are used to assess the directionality of the metropolitan area. Studies of the CBD and

TABLE 3 Seven Spatial Measurements and Des Moines Urban Form

Spatial Measurements	Urban Form Description
Homogeneity	Most commercial areas are located along major highways (I-235 and I-35), primary roads (14th St.) or at the junction of highways. High-density housing surrounds the downtown core. Industrial use is located along rivers and railroads.
Density pattern	High density population can be found in every city in the metropolitan area except the City of Cumming. Most of the population lives in the central part of the metropolitan area.
Connectivity	I-35 and I-80 run through the northern part of the metropolitan area. I-235 cuts through the heart of the metropolitan area. There are several east-west primary roads going through the entire area, while only 14th St. goes through the entire metropolitan area in the north-south direction.
Density gradient	The density gradient measures population density from the CBD of the City of Des Moines. It shows that the central part of the metropolitan area has the highest population density. Logarithmic regression seems more accurate than linear regression to reflect the overall density distribution from the CBD of the City of Des Moines.
Concentricity	Population and housing are highly concentrated in the central part of the metropolitan area. Work trips are also highly concentrated in the CBD of the City of Des Moines.
Directionality	The work trips are more west-east oriented than north-south. The difference between them is about 22%. The centers of the CBD of the City of Des Moines, the City of Des Moines itself and the entire metropolitan area are different. Development tends to shift to the geometric centers of a city or a region.
Sectorality	Sectorality is analyzed through CBD and corridor studies. The total population in the CBD and corridors is 15.5% of the total population in the metropolitan area, while the trip productions and attractions are 19.2% and 33.4% respectively. Activities are located in the CBD and along corridors.

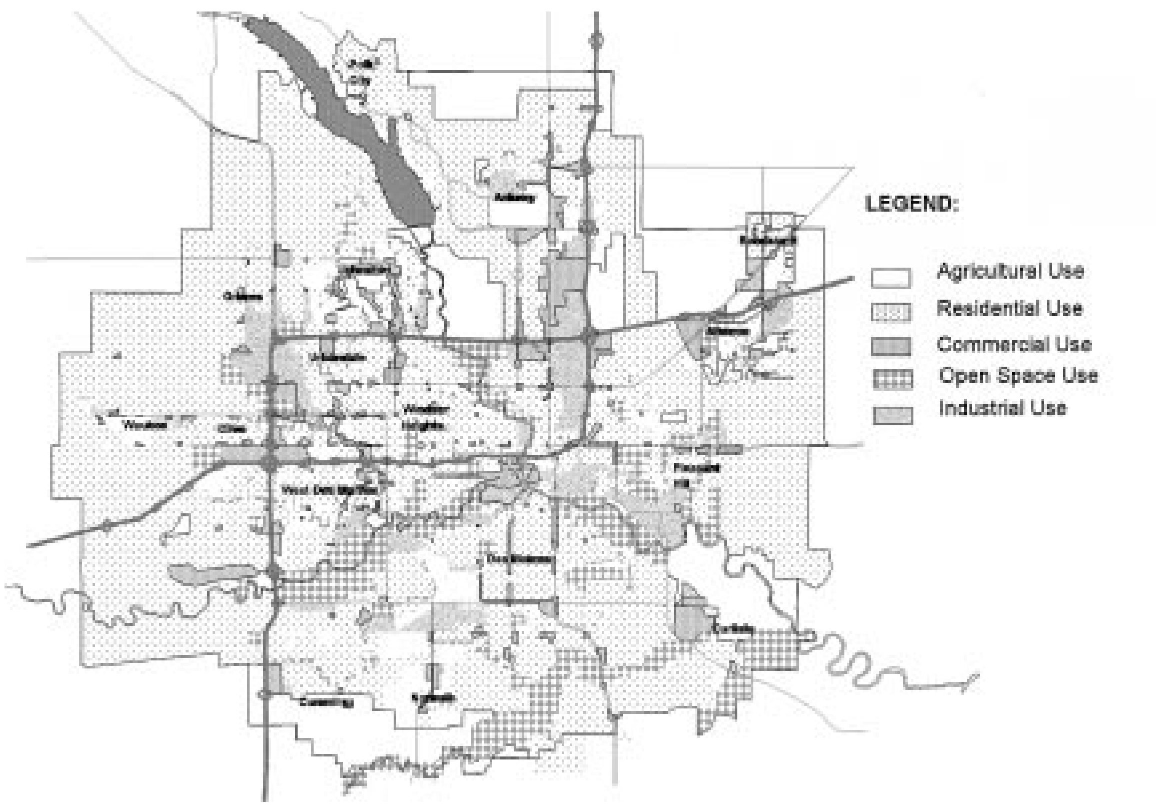

FIGURE 1 Different social areas and the transportation network in the Des Moines metropolitan area.

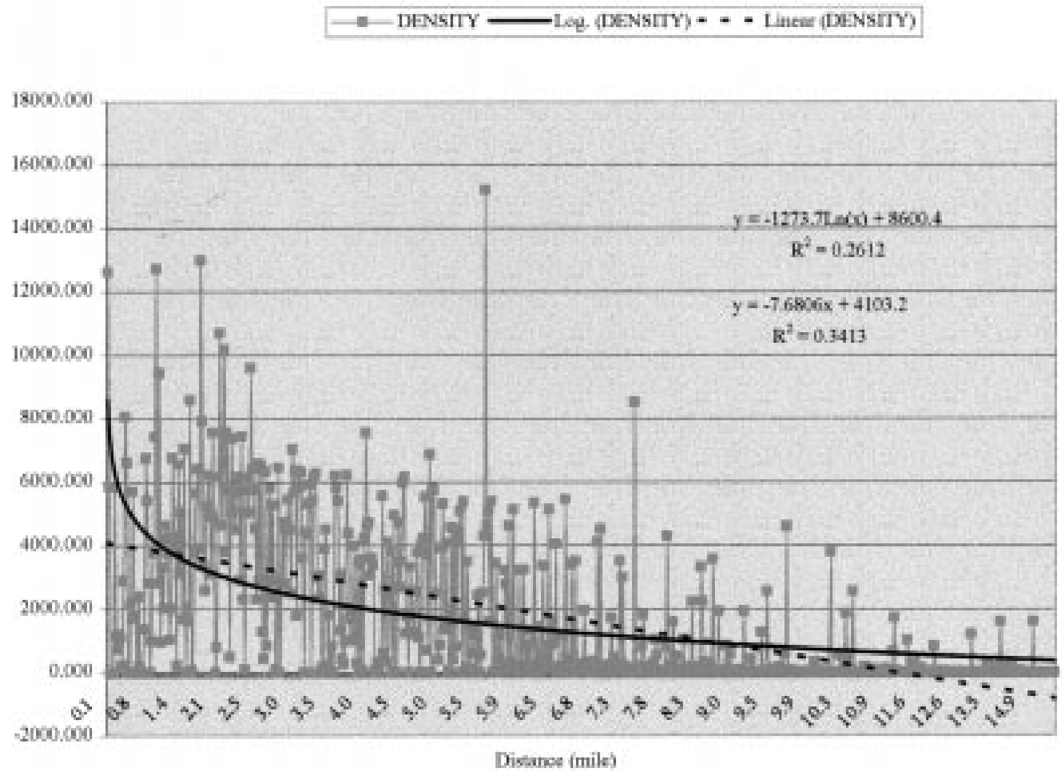

FIGURE 2 Population density gradient for the Des Moines metropolitan area.

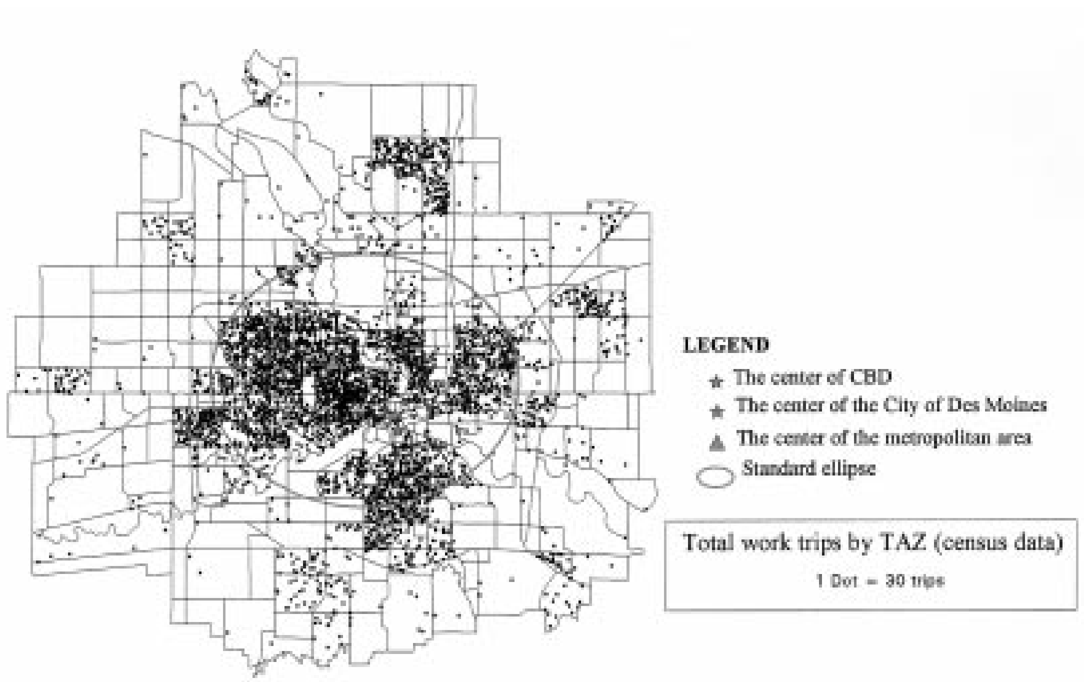

FIGURE 3 The standard ellipse and the centers of CDB, the City of Des Moines and the metropolitan area.

TABLE 4 The Number of Housing Units

Different Period	City of Des Moines	The Other 15 Cities
Housing built before 1939	26,718	2,279
Housing built in 1940s	10,032	1,556
Housing built in 1950s	15,026	5,144
Housing built in 1960s	11,718	9,345
Housing built in 1970s	12,312	14,682
Housing built in 1980s	7,553	14,336
Total number of housing units	83,289	47,352

TABLE 5 The Relative Locations of CBD, the City and Metropolitan Area Centers

Points	Coordinates (feet) X (feet)	Y(feet)	Distance from CBD Center (feet)	Distance from CBD Center (mile)	Direction From the CBD
CBD center	1966932.7	578564.0	-----	-----	-----
City center	1967512.8	575599.5	3020.72	0.57	South south east
Metro area center	1953970.0	589624.9	17040.40	3.23	Northwest

TABLE 6 The Summarized Results of the CBD and Corridor Study

Study Area	Land Use Description	Population	Trip Productions	Trip Attractions
CBD	General commercial uses	3,347 (0.94%)	57,493 (4.9%)	202,563 (15.7%)
E14th St. Corridor	Mixed uses with residential, commercial and open space uses	12,558 (3.54%)	35,994 (3.1%)	32,161 (2.5%)
Intersection I-235 and I-35 to intersection I-235 and NE 14th St.	Mixed residential uses with commercial uses	29,180 (8.2%)	102,709 (8.7%)	162,688 (12.6%)
Intersection I-235 and NE 14th St. to intersection I-235 and I-35	Mixed uses with more industrial uses	9,902 (2.8%)	29,701 (2.5%)	32,897 (2.6%)
Total Percentage		15.48%	19.2%	33.4%

corridor are used to examine the sectorality of the metropolitan area. Table 3 summarizes the results of the urban form of the Des Moines metropolitan area using these seven measurements.

Figure 1 shows different social areas and the transportation network in the Des Moines metropolitan area. Figure 2 shows the population density gradient. Figure 3 shows the center displacement and standard ellipse. Table 4 provides data on the housing units in the City of Des Moines and the other fifteen cities of the metropolitan area. Table 5 shows the relative locations of the centers of the CBD, the City of Des Moines and the metropolitan area. Table 6 shows the summarized results of the CBD and corridor study.

RESULTS

From the historical review of the development of the Des Moines area, the conventional transportation modeling process, and the seven spatial measurements, the results can be summarized as following:

1. The population density gradient shows that the central part of the Des Moines metropolitan area has the highest population density. Even though some other metropolitan area cities have been growing rapidly, they have not influenced the central city function of the City of Des Moines. The City of Des Moines is still the focal point for employment (approximately 60,000 employees per day in downtown) and population in the metropolitan area. Des Moines is the civic and cultural capital of the metropolitan area. The other cities are chiefly bedroom communities, even though they are beginning to show significant commercial and retail development. This development largely follows interstate highway development along I-235, I-80 and I-35. Some of the outlying cities may develop into special function cities, but are likely to retain the status of "satellites" of the City of Des Moines. The urban pattern of the Des Moines metropolitan area is radial in terms of trip attractions.

2. The location of the CBD of the City of Des Moines was largely influenced by the Raccoon River and the Des Moines River. Development in the City of Des Moines has since shifted southward. Within the metropolitan area, new developments are located northwest of the geometric center of the metropolitan area, which is close to the cities of Urbandale, Clive, West Des Moines and Windsor Heights. It is assumed that new developments tend to shift to the geometric center of a city or a region to overcome the friction of distance or space. People tend to make tradeoffs between transportation costs and land values. It is suggested that when examining the development trend for a city or a region, the geometric center or its vicinity may be the first measure that should be considered.
3. Based on census data, bicycle trips comprise only 0.2% of total work trips while walk trips make up 3.2% and bus trips are 2.9%. Future urban design could consider more use of these modes to make Des Moines more walkable and more bicycle and transit friendly.

RECOMMENDATIONS FOR FUTURE STUDY

When the urban form of one area was examined in this paper, no consideration was given to the organizational level, policy or decision-making levels of urban form. As both transportation planning and urban form changes are dynamic processes (*12*), the following items are recommended to better understand the relationship between transportation and urban form.

1. Explore the use of measures which explicitly account for urban form into transportation models, e.g, if a logit model is used for modal split, can say spatial measurements of the standard ellipse be applied to better calibrate the model?
2. Examine different transportation modes for effects on urban form.
3. Measure sewers, water and utility lines as determinants of urban form.
4. Examining the influence of zoning ordinances, building codes, other local policies, and national transportation policies that may shape urban form and the transportation network,
5. Assess the importance of life style as a determinant of urban form.
6. Measure more cities with different urban patterns and cities of different sizes to determine the statistical relationship between density gradient, urban pattern and transportation networks.

Transportation is surely not the only determinant shaping urban form. Other factors like those listed above are sometimes critical. However, realizing that not all transportation networks and investments are rational, truly understanding the relationship between transportation and urban form helps us make more rational decisions. The purposes of research on transportation and urban form are to provide better transportation networks and make more efficient investments on the existing network, to provide the residents a better place to live and work, and to make a more livable and sustainable city based on the existing transportation network.

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FORETELL™: Providing Integrated Weather Information Services Across the Upper Midwest

PETER DAVIES, DEAN DEETER, AND CLARE E. BLAND

Weather has an enormous effect on travel and road conditions. Drifting snow, ice, fog, and gusty winds are some of the weather events that contribute to the deaths of more than 1150 U.S. and Canadian highway users every winter. Adverse conditions cut surface friction, impact highway capacities and reduce accessibility, damaging industry and rural economies alike. To help address these difficulties, various agencies within the U.S. and Canada are currently working together in the FORETELL™ project to develop an integrated strategy for providing detailed, up-to-the-minute weather information dissemination services, bringing to fruition many of the products of meteorological and transportation research initiatives. This paper explores the possibilities for innovative approaches to providing road and weather condition information to diverse users, and suggests ways to broaden the traditional views of information provision through wide-ranging public / private partnerships. Details are provided of the FORETELL™ initiative, currently funded through FHWA research and development, which involves the Departments of Transportation of Iowa, Missouri, and Wisconsin. Many other public and private sector agencies are also participating.
Key words: road conditions, weather, maintenance, safety, travel information service.

INTRODUCTION

Weather has an enormous effect on travel and road conditions. Drifting snow, ice, fog, and gusty winds are some of the weather events that—at least in part—kill more than 1150 U.S. and Canadian highway users every winter. Adverse conditions cut surface friction, impact highway capacities and reduce accessibility, hurting industry and rural economies alike. Over $2 billion is spent on snow and ice control each year in North America (*1*). Despite this, estimates indicate:

- between 25 and 35 percent of inter-urban incidents occur during adverse weather conditions;
- accidents increase during adverse weather by factors of between two and five;
- U.S. injury accidents alone exceed 65,000 due to adverse road conditions.

Adverse conditions increase travel times, boost drivers' anxiety and stress levels, and slash safety margins. The 1994 Minnesota Guidestar Rural Scoping Study (*2*) found that travelers' most-desired information would detail road and weather conditions. Other weather information needs analyses, such as the Strategic Highway Research Program (SHRP) Storm Monitoring and Communications Project, as well as studies conducted by intelligent transportation systems (ITS) groups such as ENTERPRISE, and road-weather programs like Aurora, show that:

- The most important need is to pull together all of the existing weather and road condition data sources. An effective road-weather information system should not be limited to one or a few data sources, but should integrate information from all available locations including the National Weather Service (NWS), Environment Canada (EC) and private "value-added" weather services, as well as Road Weather Information System (RWIS) field stations.
- As weather honors neither political boundaries nor institutional divisions, agencies and firms must share information to improve their weather tracking and monitoring capabilities. A key concept is to gain synergy through seamless data exchange and joint development efforts.
- Road-weather information must be timely and accurate. Detailed, location-specific forecasts and nowcasts are essential. The numerous advances in meteorology, computing power and telecommunications have created a situation in which forecasters have more to offer transportation operators and users than ever before.
- Users require multiple means of information access, including radio, TV, conventional and cellular phone, pager, and Internet. Information must be available on-demand and should also be available in tailored packages "pushed" to key users when threshold conditions are exceeded.
- Users need flexible information presentation formats. The information is too complex to load all the data on everyone. Effective, proven decision support systems are vital for user buy-in.
- An open system architecture is essential. Due to the complexity of advanced weather systems and ITS, and the number of fast-evolving information systems that should be linked together, it is imperative that standardized communications protocols, such as the NTCIP "Environmental Sensor Stations (ESS)" initiative, be utilized from the start.

THE FORETELL™ INITIATIVE

FORETELL™'s state, research agency and private sector partners come from diverse ITS and meteorological backgrounds, and yet

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share a common resolve to see detailed road and weather information become an everyday commercial reality. Iowa DOT is the lead public sector agency. Iowa has made major investments in RWIS, has equipped 39 rest areas with real-time weather information systems, has led the ITS ENTERPRISE Program, and is active in the road-weather Aurora pooled fund study. Castle Rock Services (CRS) is the private sector lead. CRS already runs two ITS Service Centers in public-private partnership with Virginia DOT, suburban and rural centers whose immediate success has already done much to validate the Service Center concept.

FORETELL™'s public sector partners have already shown their trust in intelligent weather systems by investing significant public funds into the sophisticated infrastructure behind this advanced weather systems/ITS approach. The initial FORETELL™ deployment will cover five states in the Mississippi Valley region, plus western Ontario. This heartland includes plains, forests, lakes and rolling terrain. FORETELL™ touches on major metropolitan areas including Chicago, Milwaukee, Minneapolis/St. Paul, Kansas City and St. Louis. This large and diverse geographic setting allows users' needs for and responses to the integrated surface transportation weather information system to be explored.

By initially investing $4,450,000 (including $1.3 million in FHWA funding) in this first, regional phase of the FORETELL™ road and weather information system partnership, the program aims to improve safety and ease winter travel, leading to more cost effective roadway maintenance, operations and environmental benefits. Additionally, FORETELL™ will serve a whole range of public, quasi-public, private and traveler client groups, eventually it is hoped generating self-sustaining cash flows and subsequent returns on investments for long-term private and public partners.

In addition to this direct funding, through its partnership with the National Oceanic and Atmospheric Administration (NOAA), FORETELL™ is building on the committed $4 billion National Weather Service modernization and on the $370M annual Federal meteorological research budget. For example, FORETELL™ partner Forecast Systems Laboratory (FSL)'s Federal funding alone exceeds $20M/year, a commitment spent entirely on intelligent weather systems development. FSL's work on precise forecasting and advanced decision support systems, executed over more than 10 years, will reach one of its full applications through FORETELL™.

Technical Approach

The basis of FORETELL™'s system design is the integration of Intelligent Weather Systems (IWS) and Intelligent Transportation Systems (ITS) technologies. FORETELL™ builds upon the National Weather Service modernization and restructuring (MAR) and on equivalent investments being made by Environment Canada. NWS atmospheric models are being linked to pavement conditions based on proven Swedish and UK approaches, providing a starting point for further, innovative developments in road condition forecasting. These data fusion, road condition prediction and multimedia information dissemination activities are focused in new ITS Service Centers, as the core of a much broader "basket" of commercially-viable ITS services.

In more detail, the FORETELL™ design concept brings together all available weather data sources, including satellites, radars, wind profilers, airborne platforms (i.e. commercial jets), and surface sites including those of NWS, DOD, aviation and conventional RWIS stations in the FORETELL™ states. To these FORETELL™ is adding road-vehicle mounted mobile data platforms. Rigorous quality control algorithms will be applied to automatically filter out suspect sensor data and support effective maintenance/ recalibration programs. State-of-the-art mesoscale models will support fine resolution (10 km grid in the U.S.; 12 km grid in Canada) nowcasts and forecasts. NOAA's proven decision support systems will allow complex data outputs to be understood and acted upon by highway maintenance staff, commercial vehicle operations (CVO) dispatchers and individual travelers.

European practice already links site-specific pavement condition forecasts based on energy balance models to mesoscale atmospheric model outputs. FORETELL™ is refining these approaches and applying them at much higher levels of detail, initially for 1 km road segments and later down to 1-5 meter microscale resolution and hourly intervals. Mobile ESS (Environmental Sensor Station) data collection platforms gathering real-time thermal profiles and air temperature/dewpoint/windspeed data will give input to these microscale models. In later deployments, a fish-eye lens digital camera with a global positioning system (GPS) and on-board processing will provide the database for innovative solar gain pavement temperature projections. Crosswinds measurement from the mobile platforms will address snow drifting, linking local eddies to the broader pattern of mesoscale surface winds. Eventually a visibility nowcaster will process air and surface temperatures, dew points and related factors.

ITS Service Centers are providing the focus for data fusion, road condition forecasting and information dissemination activities. Planned dissemination media include commercial radio and TV, conventional and cellular telephone, pager, and Internet, as well as existing and upcoming ITS traveler information systems such as highway advisory radio, dynamic message signs and (when commercially viable) AM phase modulation and/or FM subcarrier systems. Data packages will be tailored to specific market segment requirements, with basic safety information provided free at the point of use. Value-added services will include those funded by advertising and sponsorship, as well as subscription services. Both on-demand and information "push" technologies will be supported.

Current Systems in Place

Because FORETELL™ builds upon the entire set of weather service, aviation, defense, agriculture and emergency management hydrometeorological data collection systems, as well as on conventional RWIS, it enjoys the benefits of a frankly awesome multi-billion dollar array of currently deployed systems. More than 95% of FORETELL™'s data comes initially from sources other than RWIS. Recent advances in radar, automated weather observing systems, super speed computers, satellite, sophisticated information processing and communications systems provide the foundations of FORETELL™'s warnings and forecasts. Systems being harnessed for FORETELL™ include:

- the Next Generation Weather Radar (NEXRAD or WSR-88D);
- the NWS Automated Surface Observation System (ASOS);
- the National Centers for Environmental Prediction (NCEP);
- weather satellites, developed and operated by NOAA's National Environmental Satellite, Data and Information Service (NESDIS);
- the NWS Advanced Weather Interactive Processing System (AWIPS)
- numerous advanced communications networks, including the

NWS one-way, point-to-multipoint satellite broadcast service called NOAAPORT;

- the 118 NWS Weather Forecast Offices (WFOs); and
- the 170 RWIS sites currently operational in FORETELL™'s initial area.

The FORETELL™ Project Structure

In Module 1, the National Center for Environmental Prediction (NCEP), National Weather Service Local Forecast Offices, NOAA's Forecast Systems Laboratory (FSL), and the Canadian Environment Service (AES) are working to provide sensor data and the output of mesoscale atmospheric models for the FORETELL™ states.

In Canada this work is being undertaken by AES in the Canadian Meteorological Centre (CMC) in Dorval, Quebec. Environment Canada is cooperating closely with NOAA agencies in FORETELL™'s Module 1, pursuing similar approaches in leading edge meso-scale numerical modeling at the Canadian Meteorological Centre to provide high resolution model outputs north of the border. Selected EC Ontario Regional Centres will integrate their data collection, forecasting and dissemination activities with those developed in National Weather Service LFOs and in ITS Service Centers in the United States.

In Module 2, Castle Rock Services (CRS) is working with State DOTs to establish Rural ITS Service Centers in the Mississippi Valley. The Service Centers will serve as data fusion points, and to operate the road condition forecast models. In Canada, EC will undertake similar activities in Dorval and its regional centers in Ontario, linking up with the Ministry of Transportation of Ontario (MTO) and its private sector winter maintenance contractors. Finally, FORETELL™ members are also working on several innovative technology insertions.

In Module 3, Castle Rock Services will further develop the service centers deployed in Module 2 to eventually support a "basket" of ITS User Services in conformance with the National ITS Architecture, ultimately providing a broad and diverse funding base with maximum self-sustaining potential. Private sector investments are funding the set up and operation of equipment and systems to perform information dissemination to the traveling public. State and Federal funding will support information dissemination to State DOTs and to other public and quasi-public agencies. Acting as public-private partnerships, with joint staffing and shared facilities, the Service Centers will serve clients drawn from the diverse "core" groups of users.

In Module 4, FORETELL™ is building on 30 American Mobile Satellite Company (AMSC) GPS/satellite communications on-board computer units already developed within Iowa DOT's CVO operational test (the on-board automated mileage & stateline crossing system for apportioning commercial vehicle fuel taxes and mileage or AMASCOT). IA/DOT is working with AMSC to add pavement temperature sensors (as currently used in thermal mapping), air temperature sensors, relative humidity and wind speed sensors. The Iowa/Minnesota/Michigan maintenance concept vehicle will thereby be enhanced to serve as a mobile platform to collect and transmit back road surface and atmospheric weather information. Other Rockwell units are being deployed on regular snowplows, state patrol cruisers, rural buses, etc., to fully evaluate the mobile platform concept across the FORETELL™ states.

FORETELL™ is integrating the mobile weather monitoring equipment with AMASCOT GPS/satellite communications on-board computers. Resulting measured air and pavement temperatures, wind speeds and directions will be logged on board maintenance vehicles but also radioed back to the ITS Service Centers in real time. Wind speed will provide a direct input into snow drift forecasting. Air temperatures and winds will be made available to the NWS, as ACARS aircraft observations are already provided today. Pavement temperature data will be used at the Rural ITS Service Centers to enhance site-specific pavement condition forecasts.

CONCLUSIONS

In conclusion, FORETELL™'s benefits are expected to include:

- an ability to "grow" multi-regional and national/North American systems, based on initial lessons learned regarding the impacts of region-wide weather nowcasts and forecasts on the behavior, productivity and safety of travelers and maintenance personnel;
- public-public and public-private partnerships that result in improved linkages to, and coordination between, FORETELL™ members, affiliates, and their existing and planned RWIS and other related ITS deployments;
- an integrated road and weather system that crosses state and national borders, allowing the comparison of United States and Canadian weather forecasting models, and the seamless distribution of tailored weather and road information to FORETELL™ users;
- the flexibility to evolve and further innovate as the system grows across the North American Continent, based upon fast-developing technologies, changing customer applications and increased expectations; and
- the promise of cutting costs and substantially benefiting the environment by increasing the levels of forecast detail, eventually to hourly and micro (1-5 meter) resolution, thereby enabling maintenance concept vehicles to greatly reduce the amount of chemical required to anti-ice and de-ice roadways.

Finally, FORETELL™ showcases how federal agencies can effectively team with the private sector to meet a common goal: to improve and modernize weather information forecasting and delivery systems. In the case of the Department of Commerce, this illustrates the vital role of the National Weather Service in partnering with key players across a multi-billion dollar user market segment—transportation. In the case of U.S. DOT, the program will lead to an integrated, seamless system that meets highway operators' and users' needs for clear and accurate road and weather information.

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Case Study: The Branson Travel and Recreational Information Program (TRIP)

TIM GARRETT

INTRODUCTION

Ten years ago, no one would have predicted that the sleepy little town of Branson, Missouri would explode to become a major vacation destination for millions of U.S. and foreign tourists. Located in the heart of the Ozark Mountains, Branson holds claim to three mountain-fed lakes surrounded by millions of acres of unspoiled wilderness. The pristine natural surroundings of Branson have always attracted large numbers of visitors interested in the camping, hiking, fishing and water sport opportunities offered.

However, in the last decade Branson has become known as the "live entertainment capital of the world." Branson now boasts 38 music theaters with over 52,000 seats – that's 9,000 more than in the theaters on New York's Broadway. The shows in Branson are home to today's greatest performing artists, making it a mecca for 6 million visitors a year. In fact, its permanent population of just 4,400 swells to more than 40,000 during peak tourist season. This growth in such a short period is extraordinary. But along with this surge come a few growing pains.

Most international visitors and many long-distance visitors fly into the nearby city of Springfield, where they can rent cars or join motor coach tours to the Branson area. However, by far the largest proportion of visitors drive, charter a motor coach for the trip, or buy a package tour that includes motor coach transportation directly from large cities within a 600-mile radius. The principal markets for the Branson/Tri-Lakes Region are, in fact, St. Louis, Kansas City, Dallas, Oklahoma City, Tulsa, Wichita, Omaha, Minneapolis, Chicago, Indianapolis, Memphis, Little Rock and Des Moines.

Traffic congestion is an unfortunate bi-product of such enormous popularity. The roadway system in and around Branson consists of mostly two-lane facilities, with the majority of traffic entering or leaving the city on US Highway 65 and Missouri Route 76. Route 76 becomes a three-lane facility through Branson, where it is known as "76 Country Music Boulevard," contains the majority of theaters and attractions in Branson and carries more than 37,000 vehicles per day during much of July and August. Throughout the summer months and into fall, the level of service (LOS) is frequently at F.

There are significant efforts underway to improve this situation. Road improvements and construction are going on for routes into and out of Branson. As an additional immediate measure, a number of alternative routes which parallel Route 76 have been constructed within Branson to ease congestion on 76. These alternative routes have been promoted by the City and MoDOT through various marketing efforts and are known as the red, blue and yellow routes with color-coded signs directing traffic to use them. The red, blue and yellow routes are designed to allow visitors to "come in the back door" for a more direct route to their lodging, attraction or restaurant. There have also been plans developed for a regional transit system and local Branson trolleys to carry patrons along Country Music Boulevard.

Intelligent Transportation Systems (ITS), particularly those providing rural travel and recreational information to visitors, are another critical element in the toolbox of solutions for the transportation needs of the Branson/Tri-Lakes Region. The Branson Travel and Recreational Information Program (TRIP), one of the first federally sponsored rural ATIS operational tests in the U.S., is a program designed to inform tourists of route options, keep them updated on current traffic conditions, provide a single source repository of tourist and recreational information on Branson and the surrounding Tri-Lakes Area. The system builds upon many of the technologies which have been tested in urban settings. However, with the layout of the streets within Branson and the fact that the city is a tourist town, the approach to implementation has differed greatly to traditional urban ATIS projects.

BRANSON TRIP MISSION AND GOALS

The Branson TRIP goal is to collect, coordinate, and disseminate multi-modal travel and tourism information to the broad variety of visitors to the region, in order to improve mobility, reduce congestion, enhance economic development, and improve the overall experience of visitors to the Branson/Tri-Lakes Region. In effect, the Branson TRIP system will be utilized to enhance and better utilize the current and future transportation infrastructure through the identification of traffic conditions and dissemination of real-time information.

The Branson TRIP system will improve the use of alternative routes through the broadcasting of real-time traffic and travel information over a multitude of media. In addition, and as a result of the TRIP system being an operational test pilot which needed to be

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established in a six month time period, the Branson TRIP system is built upon the use of the significant resources currently available within the Branson area. These resources include three highway advisory radio (HAR) stations, a dedicated regional cable TV vacation channel and commercial radio station, existing tourist information web sites, changeable message signs (CMS) and the traffic and incident information currently obtained by the local police department.

PRE-TRIP, EN-ROUTE AND ON-SITE INFORMATION

The Branson TRIP is based on the foundation of providing tourists with comprehensive tourist attraction, weather, traffic and road construction information. To effectively capture the intended audiences and to provide the greatest potential for successful reception of information, the Branson TRIP utilizes existing information resources and advanced technologies to provide traveler information pre-trip, while en-route and at designated on-site attractions.

Pre-trip information is provided to potential tourists or individuals simply interested in Branson through a dedicated Internet web site and a dial-in telephone information service, or interactive voice response system (IVR). The information provided over both of these media will concentrate on providing users with information on tourist attractions, current and forecast weather, historically congested travel routes (with associated recommendations for travel route), area maps, and road construction schedules. Users of the Internet site will able to look at a map of Branson which identifies the current traffic conditions and traffic incidents. Snapshot images from the TRIP's four CCTV cameras will also be provided on the web site.

En-route traveler information is provided through portable and static changeable message signs (CMS), static advisory signs which direct patrons to tune into the highway advisory radio (HAR) operated by TRIP, the HAR and commercial radio. The en-route information concentrates on real-time traffic conditions, suggestions for alternative routes, and road construction updates.

On-site information is provided through interactive kiosks which are being deployed throughout Branson and surrounding areas and the dial-in IVR system. As a secondary information source, data collected from the Branson TRIP equipment is sent a local cable television station, the Vacation Channel, which utilizes this information to update their current programming on traffic and travel conditions. The on-site information provides tourists in the Branson Tri-Lakes area with information on how to get to their destination in Branson, the local and regional tourist attractions, and the best departure times and routes.

SYSTEM ARCHITECTURE AND DEVELOPMENT APPROACH

The driving concept behind the Branson TRIP project is to consolidate the data currently being collected by various public and private organizations in Branson, collect additional traffic, travel and tourism data and broaden information dissemination. This is achieved through the implementation and operation of the following subsystems:

- A central database where all the collected data is validated, normalized and fused
- Additional traffic detectors using inductive loop and radar technologies
- Video systems to capture and transmit video from remote locations
- Interactive kiosks providing a broad range of information services
- An Internet web site
- Changeable message signs
- Coordinated links to radio and television stations for direct broadcast
- Full-area HAR coverage.

In order to identify how to access and coordinate the data and dissemination methods planned for the program, the project team performed a review of each of the existing traffic systems, congestion and traffic concerns and landline and RF communications availability. Optimal strategies for system architecture were then developed based on regional availability, operating cost, and tie-in with other initiatives. A central database and traffic information center was developed to fuse the data sources into consistent formats for information dissemination. Information dissemination component technologies include the following:

Interactive Voice Response (IVR) Telephone System

This system permits users to dial-in and access information using a touch-tone telephone. To provide customized information to the end user, the IVR system uses a menu tree structure which queries the user to select from a list of information topic categories and navigate to the specific travel or tourist information sought.

The information provided over the IVR system is obtained directly from the TIC database. The IVR uses pre-recorded messages and inserts the real-time data at appropriate locations within the text. With multiple dial-in phone lines connected to the IVR, the system can handle up to four simultaneous calls at the same time with expansion capabilities up to an unlimited number of lines.

Web Site

The Branson TRIP web site fuses existing Branson area web sites with the most appropriate and informative data on attractions, lodging, events and restaurants in and around Branson into a single Branson area information source. The TRIP web site also incorporates information from the TIC, a graphical interface to display traffic conditions in Branson and images obtained from the systems CCTV cameras located throughout Branson. The TRIP web site development has been closely coordinated with the kiosk development to allow low-cost sharing of the multimedia development and interactive user interface activities.

Kiosks

Branson TRIP kiosks are being deployed throughout the City and surrounding areas. The kiosk design was based on obtaining data from the Branson TRIP web site, thus providing an efficient way of leveraging the existing development efforts to focus on broadening the audience and enhancing the existing information and services. The kiosk design activities have also focused on identifying future services that could be provided through the kiosks, for example ticket purchases, hotel, restaurant and sports reservations, map print-ing, coupon production, and registering to be informed of special offers in real-time.

Television and Radio Media Production

A cable television channel, which broadcasts tourist information to all of the hotels in Branson and a commercial radio station in Branson provide conduits for travel and tourist information, generated by the Branson TRIP project, to be broadcast to visitors. The data that is provided to the radio and television partners is sent directly from the TIC computer via a remote client terminal connected into the TIC LAN. Video from roadside CCTV cameras is being converted to broadcast quality for transmission as part of the cable channels programming.

Changeable Message Signs

Changeable message signs (CMS) are used to identify traffic conditions to traveler who are en-route to Branson. The CMS deployment process was based on assessing traffic flows and access points to the Branson area, identifying the travel decision points, such as prior to alternative route departure locations and defining appropriate messages which allow travelers to reach their destination in the shortest period of time or with the least inconvenience from traffic congestion. To provide more comprehensive data to the patrons of Branson, the CMS signs also direct patrons to tune into the real-time highway advisory radio.

Highway Advisory Radio (HAR)

Three existing Highway Advisory Radios (HAR) are being used to provide more comprehensive travel and traffic information then could be placed on the roadside CMS. The HAR messages have been developed to provide route specific information as a result of the multiple origin and destination locations of Branson travelers. This has been accomplished by dividing the city into a grid and identifying the most appropriate alternative route based on the patrons location within the grid system.

System Deployment Schedule

The system integrators are currently completing the development of the Branson TRIP, with the start of system operations occurring on June 1, 1998. With the cooperation of a true public/private partnership, the project team will have completed the initial system development in only 6 months. After June 1, certain development and deployment activities will continue to enable the operating system to be used as a marketing tool to gain new participants, and to allow some of the subsystems to be brought on line. Modifications to the system design and operational procedures based on operating experience are anticipated to take an additional 2 to 3 months.

The system is scheduled to be evaluated under the FHWA operational test for a full 11 months. However, current plans are underway to obtain further funding and private sector investment to allow the operation to continue after this period.

Public/Private Partnership

The Branson TRIP project has brought together private organizations and public agencies that represent all levels of government, from state transportation and tourism agencies to local counties, cities, and chambers of commerce. Together the public and private groups are making the Branson TRIP a successful operational test, leading to a self-sustaining operations center in Branson after the test.

The partnership agreements developed for this operational test were established in less than two weeks. Some agencies pledged support to the project and future expansion, but were not sure where they fit into the initial development and deployment. As a result, the partnerships were divided into three categories: Branson TRIP Partners, for those organizations that would be directly involved in the system design process and operational test; Branson TRIP affiliates, for those organizations that would support the program development within Branson; and Great Plains affiliates, for those organizations that would supply input into the development of the program for future expansion into a larger 6 state geographical area consisting of Missouri along with Arkansas, Iowa, Kansas, Nebraska and Oklahoma.

The Branson TRIP Partners

Missouri Department of Transportation
City of Branson
Missouri Division of Tourism
Stone County
Taney County
Branson Police Department
Southwest Missouri Advisory COG
Branson/Lakes Area Chamber of Commerce
Table Rock Lake Chamber of Commerce
Castle Rock Black & Veatch
ADDCO
Intuitive Solutions
The Branson Connection
KOMC FM Radio
KRZK FM Radio
The Vacation Channel

The Branson TRIP Affiliates

US Army Corps of Engineers
Silver Dollar City
Bass Pro Outdoor World
Western Transportation Institute
Perceptions

The Great Plains TRIP Affiliates

Arkansas Department of Transportation
Iowa Department of Transportation
Kansas Department of Transportation
Nebraska Department of Roads

Oklahoma Department of Transportation

While the initial deployment of the Branson TRIP is based on hardware deployment in Southwest Missouri, the broader program goal is the development of a seamless rural travel and tourist information system throughout the states of Missouri, Kansas, Arkansas, Oklahoma, Iowa and Nebraska. This natural expansion of the Branson TRIP, known as the Great Plains TRIP, will be coordinated and developed in conjunction with the initial operational test phase. Representatives from all six states will act as an advisory team as the first step in the deployment of a six-state Great Plains regional Travel and Recreational Information Program. Their activities will include reviewing the system development, identifying similar efforts in their states and identifying methods for expansion of the project to the Great Plains TRIP.

Cross-Cutting Study of Advanced Rural Transportation System ITS Field Operational Tests

KEITH JASPER

INTRODUCTION

USDOT has funded seven Advanced Rural Transportation System (ARTS) projects focused on traveler safety under the ITS Field Operational Test Program. Booz-Allen & Hamilton was contracted to oversee the evaluation of these (as well as 50+ other) field operational tests funded by FHWA. As the tests have advanced towards completion, Booz-Allen & Hamilton has studied groups of similar tests, identifying common issues and comparing findings. A significant component of this effort is a program of outreach to share these results, as well as lessons learned in implementing the projects, so that other ARTS deployments continue to build on the success established by these early projects. This paper describes the cross-cutting study of these seven tests:

- Advanced Rural Transportation Information and Coordination (ARTIC)
- Herald En-Route Driver Advisory System Via AM Sub Carrier, Phase II
- Idaho Storm Warning System
- Oregon Green Light Commercial Vehicle Operations Test
- San Diego Smart Call Box
- TransCal Interregional Traveler Information System
- Travel Aid.

DESCRIPTION OF THE TESTS

Advanced Rural Transportation Information and Coordination (ARTIC)

Introduction

The ARTIC ITS Field Operational Test combines the communications dispatch operations of four public service agencies into a single communications center that serves a remote area in the Arrowhead region of northeastern Minnesota. The ARTIC partnership crosses state agency jurisdictions and functions, and fosters cooperation between highway and transit interests. This cooperation is critical in remote, rural regions where resources are limited and pooling of assets is necessary to satisfy the operational requirements of multiple agencies.

The goals of the project are to coordinate and pool resources to reduce duplication, improve transportation system efficiency, and improve user and driver safety. ARTIC responds to the challenges of providing transportation services in a remote area with low population density, a harsh winter climate, an aging population, and the inefficient use of existing transportation resources.

The testing phase began in October 1997 and is continuing. Evaluation of the project focuses on user acceptance and satisfaction, system technical and functional performance, system efficiency and effectiveness, system costs, and legal and institutional issues.

Project Description

The test operation commenced in October 1997. Figure 1 illustrates the test area. A consolidated center located in Virginia, Minnesota, houses the emergency response functions and communications equipment for both the State Patrol and the Minnesota Department of Transportation (MnDOT). The center also houses the fleet management operations for Virginia Dial-a-Ride and Arrowhead Transit. Automatic Vehicle Location (AVL) devices and Mobile Data Terminal (MDT) equipment have been installed in 4 State Patrol cruisers, 15 MnDOT plow trucks, 12 Arrowhead Transit buses, and 3 Virginia Dial-a-Ride buses. This equipment provides operations personnel with the following features:

- Up-to-date information on vehicle location and availability
- Improved communications capability during emergencies.

The test implemented a computer-assisted transit scheduling system. The test also deployed a computer-aided dispatch (CAD) system to automate State Patrol call taking, communications, and records management functions. This deployment is part of a statewide program to expand the CAD system currently under development in the Twin Cities metro area to all Patrol districts outside the metro area.

The evaluation of the project focuses on:

- User acceptance and satisfaction
- System technical and functional performance
- System efficiency and effectiveness
- System costs

Booz-Allen & Hamilton, 8201 Greensboro Drive, Suite 609, McLean, Virginia 22102-3812.

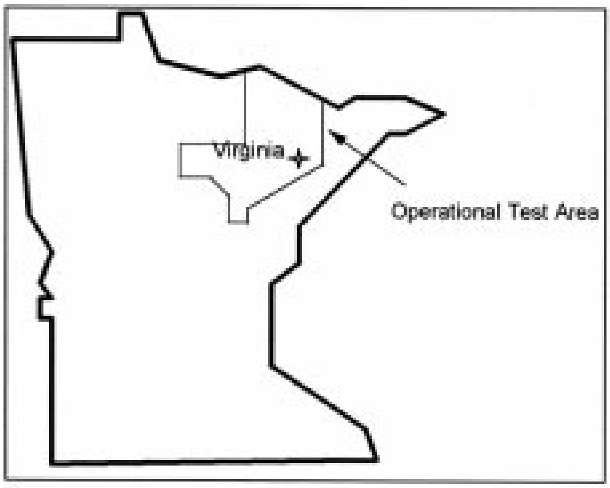

FIGURE 1 ARTIC operational test area in northeast Minnesota.

- Legal and institutional issues.

Herald En-Route Driver Advisory System Via AM Sub Carrier, Phase II

Introduction

The Herald En-Route Driver Advisory System Via AM Sub Carrier, Phase II (Herald II) ITS Field Operational Test evaluates the utility of providing traveler information in rural areas. The Herald II system employs a subcarrier on a commercial AM radio broadcasting to remote areas in Colorado and Iowa. The project proposes to test the feasibility of generating, transmitting, and receiving messages over a large geographic area. The test assesses the use of AM subcarriers as a reliable, low-cost medium to communicate traffic messages in the challenging terrain of Colorado and the potentially interfering environmental conditions of Iowa.

Phase I of the test occurred from October 1995 to December 1995. Phase II began operation in 1996. A final report is expected in the second quarter of 1998.

Project Description

The Herald project is being conducted in two phases. Phase I of the test consisted of a communication technology feasibility study funded entirely by the ENTERPRISE group. (See the Test Partners section for a description of the members of this group.) Activities in Phase I included a literature search to help determine the design approach, the development of specifications, data requirements, and simulation models, the development of a prototype system, and the performance of pilot tests.

Phase II is the actual field test and evaluation and is supported by the Federal Highway Administration. This Phase consists of developing the prototype mobile receivers, modifying and installing the transmitter sites, developing message formats, and collecting, analyzing, and evaluating data. The project will assess the performance of an AM subcarrier as a basic data communication channel. The project will also assess the impact of the AM subcarrier's channel characteristics on the channel's ability to disseminate traffic messages reliably and efficiently.

The project will provide two types of services: en-route driver information and traveler services information.

Herald consists of components that will address message generation, transmission, and reception. Figure 2 shows these components. The message generation component formats the traveler information. The message transmission component translates the formatted messages for transmission. The message reception component (in the vehicle) receives, decodes, translates, and presents the data in a format useful to the traveler.

To test the system in the field, test personnel are setting up the transmitters and installing receivers and measurement systems in test vehicles. These measurement systems will assess the AM subcarrier performance. The testing proceeds incrementally, gathering a small data sample and analyzing it before collecting more data. Test personnel start sampling the transmission at points close to the transmitters. As the test continues, testing will occur at greater distances from the transmitters and at varying times of day. While messages are being broadcast, test personnel will measure the signal strength according to standard criteria. Test measurements will eventually be taken in all planned terrain types and times of day.

The evaluation of the project will address two significant research questions about AM subcarrier modulation technology:

- Can it provide adequate signal coverage in a rural or rugged terrain?
- Can it provide accurate traveler information?

Idaho Storm Warning System

Introduction

The Idaho Storm Warning System ITS Field Operational Test is an Advanced Rural Transportation System test that is evaluating a system to warn motorists about adverse weather conditions. The system consists of a group of sensor systems that provide visibility and weather data coupled to a set of variable message signs (VMS) located along the highway. The system operates along a stretch of Interstate 84 in Idaho and northern Utah. The primary goal of the system is to reduce the number and severity of visibility-related multiple-vehicle accidents along this section of I-84.

Testing of the system components began in 1994. Due to a lack of visibility events in the early winters of the test and because of equipment operation problems, the data collection period was extended until March 1998.

Project Description

The project consists of two phases. The first phase tested three visibility sensors incorporating two weather information systems The purpose of the first phase was to determine the suitability of the sensors and weather systems for use in Phase II. The first phase also established the baseline information regarding driver behavior on the test section of Interstate 84 in southern Idaho. The second

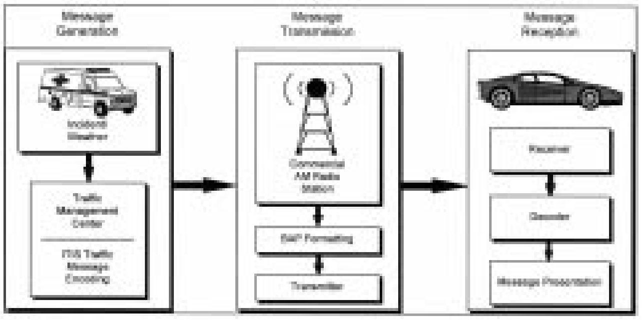

FIGURE 2 Components of Herald design concept.

phase is integrating the sensing technologies and a set of variable message signs (VMS) into an alarm and warning system to advise motorists of adverse visibility conditions.

The project need and purpose arose because of the history of accidents on a 100-mile long section of I-84 (see Figure 3). Certain areas in the test location are subject to low visibility conditions caused by blowing snow during winter and dust during spring. This section of I-84 also functions as a passenger and commercial vehicle travel route between Boise, Idaho, and Salt Lake City, Utah. From 1988 to 1991, this area experienced 18 major visibility related accidents, according to Idaho Transportation Department statistics. The percentage of trucks involved in these accidents (44 percent) exceeds their proportional representation (33 percent) in the traffic stream.

The project intended to reduce the number and severity of accidents on the subject section of I-84. The purposes of Phase I include evaluating the capability of three sensor systems to provide weather and visibility information and establishing baseline information about vehicle speeds before the installation of VMSs. In Phase II of the project, test partners are installing and integrating the VMSs. They are also evaluating the capability and suitability of the entire system in providing weather and visibility information to motorists.

Phase I of the test installed and evaluated three sensing systems: SCAN, Handar, and LIDAR. The SCAN system incorporates two separate visibility sensors, one using visible light and the other using infrared light. This system also includes four weather measurement sensors for wind speed and direction, air temperature, relative humidity, and type and amount of precipitation. The Handar system includes weather sensors similar to the SCAN system and a point detection visibility sensor similar to the visible light sensor of the SCAN system. The Light Detection And Ranging (LIDAR) system is a single visibility sensor using advanced laser technology. The LIDAR system operates similar to radar systems and can provide visibility measurements over a larger area than the other two technologies. During this phase, test personnel also used a video camera system to provide real-time verification of the conditions at the test site. Information from all these systems was transmitted to a master data collection computer at the Cotterell Port of Entry (POE) facility. The computer collected and analyzed sensor data every five minutes and alerted POE personnel if visibility fell below a predetermined threshold. If a visibility event occurred, system operators at the Cotterell POE confirmed the event using the video system. In Phase II when the operators confirm a low visibility event, they will manually activate the VMSs to advise motorists.

Oregon Green Light Commercial Vehicle Operations Test

Introduction

The Oregon Green Light ITS Field Operational Test is an evaluation of three major technical components intended to enhance commercial vehicle operations throughout Oregon. An electronic preclearance system employs transponders and weigh-in-motion (WIM) devices to reduce required stops by commercial vehicles at 22 weigh stations. The Downhill Speed Information Systems (DSIS), located at Emigrant Hill and Siskiyou Summit, calculates and displays a safe downhill speed for each passing truck. Of interest to this paper is the Road Weather Information Systems (RWIS), installed at Ladd Canyon, Columbia Gorge, and Siskiyou Summit, which collects weather data, processes it, and automatically informs motorists of abruptly changing weather conditions.

Systems are being installed and data collection began in fall 1997.

Project Description

The project is testing systems to make commercial vehicle operations safer, more efficient, and less expensive to both operators and the general public.

The purpose of the RWIS is to reduce the application of environmentally harmful abrasives. The project installed the RWIS in locations of rapidly changing weather patterns. A sensor package measures air and pavement temperatures, dew point, wind speed, visibility and precipitation. An on-site remote processing unit (RPU) autonomously detects hazardous conditions and displays a warning message on variable message signs. The RPU communicates with a central processing unit (CPU) in Salem, which displays all alerts on a website as well as on kiosks installed in major truck stops. From the CPU a system operator can also override the RPU and display other messages.

San Diego Smart Call Box

Introduction

The San Diego Smart Call Box ITS Field Operational Test evaluated the feasibility and cost effectiveness of using enhanced roadside call boxes for data collection, processing, and transmission. Smart Call Boxes are an improved version of devices used as emergency call boxes in California. The test examined using the smart boxes for traffic census data collection, incident detection, hazardous weather reporting, changeable message sign (CMS) control, and video (CCTV) surveillance. The evaluation focused on cost effectiveness compared to other methods.

The test had two goals:

- Evaluate the cost effectiveness of smart call boxes
- Document and discuss the institutional issues encountered.

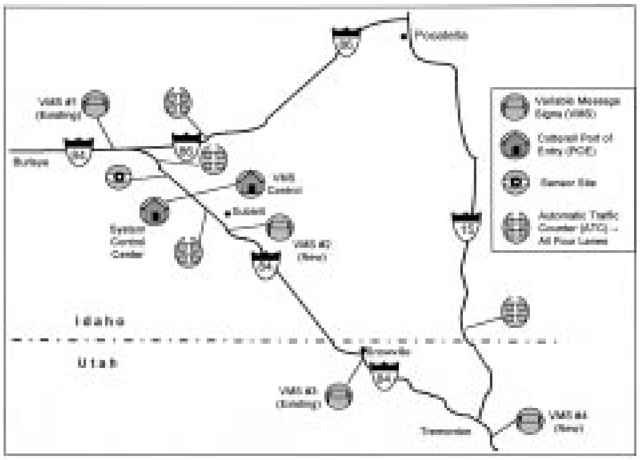

FIGURE 3 Map of project location.

The tests were conducted at numerous sites on the interstate and state highway system of San Diego County, California (see Figure 4). The test took place from September 1995 to June 1996.

Project Description

To improve motorist safety and emergency response, Caltrans (the California Department of Transportation) has installed an emergency phone system (call boxes) along many of the highways in the state. Motorists can use these phones, located at regular intervals, to connect directly to emergency dispatch centers. This Field Operational Test explored the possibility of using the established call box infrastructure to gather and transmit additional traffic and weather information.

The test planned to conduct five subtests, one for each data processing and transmission task. The project actually tested functional systems for traffic census data collection, hazardous weather reporting, and CCTV surveillance. Test partners canceled the changeable message sign subtest when the tested call box system proved incompatible with the California CMSs. The installed incident detection systems did not function properly.

The information collected by the call box installations was transmitted to the Caltrans District 11 Transportation Management Center.

The test had several objectives related to the project goals. Test evaluators attempted to determine the relative effectiveness of smart call boxes compared to a baseline system of conventional telephone lines and controllers. They also wanted to determine the projected life-cycle costs of the two systems and the tradeoffs between the systems. The ultimate objective was to determine which system is best for each task. The evaluators attempted to determine whether any institutional issues encountered have the potential for affecting the performance of similar systems.

TransCal Interregional Traveler Information System

Introduction

The TransCal ITS Field Operational Test evaluates an Interregional Traveler Information System (IRTIS). The IRTIS provides coverage for the Interstate 80 and US 50 corridor between San Francisco and Tahoe/Reno-Sparks area. The IRTIS proposed to disseminate customized traveler information via telephone, personal digital assistants (PDAs), and in-vehicle navigation devices (IVDs) as well as traditional broadcast media. The primary objective of TransCal is to disseminate comprehensive, accurate, and timely pre-trip and en route traveler information to help mitigate the impacts of congestion and incidents.

The Traveler Advisory Telephone System (TATS) component of the TransCal field operational test (FOT) became unofficially operational in March 1997 and will continue until September 1998. Testing of the PDAs and IVDs continued until March 1998. The Final Evaluation Report is expected in March 1999.

Project Description

TransCal implements a comprehensive interregional traveler information system that integrates road, traffic, transit, weather, and value-added traveler services from various sources. The project demonstrates the utility of an advanced traveler information system and showcases emerging capabilities in computing, communications, and consumer electronics. Figure 1 shows the area of IRTIS operation during the field operational test.

TransCal originally included two other components. These components were an emergency notification system to test a satellite-based two-way communication system, and a Tahoe transit frequent passenger program to increase transit use in the Lake Tahoe Basin. TransCal's Management Board, however, voted to eliminate these components from the project and redirected the funds in support of the IRTIS component.

The IRTIS operates from the TransCal Traveler Information Center in Sacramento, California. It receives real-time traveler related information from existing public and private interregional sources. It processes and fuses this data with existing static and periodic data and maintains a real-time traveler information database. The system disseminates the information to travelers via wireline and cellular telephones and FM subcarrier networks. The general public can access this information via telephone and traditional broadcast media. Test personnel are evaluating accessing the information using PDAs and IVDs. The paragraphs below briefly describe these devices.

- Personal Digital Assistants (PDAs) - The PDAs are hand-held, portable devices that provide users with information contained in the IRTIS database. The PDAs receive dynamic information types through the FM subcarrier data broadcast system.
- In-Vehicle Devices (IVDs) - The IVDs provide interactive access to detailed maps and the use of an integrated GPS receiver to determine the vehicle's current location. The IVDs receive dynamic information types through the FM subcarrier data broadcast system.

The IRTIS uses three types of data definitions: static, periodic, and dynamic. Static data remains relatively constant over time and for the duration of the test. Periodic data remains relatively con-

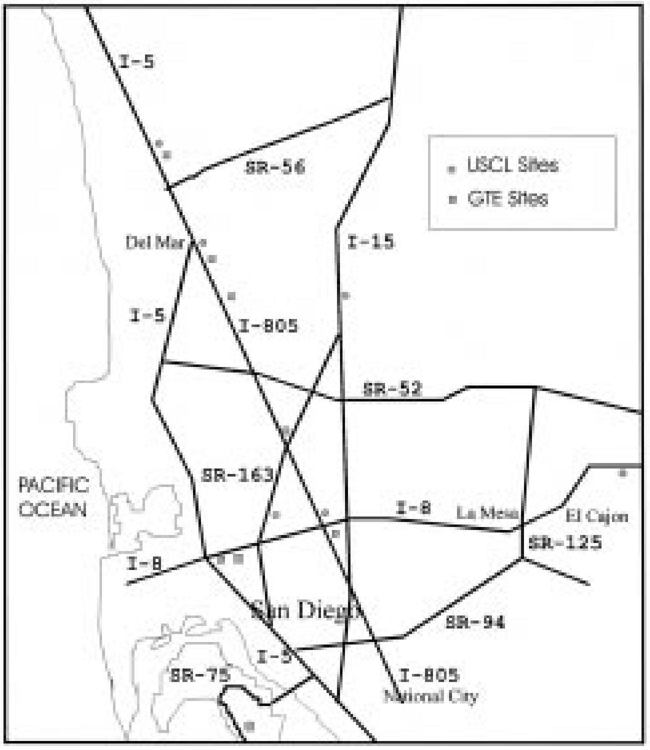

FIGURE 4 Smart call box field test sites.

stant for short periods of time - on the order of weeks. Dynamic data consists of current conditions obtained as they occur. Static data may reside in the IRTIS database or within the dissemination devices themselves. Periodic and dynamic data is processed and maintained by the IRTIS computer workstations in real-time. Table 1 provides a list of data types within each data category.

IRTIS consists of a data processing subsystem (IRTIS main database), a data dissemination system, and multiple end-user technologies designed to receive data from the IRTIS main database.

The data processing subsystem combines data from multiple sources to produce an integrated list of freeway and arterial incidents, emergency maintenance, planned event status, and regional weather status. The subsystem also determines the initial status of new traffic incidents and updates the current incident list as appropriate. The data processing done by the IRTIS uses the TRW Trans View advanced traveler information software. This software supports the collection, processing, and dissemination of real-time traffic and transit information. The data processing subsystem consists of computer workstations and servers on a local area network that is scaleable in size to accommodate any number of inputs and outputs. The network connects to a wide range of traveler services and products. A separate computer server acquires, processes, maintains, and disseminates the information.

The IRTIS automates the data collection process as much as possible. An IRTIS operator, however, must manually input data from some data sources. An IRTIS operator is also responsible for keeping traveler information accessible via public telephone through a voice processing system called the Traveler Advisory Telephone System. The operator makes a voice recording of any changes in traveler information based on reported changes of the current traveler information database. Travelers can access this information by calling a single telephone number.

The evaluation goals of the TransCal project include:

- Assess user acceptance from the perspective of the end-users, public partners, and private partners
- Assess benefits and costs of IRTIS
- Assess system performance of IRTIS as an integrated system and by system component
- Assess IRTIS impact on travel behavior
- Assess institutional and legal issues.

Travel Aid

Introduction

The Travel Aid ITS Field Operational Test intends to improve safety and reduce accidents for travelers crossing the Snoqualmie Pass along Interstate 90 north of Seattle, Washington. The test will achieve this goal by transmitting suggested speed limits and traveler advisory messages to variable message signs (VMS). The Travel Aid system broadcasts advisories throughout the 40-mile length of freeway included in the Travel Aid test.

Field testing is currently underway. A final evaluation report is expected in September 1998.

Project Description

Accident data has shown that the accident rate on I-90 across Snoqualmie Pass in January is 12 accidents per 100,000 vehicles; during July the rate is 1 accident per 100,000 vehicles. During winter, snow, ice, fog and other weather extremes make driving more difficult than at other times. The traffic mix over the Pass in winter months includes recreational travelers traveling to and from the various wintertime recreation destinations, as well as a significant number of tractor-trailers. The trucks must proceed at reduced speeds when climbing or descending the Pass. During inclement weather, snow removal equipment is out in force to maintain the roadway. The Washington State Patrol and Washington State Department of Transportation maintenance staff have indicated that many accidents are caused by drivers traveling too fast for the prevailing weather and traffic conditions. The result is a very high winter season accident rate.

The goal of the Travel Aid test is to reduce the frequency and severity of accidents on Snoqualmie Pass. The test focuses on the winter weather season, but is applicable to any time of year, since weather and driving conditions are unpredictable and can be severe due simply to the elevation of the Pass.

Travel Aid transmits speed limit information and traveler advisory messages to variable message signs (VMS) (State police have issued citations to motorists exceeding the speed limit posted on the VMS.) The Travel Aid system provides three types of information to a software-based algorithm that generates suggested speed limits for vehicles. Radar detectors gather average vehicle speed data. Sensors embedded in the pavement determine pavement conditions. Weather stations record information including wind speed, temperature and precipitation. Figure 5 presents the design overview of the system.

TABLE 1 IRTIS Data Definitions

Static Data Definition	Periodic Data Definition	Dynamic Data Definition
Freeway segment definitions	Transit schedule	Freeway incidents
Arterial segment definitions	Transit fares	Arterial incidents
Transit segment definitions	Planned lane closures	Transit incidents
Transit stop locations	Planned detours	Transit schedule change
Major points of interest	Planned events	Emergency maintenance
Transit route definitions	Airline phone numbers	Planned event status
		Regional weather status

This information is synthesized and processed in the central Travel Aid file and communications server at the operations center in Hyak. The computer algorithm suggests a speed limit and the Travel Aid operator reviews it. If the operator concurs with the limit, he transmits traveler advisory messages to the variable message signs (VMSs) and in-vehicle units. This transmission occurs via radio and microwave.

TEST STATUS AND RESULTS TO DATE

Advanced Rural Transportation Information and Coordination (ARTIC)

Test Status

The project began operations in October 1997. Data collection will continue until September 1998. The Final Evaluation Report is anticipated in December 1998.

Results

Although system operations have just begun, the use of the communications facility is already yielding benefits. Anecdotal evidence exists that describes rapid responses to emergencies, particularly in winter conditions, that would not have been possible prior to system deployment. In one case, a MnDOT snowplow responded to a vehicle accident location in a fraction of the time that it would have taken for other law enforcement assets to respond. The snow plow operator was able to relay critical information to help resolve the accident situation.

All of the participating agencies are very enthusiastic about the project. The agencies look on the project more as an actual deployment than as a test. The transit agencies in the project have begun specifying that their vehicles be equipped with AVL technology as part of the original purchase. State agencies are already planning to include continuing operation funding in their respective budgets. In short, the participating stakeholders already consider the test a success.

Herald En-Route Driver Advisory System Via AM Sub Carrier, Phase II

Test Status

Test personnel are currently analyzing the initial data collected. Additional data collection is tentatively scheduled for six weeks, starting in April 1998. The evaluator will analyze the data in parallel with its collection.

Results

No interim results are available.

Idaho Storm Warning System

Test Status

Phase I of the test is continuing in parallel with Phase II, which began in November of 1996. Due to equipment operation problems and the lack of visibility events in winter 1996/1997, data collection was extended until March 1998.

Results

The results presented in this summary come from an interim report on Phase I dated January 1997 and a progress report for the winter of 1996/1997. The LIDAR system visibility sensor was not operational during the winter of 1995/1996 as reported in the Interim Report. The test defines a visibility event as one in which sight distance drops below 1,200 feet. According to available sensor information, there were no visibility events during the winter of 1996/1997. The LIDAR system did operate during this later period but the accuracy of the LIDAR data is subject to question because the system was still being calibrated. The Final Report is expected in August 1998.

The primary purpose of Phase I of the test was to determine whether the three sensor systems were capable of measuring visibility accurately. Test evaluators analyzed the visibility sensor effectiveness through three methods: POE personnel confirmation, video playback visibility comparison, and correspondence between the operating sensors.

The POE personnel confirmation and the video playback comparison were both subject to many problems that reduced their effectiveness as an accurate confirmation method. POE personnel confirmation problems included technical problems that caused the POE personnel to lose confidence in the system and the complicated and time-consuming nature of the method. Video playback problems included an ineffectiveness at determining precise visibility distances and several technical problems that resulted in limited availability of information. Considering the limited informa-

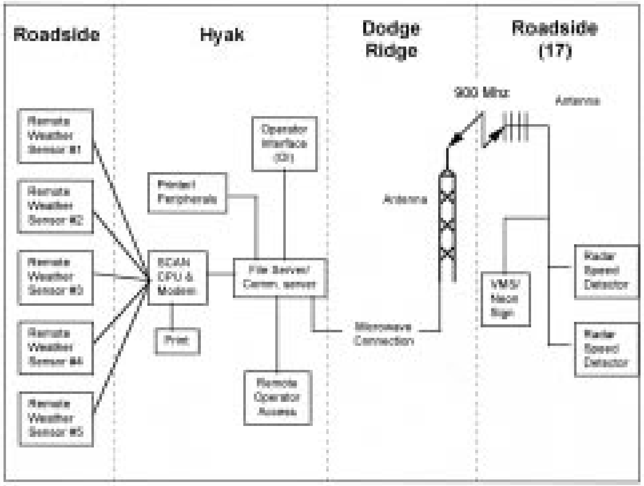

FIGURE 5 Travel aid system design overview.

tion available using POE personnel confirmation and video playback, this confirmation method shows general agreement between the observations of the personnel and the sensors' visibility during several event periods. Precise correlations, however, proved to be impractical due to differences in individual judgment and the POE personnel's busy schedule of other duties.

Despite the problems that reduced the effectiveness of the video playback confirmation method, the available Handar and SCAN data show a general agreement between the observed distances in the videos and the sensor distances.

The comparison of visibility readings from the Handar and SCAN sensing systems (involving three different sensors) showed that this comparison is unable to determine if the sensors provide accurate visibility distances. The sensors do, however, show a high correlation between themselves. Although they provide significantly different visibility readings, it was common to have correlation values of over 0.900 between different sensors. In particular, the infrared sensor tended to give lower visibility readings during snowy conditions but higher readings during foggy conditions compared to the two visible light sensors. In spite of these differences, the three sensors showed a strong, positive linear relationship. This means that when one sensor shows a decline in visibility, the other two also exhibit a decline. Conversely, when visibility improves according to one sensor, the others also show improvement.

The LIDAR information from the winter of 1996/1997 indicated more frequent low visibility readings than the other sensors. As noted earlier, however, the LIDAR system was still being calibrated. The LIDAR system shows the complexity of visibility measurements because it produces visibility estimates at one-quarter mile intervals for up to several miles away from the site. These discrete quarter-mile estimates show significant variations in visibility (as much as several thousand feet) from one interval to another. This implies that information from point sensors like Handar or SCAN is highly dependent on sensor location.

The information from Phase I indicates that the sensors have the potential to provide useful information regarding low visibility. The mixed results, however, mean that in Phase II it is important to have information from all three sensors to determine the nature of the message to be displayed on the VMSs.

Oregon Green Light Commercial Vehicle Operations Test

Test Status

Contractors completed installation and testing of the RWIS equipment in the third quarter of 1997.

In light of the large scale of both test and evaluation, the earliest results are expected in January 1998. The final report is due in April 2000.

Results

No interim results are available.

San Diego Smart Call Box

Test Status

The test has concluded.

Results

The evaluation found the smart call box concept to be feasible but not necessarily optimal. Smart call box systems will often be cheaper to deploy than hardwired systems. Smart call box systems, however, did not necessarily prove to be superior to other wireless systems.

The planned tests encountered problems. Most test equipment experienced varying periods of three conditions: operational, operational with problems, and non-operational. Test personnel were not able to control changeable message signs using the tested call box system because the CMSs used in California proved incompatible. Therefore, test personnel canceled the CMS subtest. Test personnel installed call box-based incident detection systems but these systems did not function properly. In the video (CCTV) subtest, the installed system could not remotely control the pan-tilt-zoom capabilities because of communication and system integration problems.

Test personnel encountered significant system integration problems. The design of portions of the system to be located at the TMC was considered to be outside the scope of the test. Therefore, test personnel made use of existing data collection components or used the simplest possible means. Problems also arose because the call boxes (owned and operated by the partners) ran software provided by the vendors. Evaluators were not sure if some data integration problems resulted from basic incompatibilities or from the project staff's lack of familiarity with the software. These problems led to reduced usefulness of some of the data or delays in integration of the field data with the TMC data.

For the three successful subtests, evaluators estimated cost savings. Evaluators noted that using Smart Call Boxes to control field devices in the three successful tests could result in substantial per site savings over other alternative control options. The possible capital cost savings ranged from about $1,500 to as much as $103,000.

Evaluators noted several institutional issues that need to be resolved prior to full-scale deployment of smart call box systems. Future systems must be rigorously tested and include design enhancements, improved reliability, and lower maintenance costs. Any agency considering deployment of such a system should prepare detailed deployment plans. The agency should also resolve other important issues, such as ownership, financing, and maintenance. Evaluators also cited inadequate involvement of the partner agencies and the potential users of the system in the development of system designs. The organizational structure of the test partnership and the cumbersome contracting procedures of the partners resulted in major delays that had a negative effect on the outcome of the subtests. Evaluators suggested that the project manager and the vendors be included as partners to overcome some of these problems.

Legacy

The results of this test led to a decision to prepare a proposal for pilot deployment of a small scale smart call box system in the San Diego area. The deployment would install systems to collect traffic census data, detect low visibility conditions, monitor wind speed, and verify CMS messages by CCTV. The deployment plan would provide for further testing and system development.

Other smart call box projects are currently in progress in California. In the San Bernadino-Riverside area, call box systems are monitoring traffic census data and weather conditions. In Sutter County, systems are collecting traffic census data and detecting low visibility conditions.

TransCal Interregional Traveler Information System

Test Status

After beginning operation of the TATS component, issues pertaining to the in-vehicle device performance and proposed kiosk database quality delayed a full conduct of the test. In February 1998, the test partners decided to end test operations. The partners will finish testing the PDAs and IVDs by the end of March 1998. The TATS will remain operational until September 1998 using state funds. The test will redirect the existing evaluation efforts to focus on capturing the institutional and technical lessons learned during the course of the test. The evaluator will prepare a final report by the end of March 1999.

Results

No interim results are available.

Travel Aid

Test Status

The test is underway and test personnel are collecting evaluation data.

Results

No interim results are available.

LESSONS LEARNED

Approach

Through published (where available), and regular contact with these seven projects, Booz-Allen & Hamilton developed summary lessons learned. These are grouped into three categories:

- Planning
- Procurement
- Development

Planning

- Do not be over ambitious, particularly for your initial deployment.
- Strong leadership is required at the implementing agency(ies), combined with multi-agency commitment to stay the course through deployment and operation/maintenance.
- Address resource and skill deficiencies, including training in agency personnel responsible for the systems.
- Consider range of communications media to find the most cost-effective and reliable.

Procurement

- Utilize appropriate procurement vehicles; ITS have many characteristics that make them different from truckloads of rock or tons of rebar.
- Make provisions for life-cycle support in the original project planning.

Development

- Realistic delivery timelines are essential.
- Systems integration in remote areas is an enormous challenge.
- Maximize the available deployment season.
- Timely maintenance procedures are required in order to gain expected benefits from the installed devices.

Iowa Thickness Design Guide for Low Volume Roads Using Reclaimed Hydrated Class C Fly Ash Bases

KENNETH L. BERGESON AND ANDREW G. BARNES

This paper is intended to provide flexible pavement thickness design parameters and a design method for low volume roads and streets utilizing Iowa reclaimed hydrated Class C fly ashes as artificial aggregates for a base material. AASHTO design guidelines are presented for using these materials untreated, or if higher strengths are needed, stabilized with raw fly ash or hydrated lime. Hydrated Class C fly ashes in Iowa are produced at sluice pond disposal sites at generating stations burning western sub-bituminous coals. They may be formed by dozing raw ash into the sluice pond where it hydrates to form a cementitious mass or they may be constructed as an engineered fill (above the sluice pond level) by placing the raw ash in lifts, followed by watering, compaction and subsequent hydration. The hydrated ash is typically mined by using conventional recycling-reclaiming equipment to pulverize the material where it is stockpiled on-site for use as an artificial aggregate. Research has been conducted on these materials, on an on-going basis, under the Iowa Fly Ash Affiliate Research Program since 1991. Test roads have been constructed using reclaimed fly ash as an aggregate base in Marshalltown (1994) and near Ottumwa (1995). They have been, and are, performing well. Based on extensive laboratory testing, this paper presents layer coefficients for reclaimed hydrated Class C fly ash bases for use in AASHTO thickness design for low volume roads and streets. Key words: fly ash, hydrated fly ash, bases, AASHTO design, layer coefficients.

OBJECTIVE

This guide is intended to provide flexible pavement thickness design parameters and a design method for low volume roads and streets utilizing Iowa reclaimed hydrated Class C fly ashes as artificial aggregates for a base material. Design guidelines are presented for using these materials untreated, or if higher strengths are needed stabilized with raw fly ash or hydrated lime. The complete report containing all of the data used in development of this guide is available on request (*11*).

BACKGROUND

Hydrated Class C fly ashes in Iowa are produced at sluice pond disposal sites at generating stations burning western sub-bituminous coals. They may be formed by dozing raw ash into the sluice pond where it hydrates to form a cementitious mass or they may be constructed as an engineered fill (above the sluice pond level) by placing the raw ash in lifts, followed by watering, compaction and subsequent hydration.

The hydrated ash is typically mined by using conventional recycling-reclaiming equipment to pulverize the material where it is stockpiled on-site for use as an artificial aggregate.

Research has been conducted on these materials, on an on-going basis, under the Iowa Fly Ash Affiliate Research Program since 1991 (*1*,*2*). Test roads have been constructed using reclaimed fly ash as an aggregate base in Marshalltown (1994) and near Ottumwa (1995). They have been, and are, performing very well (*3*,*4*).

PAVEMENT THICKNESS DESIGN BY AASHTO METHODS

The following formula controls selection of layer thicknesses in the low volume AASHTO design method (*5*).

Equation (1):

$SN = a_1D_1 + a_2D_2 + a_3D_3$

where

SN = Structural Number

a_1, a_2, a_3 = layer coefficients representative of surface, base, and subbase courses, respectively

D_1, D_2, D_3 = actual thicknesses (in inches) of surface, base, and subbase courses, respectively

The AASHTO Design Guide also provides correlations from common laboratory tests such as the California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) to layer coefficients. Once a layer coefficient is selected for each layer, pavement layer thickness may be determined from the SN equation. There is no unique solution, therefore, the one which is the most economical should prevail.

The AASHTO Design Guide also suggests a simplified procedure for the design of low-volume roads. The simplified variables for this design procedure are basically the same as in the standard design:

- U.S. Climatic Regions
- Relative Quality of Subgrade Soil
- Traffic Level
- Inherent Reliability

CORRELATION OF STANDARD TESTING AND MATERIAL TYPE TO LAYER COEFFICIENT

There is limited information on the correlation of standard testing results to layer coefficients for flexible pavement design. This sec-

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TABLE 1 Correlations of Standard Tests to Layer Coefficients (a_i) from the AASHTO Design Guide

a_i	CBR, %	Unconfined Compressive Strength, psi
0.10	25	[a]0
0.11	35	100
0.12	40	175
0.13	70	225
0.14	100	300
0.15	-	390
0.16	-	425
0.17	-	500
0.18	-	575
0.19	-	610
0.20	-	675
0.21	-	750
0.22	-	800

tion will summarize suggested layer coefficients based on UCS, CBR, and material description. Surface, base, and subbase layer coefficients a_1, a_2, and a_3, are all measured on the same scale, so in the following discussion any layer coefficient will be designated a_i.

The AASHTO Guide For Design of Pavement Structures (*5*) offers a fairly comprehensive correlation between CBR and a_i for granular layers, and between unconfined compressive strength and a_i for cement treated layers. The CBR correlation, derived from work in Illinois, is non-linear and does not account for CBR values in excess of 100. The correlation for UCS, from work in Illinois, Louisiana, and Texas, is approximately linear. The UCS is typically used in assigning layer coefficients where CBR values are in excess of 100 percent. Table 1 summarizes AASHTO correlations.

The National Asphalt Paving Association published *A Guide to Thickness Equivalencies For the Design of Asphalt Pavements* in 1984. They determined and suggested layer equivalencies based on the AASHTO Road Test results and AASHTO committee judgment (*6*). The unique quality of their assignations is that they are relative values, with no specific layer coefficient given. Table 5 shown on Figure 5 shows the coefficients resulting from the assumption that the coefficient for dense graded crushed stone is 0.14.

The most interesting approach to the assignment of layer coefficients for pozzolanic pavements was presented in a paper by Walter Morris (*7*). Morris' paper focused on the fact that pozzolanic and cement treated bases were not really considered in the AASHTO Road Test. Morris then cites the finding by the Pennsylvania DOT that structural coefficients change with layer thickness. Layer coefficients of semi-rigid to rigid bases increase as the layer thickness increases. He then presents a correlation of UCS to layer coefficient for differing thicknesses of pozzolanic base as shown on Table 2.

Fly ashes are pozzolanic materials and Class C fly ashes are both self-cementitious and pozzolanic. The reclaimed hydrated Class C ashes although hydrated and hard still contain an abundance of unreacted pozzolanic materials. When used untreated and compacted into a base layer they slowly develop strength over time due to continued pozzolanic reactions. The addition of a calcium source to the reclaimed hydrated ashes serves to activate additional pozzolanic reactions and results in dramatic increases in strength. Development of layer coefficients for design considers this in addition to adjustments of coefficients based on Morris' work.

TABLE 2 Layer Coefficients for Pozzolanic Bases from Morris, 1989

	Thickness of Pozzolanic Base, inches					
Design Strength, psi	6	7	8	9	10	12
600-800	.20	.22	.24	.26	.28	.34
800-1100	.23	.26	.29	.32	.35	.41
1100-1500	.28	.32	.35	.38	.41	.44
1500 & above	.32	.36	.40	.44	.44	.44

TABLE 3 Typical CBR Values for Different Materials

Description of Material	**CBR, %**
Well-graded crushed aggregates	100
Well-graded natural gravels	80
Gravelly sands (predominately sand)	20-50
Silty or clayey sands	10-40
Fine clean sands	10-20
GW: gravel or sandy gravel	60-80
GP: gravel or sandy gravel	35-60
GM: silty gravel or silty, sandy gravel	40-80
GC: clayey gravel or sandy, clayey gravel	20-40
SW: sand or gravelly sand	20-50
SP: sand or gravelly sand	10-25
SM: silty sand	20-40
SC: clayey sand	10-20
CL: lean clays, sandy clays, gravelly clays	5-15
ML: silts, sandy silts	5-15
OL: organic silts, lean organic clays	4-8
CH: fat clays	3-5

DEVELOPMENT OF LAYER COEFFICIENTS FOR RECLAIMED FLY ASH

In order to develop layer coefficients for reclaimed fly ash, CBR tests were utilized in conjunction with UCS tests. Tests were conducted on untreated reclaimed ash and reclaimed ashes activated with raw fly ash and hydrated lime (*11*).

California Bearing Ratio

The CBR was developed by the California Highway Department, and is one of the leading flexible pavement design procedures in the world (*8*). It can be conducted in the laboratory, or on field samples, and although it is most appropriate for fine-grained soils, it is also used to characterize aggregates for road base applications. The CBR test is most effectively used in the case where layers of the pavement increase in strength from the subgrade to the surface (*9*). It is important to realize that while CBR gives an index of strength, it is not a strength measurement. Table 3 is condensed from Rollings and Rollings (*8*), and gives some typical CBR values.

TABLE 4 Range of specific gravity and absorption for reclaimed fly ash materials

Bulk Specific Gravity of Coarse Material (+#4)	Bulk Specific Gravity of Fine Material (-#4)	Absorption of Coarse Material (%)
1.45 - 1.53	1.99 - 2.26	24 - 37

Care should be taken in the assignment of a CBR value to a highly cemented granular material, as strain induced by traffic may break the cementitious bonds (*9*). Factors affecting CBR value include maximum aggregate size, gradation, aggregate shape and texture, and plasticity. The American Society for Testing and Materials (ASTM) Standard Test Method ASTM D1883 for CBR suggests that materials containing a substantial fraction larger than the #4 sieve will yield more variable results (*10*).

Unconfined Compressive Strength

The UCS has long been used to evaluate the strength of soils and stabilized materials.

MATERIALS

For development of this guide the following material sources were used.

Reclaimed hydrated Class C fly ashes were obtained during 1995 and 1996 from the following generating stations: Ottumwa, Port Neal 3, Prairie Creek, and Council Bluffs. Raw fly ash used to activate the reclaimed ashes were obtained from the respective generating stations during 1996 and 1997. UCS and CBR tests were conducted using 10, 15 and 20 percent raw ash. Little additional benefit was obtained from reclaimed ash activated with 15 percent and 20 percent raw ash. Reagent grade hydrated lime was also used as an activating agent. Cement kiln dust (CKD) and Atmospheric Fluidized Combustion Residue (AFBC) have also been used as activating agents. Chemical composition and test data are included in reference 11.

RECLAIMED FLY ASH ENGINEERING PROPERTIES

Particle Size Distribution

The reclaimed ash stockpile typically contains 10 to 15 percent plus 3/4 inch material. This material is crushed to pass the 3/4 inch sieve and reincorporated into the sample prior to testing. The results of particle size analysis conducted according to ASTM C136 is shown on Figure 1.

Figure 1 shows that all the reclaimed ashes are reasonably well graded. The Council Bluffs and Neal 3 material have nearly identical gradation, and are the coarsest of the four, with about 15 percent passing the #200 sieve. The Ottumwa material is the finest, with about 25 percent passing the #200 sieve. The Prairie Creek material falls halfway between the others, at about 20 percent passing the #200 sieve. These results are typical of numerous other samples that have been tested under previous Affiliate research (*1*,*2*).

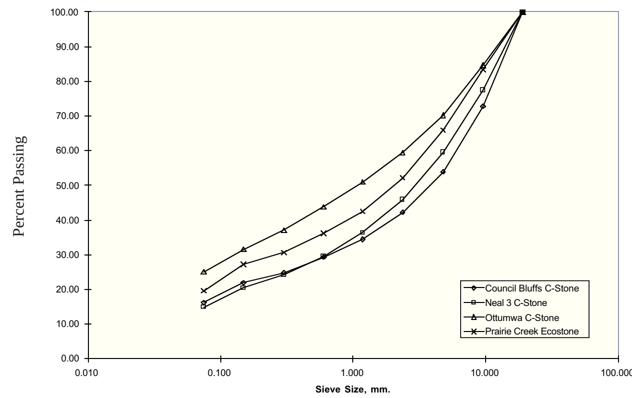

FIGURE 1 Particle size distribution of reclaimed fly ashes.

Specific Gravity and Absorption

Specific gravity and absorption of the coarse fraction of the reclaimed ash was conducted according to ASTM C127, Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate. The specific gravity of the fine fraction (material passing the #4 sieve) was determined by the Iowa State Materials Analysis and Research Laboratory (MARL) using a helium pycnometer. The range of results of the specific gravity testing are shown in Table 4. The specific gravity of the materials is in the range of a lightweight aggregate. The high porosity is evidenced by the high absorption values.

Moisture/Density Relationships

Moisture/Density relationship of the reclaimed fly ash was determined according to ASTM D698, Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort. Tests were conducted using molds 6 inches in diameter by 4.584 inches high. Specimens were formed in three equal layers, with each layer being compacted with 56 blows from a 5.5 pound hammer dropped twelve inches. Specimens were weighed to the nearest gram, and a moisture sample was taken from each. One curve was developed for each combination of reclaimed fly ash and additive. All additive levels are based on the oven dry weight of reclaimed fly ash. Table 5 summarizes the range of moisture/density relationship for all the materials tested.

TABLE 5 Range of ASTM D698 Test Results for Reclaimed Fly Ash Materials

Material	Optimum Moisture (%)	Maximum Dry Density (pcf)
Untreated	27 - 37	79 - 88
Raw Fly Ash, activated 10%	26 - 38	75 - 91
Lime, activated 2-1/2%	27 - 39	77 - 87

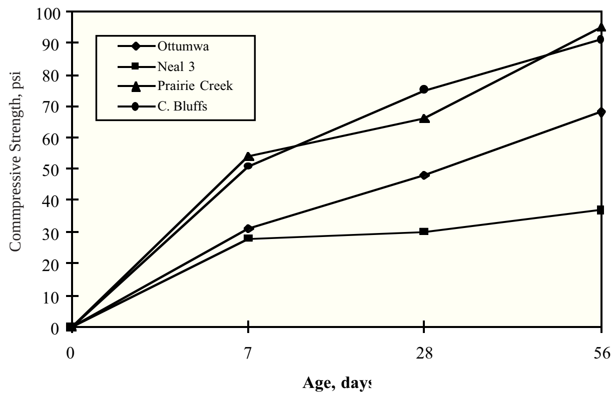

FIGURE 2 Untreated reclaimed ash strength gain.

Strength Development Characteristics

Untreated reclaimed ash - laboratory results

When moistened and compacted, untreated reclaimed fly ash materials gain strength as a function over time due to pozzolanic reactions. Figure 2 illustrates this for the 4 reclaimed ashes used in this study. As long as water is available it is believed that this pozzolanic strength development continues to occur over a long period of time (years) and the material is believed to be able to heal itself following crack development (autogenous healing). Untreated ash bases perform primarily as a flexible base.

Calcium activated reclaimed ash - laboratory results

The addition of an external source of calcium to the reclaimed ash during base construction can dramatically in-crease strength development. The calcium activates additional pozzolanic reactions among the aggregate particles thereby cementing them together. Strength gains can be dramatic and are long-term. Figure 3 illustrates strength gain characteristics of 10% raw fly ash activated reclaimed ash. Fly ash activation transforms the material into a semi-flexible base.

Figure 4 illustrates strength gain characteristics of 2-1/2 % hydrated lime activators and shows the strong increase in strength being developed from pozzolanic activity. Lime activation transforms the material into a semi-rigid base for heavy load applications.

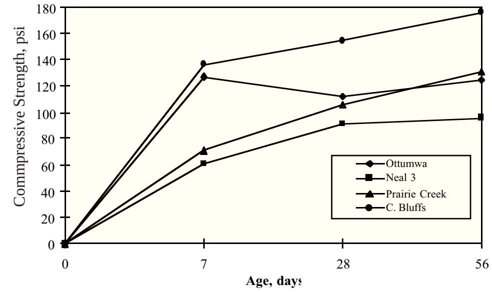

FIGURE 3 10% Raw fly ash activated reclaimed ash strength gain.

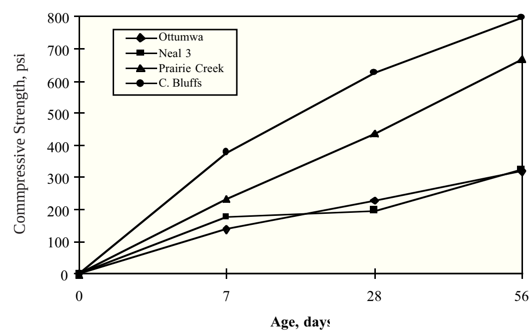

FIGURE 4 2-1/2% Lime activated reclaimed ash strength gain.

DESIGN PARAMETERS

Range of Test Results

Numerous CBR and UCS tests were conducted on untreated and stabilized materials used in the development of this guide.

Layer Coefficients

The coefficients for a 6 inch base have been developed using 7 day CBR values in the cases where the CBR is less than 100 percent and using UCS values at 56 days of age in the cases where the CBR's were in excess of 100 percent. The coefficients have then been modified for greater base thicknesses using Morris's data for pozzolanic bases.

Based on the testing data the suggested layer coefficients for use in preliminary design are given in Table 6 shown on Figure 5 as a function of base thickness. These coefficients are believed to be conservative. Testing should be conducted on project specific materials to confirm or increase the design coefficients.

DESIGN METHODOLOGY

The AASHTO method suggested for low volume pavement thickness design using reclaimed fly ash follows.

SUBGRADE RATING

Determine the average quality of the project subgrade soil from Table 1.

Table 1. Subgrade soil quality.

Rating (%)	Modulus of Subgrade Reaction (k, pci)	CBR Value
Very Good	Greater than 550 pci	> 55
Good	400 to 550 pci	40 - 55
Fair	250 to 350 pci	20 - 35
Poor	150 to 250 pci	6 - 20
Very Poor	Less than 150 pci	< 6

TRAFFIC LEVEL

From estimated traffic projections determine the traffic level anticipated from Table 2.

Table 2. Traffic levels for the low-volume design method.

Traffic Level	ESAL's
High	700,000 to 1,000,000
Medium	400,000 to 600,000
Low	50,000 to 300,000

STRUCTURAL NUMBER (SN)

Estimate the structural number from Table 3.

Table 3 Recommended ranges of Structural Number (SN) flexible pavement design catalog for low-volume roads.

Relative Quality of Subgrade Soil	Traffic Level	75% Level of Inherent Reliability (SN)
Very Good	High	3.0 - 3.2
	Medium	2.7 - 3.0
	Low	2.0 - 2.6
Good	High	3.3 - 3.4
	Medium	3.0 - 3.2
	Low	2.2 - 2.8
Fair	High	3.4 - 3.5
	Medium	2.7 - 3.3
	Low	2.3 - 2.9
Poor	High	3.7 - 3.0
	Medium	3.4 - 3.6
	Low	2.5 - 3.2
Very Poor	High	3.8 - 4.0
	Medium	3.5 - 3.7
	Low	2.6 - 3.3

MINIMUM THICKNESS

Establish the minimum thicknesses of the asphalt surface course and base from Table 4.

Table 4. Recommended minimum pavement layer thicknesses (inches).

Traffic, ESAL's	Asphalt Concrete	Aggregate Base
Less than 50,000	1.0 (or surface treatment)	4
50,001 - 150,000	2.0	4
150,001 - 500,000	2.5	4
500,001 - 2,000,000	3.0	6
2,000,001 - 7,000,000	3.5	6
Greater than 7,000,000	4.0	6

SUBBASE

The need for a subbase should be determined by the designing engineer based on the quality of the subgrade soil.
If the subgrade is so soft that stabilization (lime, fly ash, cement) is required, CBR tests should be conducted on the stabilized soil and the subgrade soil quality then determined from Table 1, before proceeding with the design.

RECLAIMED ASH BASE

Determine source and type available (described under layer coefficients).

LAYER COEFFICIENTS

- **Assign layer coefficients**
  - ➔ HMA Surface Course ....(a_1) - Table 5
  - ➔ Reclaimed Fly Ash Base ....(a_2) - Table 6
  - ➔ Other Bases ....(a_2) - Table 5
  - ➔ Subbase ....(a_3) - Table 5

Table 5. Layer coefficients (a_i) derived from thickness equivalencies, developed by National Asphalt Pavement Association, 1984.

Layer	Thickness Equivalency	a_i
HMA surface, binder, leveling and base courses		
mixes containing crushed coarse aggregate	3.00	0.42
mixes containing uncrushed coarse aggregate	2.75	0.39
mixes containing no coarse aggregate (sand asphalt)	2.50	0.35
Cement-treated base courses		
650 psi compressive strength or more	1.50	0.21
400 to 500 psi compressive strength	1.25	0.18
less than 400 psi compressive strength	1.00	0.14
Other base courses		
portland cement concrete	3.00	0.42
untreated dense-graded crushed aggregate	1.00	0.14
lime/fly ash treated, plant mix	1.00	0.14
lime/fly ash treated, road mix	0.75	0.11
soil cement	0.75	0.11
crusher run aggregate	0.75	0.11
Subbases		
gravel	0.50	0.07
sand	0.50	0.07

Table 6. Suggested layer coefficients for reclaimed fly ash bases for preliminary design as a function of thickness.

A. Base Thickness ≈ 6 inches

Source	Untreated	10% Raw Fly Ash	2-1/2% Lime
Ottumwa	0.12	0.13	0.14
Neal 3	0.13	0.14	0.15
Prairie Creek	0.13	0.14	0.17
Council Bluffs	0.13	0.14	0.19

B. Base Thickness ≈ 8 inches

Source	Untreated	10% Raw Fly Ash	2-1/2% Lime
Ottumwa	0.14	0.15	0.16
Neal 3	0.14	0.16	0.17
Prairie Creek	0.15	0.16	0.19
Council Bluffs	0.15	0.16	0.21

C. Base Thickness ≈ 10 inches

Source	Untreated	10% Raw Fly Ash	2-1/2% Lime
Ottumwa	0.16	0.17	0.18
Neal 3	0.17	0.18	0.19
Prairie Creek	0.17	0.18	0.21
Council Bluffs	0.17	0.18	0.23

D. Base Thickness ≈ 12 inches

Source	Untreated	10% Raw Fly Ash	2-1/2% Lime
Ottumwa	0.18	0.19	0.20
Neal 3	0.19	0.20	0.21
Prairie Creek	0.19	0.20	0.23
Council Bluffs	0.19	0.20	0.25

DESIGN PROCESS

- The design process for reclaimed ash bases is trial and error. A design thickness is assumed to establish a layer coefficient. A design thickness is then calculated. If they agree within ± 1 inch, the design is satisfactory. If not a new thickness is assumed and a design thickness recalculated.

CALCULATIONS

- Using the SN and layer coefficients determine the various layer thickness in inches (D_1, D_2, D_3) from the AASHTO equation:
  $SN = a_1D_1 + a_2D_2 + a_3D_3$
- When assigning layer thicknesses, economics and minimum layer thicknesses should be considered.

IOWA FLY ASH AFFILIATE RESEARCH PROGRAM - IOWA STATE UNIVERSITY

- Kenneth L. Bergeson P.E., Program Director - 515-294-9470
  Thickness Design Report ISU-ERI-Ames Report 98401 , January 1998

FIGURE 5 Pavement thickness design guide using reclaimed fly ash.

PAVEMENT THICKNESS DESIGN GUIDE USING RECLAIMED FLY ASH

Design Methodology

The AASHTO method suggested for low volume pavement thickness design using reclaimed fly ash is shown on Figure 5.

DESIGN COMMENTS

The layer coefficients assigned to the reclaimed fly ash are believed to be conservative considering their long term strength gain properties, so if the designing engineer has other experience on similar materials, a less conservative correlation, may be considered. It should be noted that bases with a high unconfined compressive strength (>800 psi) are considered as rigid bases and may contribute to reflective cracking in the asphalt surface layer (*9*).

The need for a subbase should be determined by the designing engineer based on the quality of the subgrade soil.

If the subgrade is so soft that stabilization (lime, fly ash, cement) is required, CBR tests should be conducted on the stabilized soil and the subgrade soil quality then determined from Table 1 shown on Figure 5 before proceeding with the design.

Determination of layer coefficients for similar base materials not included in this study may be made through simple laboratory testing, as described in the following steps.

a. If the material is not to be stabilized, or if its design unconfined compressive strength is expected to be less than 300 psi, determine the CBR value and select a layer coefficient from Table 1, page 3.
b. If the material is stabilized with lime or similar activator (expected strength greater than 300 psi) determine the 28 day unconfined compressive strength and select a layer coefficient from Table 1.
c. If the material is stabilized with Portland cement (expected strength greater than 300 psi) determine the 7 day unconfined compressive strength and select a layer coefficient from Table 1.

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Review of Cold In-Place Asphalt Recycling in Iowa

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A research project was undertaken to evaluate the performance of cold in-place recycled (CIR) asphalt projects in the state of Iowa. A sample of eighteen roads was selected. Then performance was evaluated in two ways: pavement condition indices (PCI) were calculated and overall ratings were given on ride and appearance. A regression analysis was extrapolated predict the future service life of CIR roads. The results were that CIR roads within the state of Iowa with less than 2000 annual average daily traffic (AADT) have an average predicted service life of fifteen to twenty-six years. Overall, CIR roads in Iowa are performing well. It appears that the development of transverse cracking has been retarded and little rutting has occurred. Contracting agencies must pay special attention to the subgrade conditions during project selection. Because of its performance, CIR is a recommended method for rehabilitating aged low volume (< 2000 ADT) asphalt concrete roads in Iowa. Key words: cold in-place recycling, rehabilitation, asphalt.

BACKGROUND

Cold in-place recycling has been in regular use in Iowa since 1986. It is a method to rehabilitate asphalt roads that, although structurally sound, have suffered from the effects of aging. It involves milling and crushing the existing asphalt pavement, rejuvenating the existing asphalt cement, and finally placing and compacting the material to a specified thickness. This process is performed in-place, so there is little need to load and transport the material. Detailed descriptions of CIR design and construction procedures are available elsewhere *(1)*.

This paper summarizes a study that reviewed the performance of CIR roads in Iowa *(2)*. The study commenced in 1996, ten years after CIR started seeing regular use in Iowa.

CIR ROADS IN IOWA

As of the 1996 construction season, 97 CIR projects had been completed in Iowa (Figure 1). Before recycling, most roads had most roads had 102 and 254 mm (4 to 10 in.) of asphalt materials which was weathered and heavily cracked. Generally the top 76 to 102 mm (3 to 4 in.) is recycled. After recycling, 51 to 102 mm (2 to 4 in) of hot mix is placed on top of the recycled material. Between 1993 and 1996, average cost of four inches of CIR for one kilometer of road (two lane kilometers) has been $8,000 to $10,000 ($13,000 to $16,000 per mile). The costs for 25 mm (1 in.) of hot mix asphalt overlay was about the same *(2)*.

Eighteen roads were selected for detailed analysis. For these roads, researchers conducted the performance surveys described below. During the selection process, researchers were careful to obtain roads with a variety of average annual daily traffic (AADT) counts, ages, and geographic locations. The AADT ranged from 300 to 2000. This is considered to be in the range of *low volume* traffic. The percent of trucks varied from five to eighteen percent. A variety of subgrade conditions were obtained by selecting roads from portions of the state that experienced a variety of geomorphological processes. Geomorphology refers to the process that formed the landscape. Contractors that have constructed CIR roads in Iowa use *similar* processes, and follow the same specifications. Due to the similarities, it is unlikely that the contractors had much influence on pavement performance. Table 1 gives information on the characteristics of the roads before and after recycling.

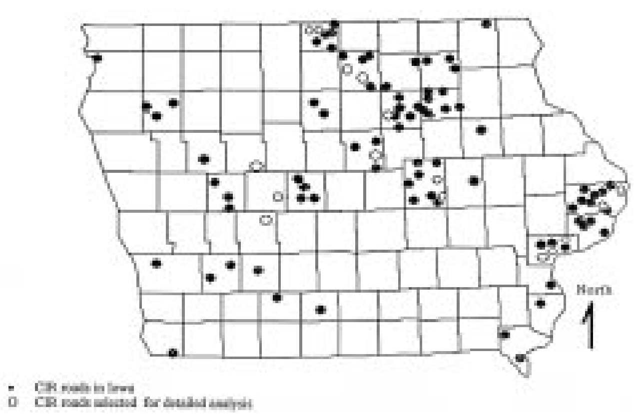

Figure 1 CIR roads in 1996.

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TABLE 1 Characteristics of Roads Before and After Recycling

County	Road	Age	AADT	Start Date	Finish Date	Existing Asphalt mm (in)	Base mm (in)	CIR	Emulsion	Hot Mix Overlay: Overlay mm (in)	Asphalt	AC %	Void %	Size mm (in)	Native Soil[1]
Muscatine	G-28	5	940	15-Sep	22-Sep	203 (8)	152 (6)	102 (4)	CSS-1	51 (2)	AC-5/10	4.9	4.5	19 (0.75)	3
Tama	V-18	5	550	24-Sep	11-Jul	0 (0)	152 (6)	102 (4)	HF-300RP, CSS-1	51 (2)	AC-5	6.1	6.5	13 (0.5)	2
Boone	E-52	5	290	25-Jun	28-Jun	203 (8)	152 (6)	102 (4)	CSS-1	51 (2)	AC-10	5.1	4	13 (0.5)	1
Clinton	Z-30	7	850	13-Jun	19-Jun	127 (5)	254 (10)	102 (4)	CSS-1	51 (2)	AC-10	6.5	6	13 (0.5)	2
Muscatine	F-70	3	950	26-Aug	20-Sep	102 (4)	203 (8)	102 (4)	CSS-1	76 (3)	AC-5	5.7	7	13 (0.5)	3
Butler	T-16	3	470	26-Jul	10-Aug	152 (6)	152 (6)	102 (4)	CSS-1	51 (2)	AC-5	5.5	7	13 (0.5)	2
Tama	E-66	6	1080	12-Jul	28-Jul	102 (4)	203 (8-pcc)	102 (4)	HF-300RP	51 (2)	AC-5	5.9	7	13 (0.5)	2
Clinton	E-50	10	520	Na	20-Aug	140 (5.5)	165 (6.5)	102 (4)	CSS-1	51 (2)	AC-10	5.1	4	13 (0.5)	2
Winnebago	R-34	6	620	18-Jul	30-Jul	152 (6)	152 (6)	102 (4)	HF-300RP	51 (2)	AC-5	5.7	4.1	13 (0.5)	1
Cerro-Gordo	B-43	7	570	30-Jul	7-Aug	152 (6)	152 (6)	102 (4)	CSS-1	51 (2)	AC-10	5.2	5.6	13 (0.5)	2
Boone	198TH	8	300	na	na	152 (6)	152 (6)	102 (4)	CSS-1	51 (2)	AC-10	5.7	3.6	13 (0.5)	1
Hardin	D-35	4	665	9-Jul	20-Jul	165 (6.5)	152 (6)	76 (3)	CSS-1	51 (2)	AC-10	5.5	6.5	13 (0.5)	1
Cerro-Gordo	S.S	6	600	8-Aug	17-Aug	203 (8)	152 (6)	102 (4)	CSS-1	51 (2)	AC-10	6.2	5	13 (0.5)	3
Winnebago	R-60	6	340	13-Jul	18-Jul	127 (5)	152 (6)	102 (4)	HF-300RP	51 (2)	AC-5	5.5	4.3	13 (0.5)	1
Muscatine	Y-14	9	990	22-Jun	4-Jul	152 (6)	152 (6)	102 (4)	HFE-150S, CSS-1, HFMS	64 (2.5)	AC-10	6	5.75	19 (0.75)	3

[1] Native Soil is defined from different geological regions:
Area 1 - Fresh glacial drift, no loess cover, areas of level terrain
Area 2 - Thin, discontinuous loess or loam over drift, gently rolling terrain
Area 3 - Alluvial features, thick sequences of clay, silt, sand and gravel

PERFORMANCE EVALUATION

Road performance was evaluated both quantitatively and qualitatively. The quantitative evaluation involves surveying the roads for severity and type of distresses. By analyzing the results at the network level, predictions may be made concerning the future performance of the road network for various maintenance and rehabilitation treatments. Economic analysis may be used to choose the best plan of action.

Public perception of road condition is based on qualitative assessments that are made as users drive over the roads. These perceptions are communicated to transportation officials who also drive over the roads and make qualitative assessments before they make maintenance and rehabilitation decisions. Thus it is reasonable to include both a quantitative and qualitative component to the performance evaluation process for CIR roads.

The quantitative assessment procedure for this project was developed using the method outlined by the US Army Corps of Engineers *(3)*. This evaluation technique requires an in-depth survey of the pavement surface. Surface distresses are recorded and classified by severity. The severity level is determined by guidelines that included in the USACE manual. Then numerical deductions are assigned for each severity and type of distress. These deductions are combined mathematically to provide a pavement condition index (PCI), which can range between zero and 100. The higher numbers indicating a pavement that is in better condition.

The qualitative assessment procedure was developed using guidelines from the American Association of State Highway and Transportation Officials (AASHTO) *(4)*. The qualitative evaluation provided separate ratings for appearance and ride (Table 2). The two evaluations were averaged to produce the PSI rating.

The results of the evaluations are provided in Table 3. The roads are sorted in order of increasing age. The PSIs ranged from 90 to 51 while the PCIs ranged from 100 to 52. The averages of the PCI and PSI ranged from 95 to 56.4. Clinton County E-50 was the first road that was recycled in Iowa. It had a PSI of 51, a PCI of 81 and an average of 66.2. A rating of 25 was chosen to be the point when a road should be rehabilitated. This corresponds to the rating where

TABLE 2 PSI Rating Scale

PSI	Ride Rating	Appearance Rating
100	New – No roughness or cracks.	Looks dark and smooth.
80	Minor roughness. Cracks cannot be felt while driving.	Minor cracks are barely visible.
60	Noticeably rough. Cracks can be felt while driving.	Cracks are clearly visible. Weathered surface.
40	Very rough	Tranverse cracks at short intervals. Longitudinal cracks present
20	Very rough. Difficult to maintain speed.	Heavily distressed. Tranverse, longitudinal and block cracking present.

TABLE 3 CIR Roads Surveyed 1996 and 1997

County	Road	AADT	Age	Alligator Cracking	Block Cracking	Edge Cracking	Longitudinal Cracking	Raveling	Rutting	Slippage	Shoving	Transverse Cracking	PSI	PCI	(PSI+ PCI)/2
Guthrie	IA-4	820	2	0	0	0	0	0	0	0	0	0	90	100	95.0
Butler	T-16	470	3	0	0	0	0	0	0	0	0	0	81	100	90.7
Calhoun	IA-175	1920	3	0	0	0	0	0	0	0	0	0	81	100	90.7
Muscatine	F-70	950	3	0	0	0	0	0	0	0	0	0	82	100	90.8
Hardin	D-35	665	4	0	8	0	0	0	3	0	0	8	65	85	75.0
Boone	E-52	290	5	0	0	3	0	0	0	0	0	2	73	95	83.8
Muscatine	G-28	940	5	0	0	0	0	0	0	0	0	2	73	98	85.3
Tama	V-18	550	5	0	0	0	0	0	0	0	0	0	70	100	85.0
Cerro Gordo	S.S	600	6	0	0	0	4	0	1	0	0	14	61	81	71.2
Tama	E-66	1080	6	0	0	0	0	0	0	0	0	6	61	94	77.7
Winnebago	R-34	620	6	0	0	0	0	0	0	0	0	10	63	90	76.3
Winnebago	R-60	340	6	0	28	0	0	0	0	0	0	0	63	72	67.3
Cerro Gordo	B-43	570	7	0	12	0	11	0	8	0	14	3	68	77	72.3
Clinton	Z-30	850	7	0	0	0	0	0	0	6	0	1	64	93	78.4
Greene	IA-144	1110	7	33	1	0	4	0	17	0	0	7	58	60	59.0
Boone	198TH	300	8	18	0	0	3	0	17	0	0	0	59	71	64.9
Muscatine	Y-14	990	9	0	0	0	5	16	9	54	0	7	61	52	56.4
Clinton	E-50	520	10	0	0	0	1	0	10	0	0	15	51	81	66.2

the USACE pavement condition scale changes from poor to very poor. This indicates that none of the roads that have been recycled thus far in Iowa are in need of rehabilitation.

With regard to rutting, only one third of the roads had deductions for rutting, as shown in Table 3. This indicates that rut depths were greater than 0.25 in for these roads. Of the roads that did have deductions, none exceeded 17. Explanations are available for the two roads that had rut deductions of 17. In Boone County, 198th Street was originally a cold mix pavement laid on earth subgrade. In locations where rutting occurred, the county engineer recalls the occurrence of equipment breakdowns that resulted in extra water being introduced in the RAP. In Greene County IA 144 was constructed during a cold, rainy period that was not favorable to CIR construction. Note that these two roads are the only roads that received deductions for alligator cracking. With these exceptions, rutting has not been a performance issue in Iowa.

The highest deduction for transverse cracking was 15. More than a third of the roads received no deduction for transverse cracking. The number of transverse cracks per kilometer ranged from zero to 126 (202 per mile). Figure 2 shows the frequency of transverse cracks. Only two of the roads exceeded 75 cracks per km (120 per mile) and more than two thirds had fewer than 36 cracks per km (58 per mile). Records were not kept regarding the original number of cracks per mile. However, it is the writers' experience that before recycling, most roads in Iowa have transverse cracks every 7 to 10 m (20 to 30 ft). This is equivalent to 110 to 160 cracks per km (175 to 260 cracks per mile). This would indicate that CIR has been effective in mitigating transverse cracks.

Muscatine County Y-14 has deductions for raveling and slipping. These deductions were the result of defects in the hot mix overlay and are not related to CIR performance.

How long will CIR roads last in Iowa? This is a difficult prediction to make in the absence of any actual experience. However, regression analysis was used to estimate the service life. First the analysis was performed on the age vs. PCI and age vs. PSI separately. Linear regression lines were extrapolated into the future and 95% confidence intervals were determined. The confidence interval indicates that if the relationship actually is linear, then with a confidence of 95% the mean response will fall within the interval. Thus the assumption is made that the roads will be maintained

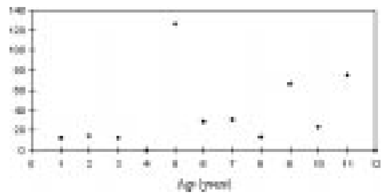

FIGURE 2 Frequency of transverse cracking.

TABLE 4 Indices Evaluation

Index	R^2	Equation	Minimum average life	Maximum average life
PSI	78.2%	=91.7-4.19*Age	14	19
PCI	49.4%	=114.97-4.84*Age	14	38
(PSI+ PCI)/2	72.9%	=101-4.56*Age	15	26

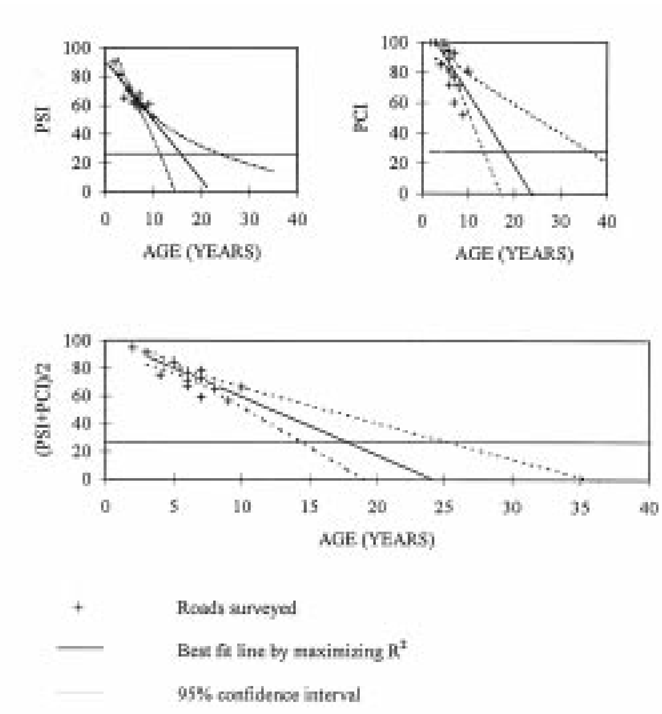

FIGURE 3 Life expectancy.

in the future so that a linear relationship between age and pavement condition will continue. If the roads are not maintained, sudden deterioration will occur at some point and the relationship between age and pavement condition will not be linear. Similarly, by applying very vigorous maintenance it may be possible to slow the deterioration of pavement condition with age, thus creating a non-linear relationship. It was also assumed that the roads reach the end of their service life, that is, they must be rehabilitated when the pavement performance index reaches 25.

The regression equations, R^2, and minimum and maximum average expected service life are shown in Table 4. The R^2 indicates the percentage of the variation of the data that is explained by the regression equation. A graphical representation of the regression analysis is presented in Figure 3. Considering the PSI only, the mean predicted service life is 14 to 19 years. Considering the PCI only, the mean predicted service life is 14 to 38 years. Considering the average of the indices, the mean predicted service life is 15 to 26 years. Since rehabilitation decisions are often a combination of a qualitative and quantitative input, the writers have chosen the average of the PSI and PCI as the best prediction. Note that the predicted lifetime will vary depending on the terminal index chosen for rehabilitation.

SUBGRADE STABILITY

The importance of proper project selection with regard to subgrade stability has been shown in Iowa. There have been three projects where recycling equipment “broke through” the remaining unrecycled asphalt due to a lack of subgrade stability: Iowa 92 in Adair County (May 1994), Louisa County G40 (May 1996), and Pleasant Creek State Park in Linn County (May 1997). These problems occurred during wet weather. In each case, subgrade conditions were so consistently inadequate that the contracting agency abandoned CIR and used alternate construction methods. Although such overwhelming subgrade stability problems are rare (only three projects out of more than 100 attempted), they can cause great difficulty for all concerned. Therefore, the research team developed quick and inexpensive test to detect possible subgrade stability problems during the planning stages of the project. The test involves drilling a two inch hole in the pavement with a hammer drill and taking a blowcount with a dynamic cone penetrometer *(2)*. The test procedure was verified during the 1997 construction season.

SUMMARY AND CONCLUSIONS

A systematic performance study has been conducted on a sample of 18 CIR roads in Iowa. They have been rated both quantitatively and qualitatively. The performance of these roads have been generally good. CIR has been effective in mitigating cracking and few problems with rutting were noted. Past experience has shown that subgrade instability is the worst problem that a CIR project is likely to face in Iowa. A dynamic cone penetrometer test was found to be effective in detecting such problems.

Specific conclusions are as follows:

1. A performance evaluation should be a combination of a systematically conducted distress survey and a qualitative rating based on ride and appearance.
2. In general performance of CIR roads in Iowa have been good. Current design and construction procedures appear to be adequate.
3. Few rutting distresses have been noted.
4. CIR appears effective in mitigating cracking. Little cracking occurs in the first five years of a typical road’s life. When cracks do develop, they have a lower frequency when compared to the road before it was recycled.
5. The predicted service life is 15 to 26 years based on a terminal index of 25.

6. A four-inch depth of recycling is equivalent in cost to one inch of hot mix overlay.
7. Subgrade stability failure is the problem that is most likely to stop a CIR project in Iowa.

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SUPERPAVE® Compaction

Brian J. Coree and Kera VanDerHorst

SUPERPAVE® has revolutionized the technology of asphalt mixture design and analysis. A significant element of this change has been the transition from impact compaction (Marshall) to that of gyratory compaction. The SUPERPAVE® Gyratory Compactor (SGC) has been developed from the Texas Gyratory Machine and the French Laboratoire des Ponts et Chaussees (LCPC) gyratory protocol. It is believed that the resulting mixtures more closely resemble those that have been compacted in normal construction practice. By recording the sample height throughout the compaction process, an entire history of compaction may be developed, in contrast to the traditional Marshall impact compaction process, wherein only the final compacted state of the mixture may be examined. There is, however, a fallacy in the method recommended by SUPERPAVE®: the resulting "compaction curve" cannot be considered to be fully representative of the compaction history of an "in-service" mixture. This is addressed in this paper. The SUPERPAVE® mixture expert task group (ETG) has recognized the concerns of state highway agencies and contractors and recommended that further strength tests be performed for Level 1 mixtures. The current direction for this additional testing is primarily focused on detecting unstable, or rut-susceptible, mixtures. The leading contenders for this type of testing are the many and various flavors of rut-testers: the Hamburg Rut-Tester, the Georgia Loaded-Wheel Tester, the Asphalt Analyzer, etc. These all require the use of added equipment and, in most cases, different sample compaction techniques. The authors propose a testing procedure using the SGC that is simple, direct and inexpensive. Key words: SUPERPAVE®, SGC, compaction, plasticity, rutting.

INTRODUCTION

A significant component of the SUPERPAVE® Volumetric (erstwhile Level 1) mix design protocol relies on the compaction procedure. This procedure uses the SUPERPAVE® Gyratory Compactor (SGC) which imparts a constant vertical pressure of 600kPa to the sample while rotating (or gyrating) the sample with an eccentricity of 1.25° from the vertical axis. It is claimed that this method of compaction results in a material that more closely resembles that on the road in terms of particle alignment and density (*1*). This laboratory compaction process proceeds through three landmarks: N_{init} which corresponds to the state of the mixture as the breakdown roller makes its first few passes; N_{des} representing the anticipated state of density in the mixture after 3 to 5 years of service; and N_{max} which represents a "factor-of-safety" condition should the traffic projections be seriously underestimated or the climate hotter than anticipated. The bulk specific gravity of the sample, G_{mb}, is measured after complete compaction to N_{max} gyrations. With this parameter and a knowledge of the maximum theoretical specific gravity of the mixture, G_{mm}, the volumetric parameters of the mixture (Va, VMA and VFA) are "back-calculated" for any degree of compaction ($0 < N_{gyr} < N_{max}$).

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SUPERPAVE® GYRATORY COMPACTION

The SUPERPAVE® compaction process was designed so that it "...would realistically compact trial mix specimens to densities achieved under actual pavement climate and loading conditions," and further "... that potential tender mixture behavior and similar compaction problems could be identified" (*1*). The first of these statements implies that the resulting compaction curve (% G_{mm} vs log(N)) will accurately represent the state of the mixture at any point in the anticipated life of the mixture, i.e., during construction and subsequently under traffic. Certainly the densities achieved by SGC compaction, measured on cooled samples, may more closely represent those measured on samples obtained from in-service pavement, although the recommended magnitudes of N_{des} may require "fine-tuning" (*3*). However, does the information presented on the SUPERPAVE® compaction curve (Figure 1) truly represent the state of the mixture during the laboratory compaction process?

During construction, the temperature of the mixture is typically in the range 80°C to 200°C. The greater part of the compaction is achieved while the mixture is in excess of 115°C. Under operating conditions under traffic, the mixture temperature may range (typically for Iowa) in the range -28°C to 58°C (as indicated by the grade of the binder, PG 58-28). In the laboratory, mixtures are compacted at an equi-viscous temperature ($T_{compaction}$) approximating a viscosity of 0.28 Pa.s throughout the compaction process. Thus the laboratory compaction procedure is reasonably representative of the construction compaction conditions, but not of in-service conditions. To what extent is this significant?

The compaction curve is being examined to determine to what extent it can be used as a surrogate "mixture suitability" test. Bahia (*4*) and others (*5*) are examining the properties of this curve. Bahia has in fact separated out the construction and trafficking components (differentiated at 92% G_{mm} or 8% air voids) and draws conclusions as to the suitability of the mixtures, separately for the construction and trafficked phases.

The authors accept the assumption that the state of the mixture during construction compaction is represented by the laboratory compaction curve up to a density of about 92% G_{mm}. Nonetheless, it should not be forgotten that this curve is developed based on sample height, and then back-calculated to provide the analytic density parameters; however, the trafficking, or in-service portion (% G_{mm} > 92%) reflects a condition of artificially elevated temperature during laboratory compaction. If it is assumed that at N_{des} the compaction is computed at 96% G_{mm}, the tacit assumption is that there are 4% air voids in the mixture. This is taken to be true at a temperature of 25°C or approximately in the cooled sample, but cannot hold at the elevated temperatures during compaction.

The compaction curve is typically a straight line, perhaps curving over to a more horizontal relationship beyond N_{des}. In Figure 1, the four conventional compaction curves (P_b = 4.2, 4.7, 5.2 and 5.7%) are essentially linear. Note, however, that the line representing the

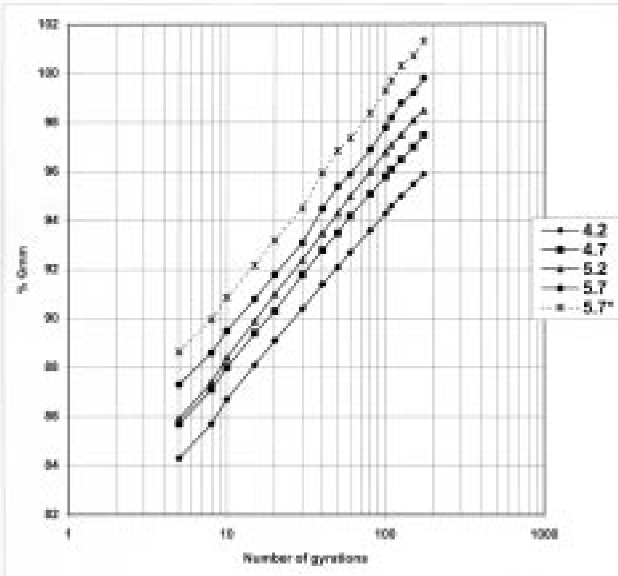

FIGURE 1 Typical SGC compaction curves (*1*).

"true" volumetrics (in this case at a $T_{compaction}$= 155°C) at P_b = 5.7 (noted as 5.7*) exceeds 100% G_{mm} at N_{max}! Even at N_{des} , the actual voids are 0.3% V_a and 97.8% VFA. These indicate that at this elevated temperature, the volume of the voidless mixture is greater than that of the mixture at 25°C when the bulk specific gravity was measured.

However, since the compaction curve is developed at elevated temperatures (~155°C), the volume of the binder during compaction is much greater than that assumed in the calculation of the G_{mm}. At a standard temperature of 25°C, the specific gravity of the binder may be assumed to be of the order of 1.020, *while at 155°C, the same material might have a specific gravity of only 0.933*, which is significantly different. In the design example given on pages 105-108 of reference (*1*), the compaction curves at different binder contents are all reasonably linear and compute to give air voids at N_{des} in the range 5.5% @ 4.2% P_b to 1.9% at 5.7% P_b and the voids filled parameter from 65% to 89% over the same binder range. Whereas during the compaction process, the actual air void contents at N_{des} can be computed to range from 4.5% at 4.2% P_b to 0.6% at 5.7% P_b and voids filled from 71% to 96%. This paints a very different picture that is not reflected by the compaction curve. The actual mixture condition during laboratory compaction (at 4.7% binder content) is V_a= 2.9%, VFA= 81% as opposed to the conventionally computed 4% and 74% respectively. A mixture with 2.9% air voids and voids filled of 81% might well be considered marginal, if not unstable, *but this potential instability is not reflected in the compaction curve*, and the picture only gets worse as the binder content increases (Figure 1). A summary of the reported and actual volumetric conditions for the example given are shown in Table 1.

TABLE 1 Reported Versus Actual Volumetrics of Example Mixture at N_{des}

Binder Content, P_b%	Reported Values V_a	VFA	VMA	Actual Values V_a	VFA
4.2	5.5	59.3	13.4	4.5	70.7
4.7	3.9	70.1	13.2	2.9	80.8
5.2	3.0	77.9	13.4	1.8	88.3
5.7	1.9	86.2	13.6	0.6	96.1

Rutting Plasticity

It is frequently assumed (*6,7*) that rutting plasticity, or permanent vertical strain, at N repetitions of load, e^p_N, can be characterized from the initial permanent strain, e^p_1, and the slope of the log(e^p_N) vs log(N) plot, in accordance with the following relationship:

$$\varepsilon^p_N = \varepsilon_1^p N^s \text{ or } \log(\varepsilon^p_N) = \log(\varepsilon_1^p) + S.\log(N)$$

It is not possible, however, to obtain neither a linear log-log relationship nor even a linear semi-log relationship using the recommended SGC. Thus, the SGC data does not conform to the rutting models adopted or investigated by SUPERPAVE® (*6, 7*).

The inability of the SGC compaction curve to highlight plastic instability is attributed to the fact that the mixture is so effectively contained within the relatively infinitely rigid walls of the mold and the equally rigid top and bottom platens that the type of lateral plastic flow observed in rutting pavement is totally prevented, even though the actual state of the mixture may be wholly plastic during laboratory compaction (e.g. at 5.7% P_b , V_a= 0.6% and VFA = 96.1%.)

CAN THE SGC BE USED TO EVALUATE PLASTIC INSTABILITY?

Given the above statements, can the SGC be used to evaluate plastic instability? The answer is yes, but with some qualification. Alternate equipment (the Hamburg Rut-Tester, the Georgia Loaded-Wheel Tester, the Asphalt Analyzer, etc.) allow lateral plastic flow to develop under the load by running tire-like loads over compacted slabs of asphalt, thereby more closely simulating actual performance in the pavement. More theoretical material models must be used in conjunction with the Superpave Shear Tester (SST) to measure "fundamental" mixture response parameters in order to yield more accurate rutting models. To therefore use the SGC, a means must be found to permit the same type and degree of plastic response in the mixture sample. The authors propose the method described below. This method is under current development and pilot-testing at this time and it is recognized that much work remains to more closely define operational standards and performance-related criteria.

Under current operational procedures, an "optimum" binder content is defined by using the method recommended under the SUPERPAVE® Volumetric design protocol. The resulting candidate mixture is further tested for resistance to stripping by AASHTO T-283. It is proposed that an extra batch (or batches) be made up at this stage and tested as follows:

- Each sample is compacted in the usual fashion (at $T_{compaction}$, 600kPa and 30rpm) to 92% G_{mm}, based on data already obtained; thereby representing compaction during construction at appropriate pressures and temperatures.

- The mold, complete with sample, is placed in a controlled oven, and brought to an operational temperature. This temperature is currently proposed as the high temperature used in the appropriate PG grading, i.e. the average 7-day maximum temperature at a depth of 20mm in the pavement. Preliminary testing has shown that it takes approximately 12 hours to achieve a state of thermal uniformity throughout the sample.
- The mold and sample are replaced into the SGC, but instead of the full 150mm dia. (nom.) top platen, a modified top platen with a reduced contact area (typically 100mm dia.) and about 25mm deep is placed between the sample and the top platen. The pressure may be adjusted to simulate different design tire pressures, for example, instead of the 600kPa on the full 150mm dia. top platen (87psi), indicated pressures of 307kPa (100psi on the reduced contact area) or 368kPa (120psi) may be used. This then simulates a *project specific traffic compaction scenario*, while allowing an annular gap of 25mm (nom.) around the axial loading plate for plasticity to develop.
- Compaction (at $T_{compaction} = T_{average\ 7\text{-}day\ maximum\ pavement\ temperature}$ and 30rpm) is continued (currently for 250 gyrations). The sample height is recorded continuously. Since the sample is no longer uniform in height or density, comparison to G_{mm} is not appropriate.
- Normalize the sample height to percent vertical strain (ε = 100 x [(height at N=1) - (height at N)]/ (height at N=1).
- Plot log(vertical strain) versus log(N): it will be observed that this relationship is essentially linear.
- By linear regression, obtain the values of the intercept, ε^p_1, and slope, S. These correspond to the SUPERPAVE® rutting model parameters and can be used in the same manner to predict rutting.

Preliminary Test Results

A preliminary set of tests has been performed using this protocol on an arbitrary aggregate gradation. This pilot study was undertaken in order to check that (a) the method itself did not compromise the equipment by testing mixtures at 58°C which are much stiffer than those tested at 155°C, and (b) that the resulting log(vertical strain) vs log(N) would be indeed be linear in accordance with the SUPERPAVE® prediction models.

Mixtures were made up at 4.0, 4.5, 5.0, 5.5 and 6.0% binder contents. The asphalt binder was a “generic” AC-10 available in the laboratory. In order to expedite the pilot study, the specific gravities of the blended aggregates were not determined, consequently the VMA and VFA parameters derived below are based on aggregate effective specific gravity, and are thus biased. The conventional volumetrics at N_{max} = 150 are summarized in Table 2.

Each mixture was tested according to the protocol outlined above. The log(vertical strain) vs log(N) results were plotted and examined. From N > 10, the relationships were closely linear and are shown in Figure 2 and therefore meet the general expectations of the SUPERPAVE® rutting model. A further plot of the vertical strain against binder content, P_b, at various levels of traffic (or N) is shown in Figure 3, in which it is clearly demonstrated that the mixtures become significantly more plastic at binder contents in excess of 5.0% at all levels of trafficking (or N). This plot is wholly complementary to results reported by Monismith and Vallerga on a similar study in 1956 (*9*).

TABLE 2 Sample Volumetrics Summarized

Binder Content, P_b%	V_a%	VMA %	VFA %	N(@ 92% G_{mm}
4.0	4.2	15.3	72.8	45
4.5	2.9	15.1	80.6	29
5.0	1.4	15.2	91.0	16
5.5	1.0	16.1	93.6	11
6.0	0.1	16.7	99.1	7

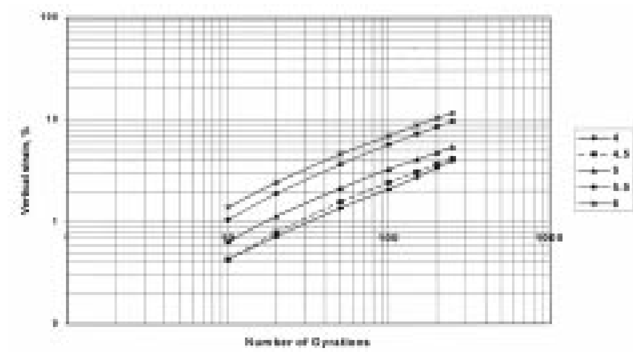

FIGURE 2 Strain development in proposed protocol.

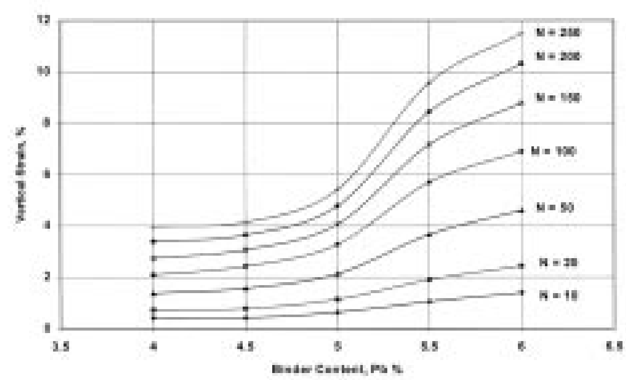

FIGURE 3 Observed strain development by binder content.

Conventional SGC Protocol Revisited

At this point, it was realized that trying to obtain a linear log(vertical strain) vs log(N) plot from the conventional SGC compaction curve (to N_{max} at $T_{compaction}$) was not necessarily appropriate. The initial state of these mixtures is not, in fact, known. It is likely that the richer mixtures will self-compact more than drier mixtures due to charging the mold even before loading begins. Using the recorded compaction information, the strains were recomputed using the heights at 92% G_{mm} as the reference height. The results are shown in the log-log plot in Figure 4, in which it is seen that potentially linear trends are being modified at higher levels of compaction such that strains become almost asymptotic to a horizontal (con-

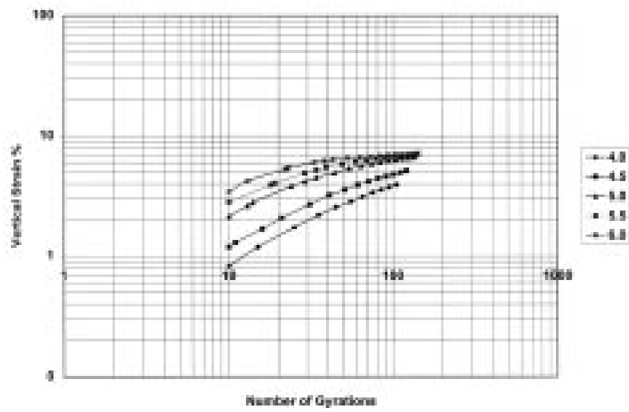

FIGURE 4 Constrained strain in SGC.

stant) value between 7 and 8%. This fresh observation lends credence to the earlier assumption that the extreme rigidity and confinement provided by the mold walls and loading platens is preventing the development of lateral plastic flow.

CONCLUSIONS

The conventional SGC compaction curve cannot be used to identify rutting plasticity in asphalt mixtures. Truly unstable conditions may indeed occur during laboratory compaction and are not so identified.

A modified protocol is proposed by which plastic instability can be readily measured and identified. This method can also yield the parameters required by the SUPERPAVE® rutting model.

Further development and refinement of the method is needed before definitive statements cat be made as to its utility in practice, in particular, validation of the method against mixtures of known rutting sensitivity will be required.

FURTHER WORK

Further testing of both laboratory and real mixtures will be undertaken to refine the operating parameters of this protocol. Real mixtures with known rutting performance (both the good and the bad) will be tested to validate the method and, hopefully, to define performance-related criteria of acceptability.

The geometry of the testing set-up lends itself to a more complete analysis of the states of stress and strain during the test. This may provide a more "rigorous" validation of the testing environment. The strain vs loading results obtained so far appear to conform to some of the concepts of mixture rheology, and may be amenable to a more focused analytical model of rutting instability in asphalt mixtures.

ACKNOWLEDGMENT

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Strategic Issues for a City Roadways Re-Engineering Project

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This paper discusses the issues arising from a re-engineering project that was successfully completed in 1998. The project commenced in 1993 with the objectives of developing and implementing needs based budgeting tools, data and business processes for the road network in the City of Saskatoon. The project implemented network and project level Pavement Management Systems, a Maintenance Management System, a zero-based budgeting business process and a method to annually assess the impact of decisions on the value of the road network. The paper discusses the organizational issues addressed including: Developing re-engineering strategy; creating re-engineering teams; identifying internal champions for the change; designing needs based business processes; developing accurate measure of condition to drive all physical surface work; developing management tools and processes. The paper focuses on the critical success factors and the major lessons learnt from the total four year project.

BACKGROUND

During 1993, the City of Saskatoon participated in a world-wide review of pavement management system tools. This review culminated in a one-week seminar hosted by the University of Saskatchewan in late 1993. During this seminar, 25 of the key city personnel were exposed to modern asset management principles and practices. At the completion of the seminar, the City decided to move towards implementing pavement management systems at both the Network Level and Project Level.

This required the City to:

- design and implement a detailed condition rating process for the street network from expressways down to local streets;
- develop performance models and cost models for the actual physical performance of the network;
- develop modern business practices that would enable the City's budgetary and accounting processes to fully maximize the advantage of the new technology.

This paper focuses on the processes used to develop the new business practices.

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KEY ISSUES

After initial implementation of the tools and technology, the City identified that the core business processes of the City would have to change. Otherwise the new technology could not be successfully implemented.

Following are the key issues that needed to be addressed:

- There were varying levels of understanding of asset management principles and practices;
- There were no uniform systematic methods for work planning and delivery;
- There was no uniform process for reporting outcomes of expenditure;
- There were no program objectives or performance measures;
- There were no meaningful uniform measures of efficiency and effectiveness.

The Challenge

Council was faced with similar decisions on an annual basis about the level of investment in preserving the road assets of the City. The only measure of this core activity was the financial expenditures. There was no measure of outcomes of the expenditure of the previous years preservation budget (a mixture of a variety of budgets including parts of Capital and Operating). A new process had to include performance measures that would ensure accountability for the budgetary decisions of the City (See Figure 1).

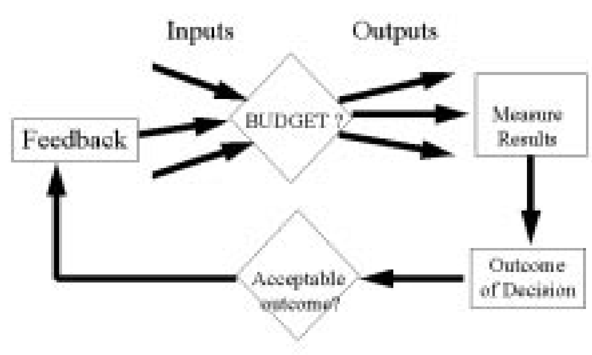

FIGURE 1 Decision process.

THE PLANNED APPROACH

In order for the City to effectively address the above issue, there was a need for a planned approach to changing the way the City does its business. The change had to be managed and objective based. There was also a key requirement that there was to be maximum input from all the internal stakeholders into the changed business process.

To facilitate the planned approach to implementing the new business processes, it was decided that the following areas need to be addressed:

1. Education
2. Identification of internal advocates and future champions of change
3. Tasks:
- Draft asset management policies
- Learn how those policies could be satisfied
- Develop procedures to enable people to comply with policies

4. Implementation training

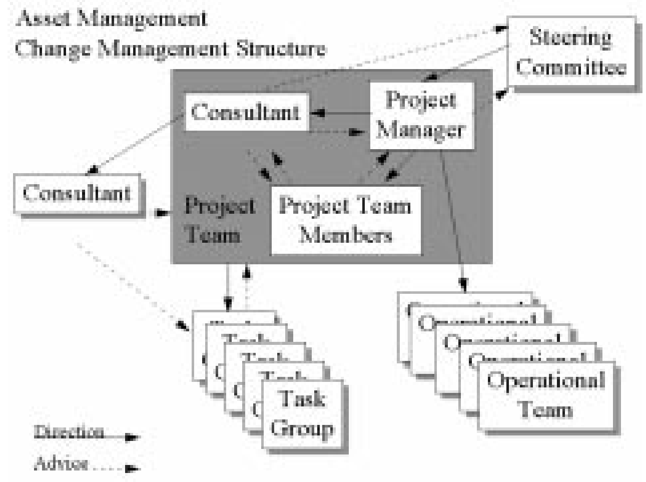

FIGURE 2 AM Change management structure.

CHANGE MANAGEMENT STRUCTURE

The stakeholder input for this project that institutionalized the end product with maximum buy-in from the internal stakeholders consisted of the following:

Stakeholder Group	***Change Management Role***
Sponsor:	
Works Committee	Ensured direction of project was consistent with City goals Promoted asset management at executive level of City Provided support for the project
Steering Committee	Confirmed the strategic direction of the project Removed internal/external “roadblocks” for project team
Project Manager	Managed project budget, deliverables, and time Co-ordinated project activities Reported to Steering Committee Provided direction on impacts across the Department
Project Team	Reviewed all starting point documents for task groups Reviewed all outputs from task groups Primary force of change Assessed progress of project
Task Groups	Determined “fit for purpose” solution for specific asset management needs Starting point defined by Project Team; reviewed by Steering Committee
Operational Groups	Performed specific activities

The above stakeholder input required in excess of 30 personnel from across the City to be involved in all aspects of the project. The description of how these groups interacted is in Figure 2.

As can be seen from the Figure, the project team members and the project manager are the key personnel that influence the outcome of the project. They are, in effect, the “engine room” of change for the overall project. The specialized input from accounting and operational fields within the City were achieved by a diverse group of people being involved in specific tasks within a task group. It is also worth noting that the consultant was part of the project team and also the consultant was external to the project team in providing advice. This meant the consultant wore two hats during the project.

The project team identified all of the implementation objectives and reviewed and/or developed all of the starting point documents for all task groups. The project objectives are discussed in a later part of this paper as are the task groups. The project team also identified the key requirements for the top three levels of management throughout the City. This was to become the project scorecard for completion of the project. The details of the project scorecard are also discussed later in the paper.

TASK GROUPS

Overall, eight task groups were identified as being needed to ensure input from the internal stakeholders regarding specific issues related to the City’s processes. The eight task groups were as follows:

Task Group–Budget Process	Convert to needs based budget process
Task Group–Capitalization Team	Required knowledge of balance sheets, CICA rules.
Task Group–Performance Performance Measures	Measured efficiency, effectiveness of resultant process

Task Group 4–MMS Implementation Strategy	
Task Group 5–PMS Implementation	Institutionalize the technology
Task Group 6–Communication	Develop reporting strategy to external stakeholders
Task Group 7–Role and Responsibility	Review roles, responsibilities vs. policies, procedures
Task Group 8-Information Systems Support Strategy	Develop ongoing support for information systems developed/implemented as part of the project.

Task groups were given a starting point document developed by the consultant and the project team and they met for several days in a closed setting to ensure unobstructed focus on the task. They were the working groups for the project and tended to have strong ownership of the results.

The task groups identified the following issues:

- Roadway preservation business must compete with all other service areas;
- Business of preservation was different from all other business in the City of Saskatoon;
- Current budget processes had absolutely nothing to do with the condition of the network;
- City needed re-engineering of the budget process and not just tinkering with budget process.

All task groups worked diligently on their allocated task. Almost all had a starting point document that defined a “straw man” business process that they could use as a starting point for their deliberations. All task groups came up with a final report or business process for their allocated tasks.

IMPLEMENTATION OBJECTIVES

At the very start of the re-engineering project, the project team and steering committee believed it was important that the overall project have firm and universally agreed-to implementation objectives that were to become the guiding principles of the project. The consultant brought to the project implementation objectives from similar re-engineering projects around the world. Those objectives were then reviewed by the project team and finalized into the nine discussed in this paper. The steering committee and the Council of the City agreed to the implementation objectives. The implementation objectives are as follows:

1. Understand Asset Management Principles and Practices

This would result in all levels from Maintenance Supervisor to Director Works and Utilities and External Stakeholders having a more uniform and modern knowledge of the practices and principles of Asset Management.

2. Be Able to Develop a Needs Based Budget for all Preservation Works

This would enable the City to determine the cost of maintaining, or of improving and degrading the current network condition or any part of the network. It was recommended the budget should include maintenance and rehabilitation works into one preservation program.

3. Communicate the Benefits of Asset Management to External Stakeholders

The City would be in a position after the implementation to articulate what had been done by the City to improve its accountability for asset management. Capitalization of the asset (determining the written-down replacement cost of the asset annually) was an example of increased accountability to external stakeholders.

4. Present Budget in an Understandable and Relevant Way to External Stakeholders

The budget must be able to be defended. Therefore, the supporting information for the budget must be able to be audited and understood by the decision-makers.

5. Develop and Implement Asset Management Policies and Procedures to Institutionalize Asset Management

Large organizations require documented policies and procedures for good things to work in the long-term. Once the initial enthusiasm has diminished and the immediate organizational priorities change, documented policies and procedures ensure the core business continues to function.

6. Respond Positively to an External Audit

Some Provincial Departments had already been subjected to a value for money audit. An external audit is an opportunity to demonstrate the level of control an organization has over its activities. At the completion of the project, the City was able to demonstrate the logic of all decisions and the links between decisions and the City’s goals.

7. Fully Account for the Outcomes of the Expenditure of Asset Management Funds

This would enable the City to compare expected performance with actual performance. As an example, short and long-term targets were set for condition of the network based on various budget scenarios. After the budget had been spent, the City was able to compare the condition expected with the condition achieved.

8. Develop a Five Year Network Level Maintenance Plan

The plan would include expected improvements in decision-making as well as the predicted performance of the network. As an example, if funding were to be increased in the short term to improve the condition of certain types of pavements there may be a net reduction in overall preservation costs in the long term. The plan needed to be able to be understood by non-technical people such as politicians.

9. Be More Objective in Asset Management Processes

A high level of subjectivity appeared to be associated with many of the asset management processes within the City. This subjectivity may be the best way of making the best decisions, however, as this cannot be proved there will always be some doubt. An example of improved objectivity was the manual condition rating system that was implemented. The maintenance workforce had always inspected road conditions, however, now they were able to all do it in a more systematic and uniform way. This minimized personal bias associated with subjective decision-making. The goal was to be more objective, not totally objective. Skilled peoples' opinions will always be of value in a well run business. However, the more people involved in the process, the greater the need for Quality Assurance of processes.

PROJECT SCORECARD

In order to move from the high level project objectives to some tangible set of specific objectives that could be understood by the large cross-section of people involved in the project, the project team and consultant developed a list of questions that were believed to be questions that one of three levels of management could reasonably be expected to ask. The new business processes and technology should enable each of those questions to be answered. This, therefore, became the project scorecard and much of the focus of the design of the business processes had to do with being able to provide answers to the questions. The questions were stratified into three levels, the highest level being the Council requirements. The Council is the elected representative of the City. They are responsible for all the executive decision-making of the City. The Council does have an active involvement in the management decisions of the City.

The next level of management was the Director of Works and Utilities. This position was the most senior engineering position within the City and was responsible for all the technical aspects of the operations of the City. The questions developed for this level of management were structured to be appropriate for the current business needs of that level of management. The third level was the Engineering Department Requirements which is the tactical level of the City's operations. Much of the short and medium term planning was the responsibility of this level of management. Delivery of services was also a key accountability of this level of management. Therefore, again, the questions were structured accordingly.

Council Requirements

To determine if an asset management framework was viable, the following questions needed to be addressed:

- What does the road network needs in terms of public money to ensure responsible allocation of limited resources for the overall good of the community?
- How much money is required to maintain current condition on certain roads whilst improving condition on others?
- What value for money did the community receive from the expenditure of funds on preservation works?
- What is the value of the infrastructure?
- What impact do the decisions of council have on the value of the road network?
- What is the impact of funding or resource constraints on the long-term condition of the road asset?
- Are we creating problems for future generations by our current actions?
- How will the condition of the asset change with different budget levels?

Director Works and Utilities Requirements

To determine if MMS and PMS were viable the following questions needed to be addressed:

- What is the current and future condition of the asset in relative terms?
- What funds are required now and in the future to maintain the network at a given standard?
- How are the various sub-elements of my network performing?
- What injection of funds is needed to improve condition and where should it be allocated for maximum effectiveness?
- What additional level of funds would be required if a short term reduction in preservation funding were imposed?
- Are standard methods applied to all preservation activities uniformly across the City?
- Are all preservation actions both effective and efficient?

Engineering Department Requirements

To determine if MMS and PMS were viable the following questions needed to be addressed:

- Where is potential for most effective expenditure of funds?
- How good or bad is any part of network compared to the rest of network?
- Most effective program requirements for manpower, plant and materials, and are these viable?
- What treatments are available and what are their consequences?
- What is most cost effective treatment for a particular segment?
- What are repercussions of changing the work program?
- Are all activities planned and budgeted to ensure optimal use of financial resources?
- Are all preservation activities performed at a better productivity than last year?

- Is at least 80% of completed preservation work program in accordance with original work program?
- Are work plans for crews consistent with overall preservation work plans for the City?
- Can the Engineering Department demonstrate that it is doing a good job in asset management?

CONCLUSIONS

Due to the massive scale of change and the vast level of resources required to deliver the project effectively, the project was identified as a long-term project. This meant that the overall project of re-engineering was to occur over multiple years. The complete re-engineering project commenced in 1995 and, at the stage of writing this paper, all the business processes have been designed and charted. Many of the policies and procedures have been documented and a full quality assured condition rating process has been implemented. Also, the City has moved towards documented standard work practices for all of the operational field staff covering all the physical assets of the City from Roadways through to Water & Sewer, Parks and Transportation branches.

The work that the Roadways branch of the City has done in the re-engineering project has been seen as a showcase for other branches. Much of the work that has been done for Roadways is now being reviewed and adapted to the other branches of the City.

It is anticipated that the project will be completed in 1998 with full documentation of all the policies, procedures and standards for the business process being presented to Council for ratification in an Asset Management Manual. The manual will include all the business processes that have been designed in the project.

It is worth noting that much of the new approach to managing the asset has already been implemented by City management, even prior to finalization of all the details.