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EXPANDING THE USE OF PAVEMENT MANAGEMENT DATA

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ABSTRACT

The process of collecting pavement data has been evolving with advances in technology, thus generating huge amounts of data to be stored in pavement management systems (PMS) databases. The rapid size increases of these databases presents a challenge for state agencies, as they attempt to understand and take advantage of the data to support pavement maintenance and rehabilitation decisions. The knowledge discovery (KDD) process and data mining are being applied to large pavement data sets with the objective of extracting useful information. For example, the data can be used to predict pavement serviceability ratings (PSR), as an indirect way of obtaining the remaining life of pavements. The Missouri Department of Transportation (MoDOT) provided pavement condition data from 1995 to 1999 to be used in the study. Research is being conducted in which results from the whole set of data will be presented and interpreted in order to obtain a better view of the condition of pavements in Missouri and be able to increase the effectiveness of the decision-making process. Parameters needed for future work that would improve the quality of information extracted are also presented.

INTRODUCTION

With advances in technology, more sophisticated equipment is being developed and adopted for the collection of pavement data. Not only has the equipment changed, but decision-making systems are developing to take advantage of the new technology.

Transportation agencies have always managed their pavement. However, it was not until the enactment of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 that it was required that all states have a pavement management system (PMS). Although the requirement was amended in 1995 by requiring that only all roadways eligible to receive Federal-Aid Funds would be covered by a PMS, most agencies have and use a PMS (*1*). The Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), the American Public Works Association (APWA), and the World Bank have provided guidance and encouragement to aid the transportation agencies in the process of developing a more comprehensive and functional PMS structure (*1*).

An effective PMS must have a comprehensive database. The database must contain reliable, objective and appropriate information in order to assist in any decision-making for planning and budget procedures. In addition, it must contain an inventory of the state's highways. With the increase in automated data collection due to both the developments in technology and growth of the highway system, the size of these databases has increased as well. Therefore, new methods or techniques are needed to assist analysts in the process of discovering useful information and knowledge in these databases (*2*).

Knowledge discovery in databases (KDD) is a process that provides the techniques and tools needed to understand large data sets. Such techniques provide "intelligent" assistance to humans in order to improve the ability to understand these data sets. The term "data mining" describes the methods used in some of the steps of the KDD process (*3*). The application of the KDD process and data mining to pavement management data will enable the identification of characteristics common to certain pavements with the goal of predicting future performance of pavements.

The primary objective of this research is to apply a systematic approach to a large pavement management database to extract useful information from pavement management data. This additional information has the potential to further increase the accuracy and efficiency of the decision-making process. In addition, the research will help agencies optimize the use of the data in the PMS databases, and evaluate examples of real data measured and analyzed by state Departments of Transportation (DOT).

The paper provides background on KDD and data mining, PMS, and data integration, followed by a description of the research approach and a case study using data from the Missouri Department of Transportation (MoDOT).

BACKGROUND

The following sections give a summary of the type of problems that data mining is often used to solve, an overview of the concept of PMS, a brief description of the evolution of the data collection process, some examples of the equipment used for the collection of pavement condition data, and a summary of the process of data integration in a PMS.

KNOWLEDGE DISCOVERY AND DATA MINING

In general, the term "data mining" includes all tools engaged to help users analyze and understand their data. However, a more specific definition is given by Moxon, "a set of techniques used in an automated approach to exhaustively explore and bring to the surface complex relationships in very large databases" (*4*). The purpose of data mining is to help find useful patterns in large databases by means of algorithms (*5*). Its applicability to a variety of problems, such as databases containing consumer and transaction information and advanced databases on multimedia, has recently raised the interest of research in this area (*6*). However, with the rapid growth of databases in the last decade, our abilities to understand and explain the data have decayed, creating opportunities for the use of knowledge discovery in databases (KDD).

KDD is a process that provides the techniques and tools needed to assist in the understanding of large data sets. The techniques are said to provide "intelligent" assistance to humans in order to improve the ability to understand these data sets (*3*). Fayyad goes on to describe the relationship between KDD and data mining, saying that, "...KDD refers to the overall process of discovering useful knowledge from data while data mining refers to the application of algorithms for extracting patterns from data..." (*3*).

Data mining typically has one of two goals, prediction or description. The term "prediction" refers to the process of using variables from the database to predict unknown values for the variables of interest, and "description" refers to the process of finding "human-interpretable" patterns to describe the data (*3*). Both are useful in civil engineering applications, for example, in modeling pavement deterioration or in looking for common characteristics of damaged pavements.

Some of the more specific tasks that data mining algorithms can perform on databases include (*6*):

- **Association** – finding relationships between different attributes, often in large customer databases. For example, an association might be that a customer who buys item A is also likely to buy item B. The idea is to figure out how to determine the presence of some sets of items, given the presence of other items in a transaction;

- **Clustering** – grouping data into ad-hoc categories with considerable similarities between items in the same group. These algorithms are mostly used for descriptive tasks. Such algorithms give the user a good idea of both the identity and nature of the similarity of the different points in each cluster; and

- **Classification** – dividing the data into pre-defined classes that can be either categorical or quantitative. In the quantitative context, classification is viewed as a type of regression. This type of algorithm is generally used in prediction problems since it creates a model based on known data.

Other data mining techniques that are generally accounted for in the classification model include:

- **Decision Tree** – used to examine the data and induce the tree and its rules that will be used to make predictions. It involves "splitting" the information into categories containing examples of a particular class with the purpose of maximize the "distance" between groups at each split (*7*);

- **Neural Networks** – used as a means of efficiently modeling large and complex problems in which there may be hundreds of predictor variables that have many interactions. It involves a network of neurons to process one or more values into a single output, which is called the "class label"; and

- **Bayesian Classifiers** –used mostly for applications in which there is a large number of variables, for example, text classification. It assumes that the outcome of an attribute value of a given class is independent of the values of the other attributes.

Pavement Management Systems (PMS)

Pavement management systems (PMS) are the key instruments that provide decision-makers at all management levels with strategies to maintain pavements in acceptable condition. Various activities are involved in the process, such as planning, design, monitoring, maintenance, reconstruction, budgeting and programming, construction, research, evaluation, and rehabilitation. A comprehensive and efficient PMS will permit agencies to achieve both national and local objectives in delivering acceptable highway services (*1*).

AASHTO describes a pavement management system as "a set of tools or methods that can assist decision-makers in finding cost-effective strategies for providing, evaluating and maintaining pavements in a serviceable condition" (*8*). However, selecting the data that will support those decisions must meet certain criteria. That is, even though the tools for collecting huge amounts of data are available, the data to be collected should be chosen cautiously. According to Paterson and Scullion, these criteria deal with relevance, reliability, affordability, and appropriateness (*9*). The reliability of the data is a key element in a PMS database. The accuracy, spatial coverage, completeness, and currency are the factors that determine such reliability.

Pavement Data Collection

The main goal of highway departments is to provide roadway users with an "acceptable" highway system at the most inexpensive cost. Therefore, a series of investigations and simulations are performed based on the data collected to better understand the behavior of pavements, as well as to improve the conditions of the roadway system. Another use of these data is to establish priorities for the maintenance of each roadway. Therefore, the collection of pavement condition data facilitates the decision-making process, by providing insights used for highway maintenance, rehabilitation, and reconstruction.

Historically, pavement management decisions have relied on data collected through visual inspection and judgment based on experience (*10*). Public agencies have found that manual inspection of pavements requires extensive effort and that it is difficult to pass experience along to younger engineers. The collection of pavement condition data is costly and time consuming; as a result, the use of automated methods for data collection has been expanding. Such methods are designed to meet the evolving management requirements of today.

At present, more state of the art equipment is being used, in which data is being collected automatically. However, some manual collection and visual inspection of pavement data is still conducted as integrated parts of the same process.

The type of data collected can be grouped in four major areas (*10*):

- **Roughness** - irregularities found in the surface of pavements that affect the ride of a vehicle. This type of data is the only measure used by some agencies for network level applications, in order to evaluate the serviceability of pavements.

- **Surface Distress** – the extent of pavement fracture, distortion, and disintegration. The three major components of pavement distress are cracking, rutting, and longitudinal profile. It is used by agencies to evaluate the deterioration, the overall composite index, and the maintenance needs of pavements for both network and project level applications.

- **Structural Evaluation or Deflection** - the ability of a pavement to support traffic. It is used by agencies for the evaluation of material properties and structural capacity of pavements for both network and project level applications.

- **Skid Resistance or Pavement Friction** - "the horizontal force developed when a tire that is prevented from rotating slides along the pavement surface" (*11*). It is used for the evaluation of safety against skidding in both network and project level applications.

Other types of data, such as ride quality, appearance, traffic, costs (e.g. construction, maintenance, user), location reference, geometric and structure data, and environment (e.g. climate, pavement temperature, drainage, water below surface, freeze/thaw), are also monitored and evaluated by agencies (*12*).

Automated Data Collection

Agencies have been changing their methods of collecting pavement data from manual to automated, as technology has evolved. Another significant factor in the use of more automated methods for the collection of data has been the growth of the highway system. The staff needed to inspect and collect data manually from today's system would be tremendous, but trying to accomplish it within a reasonable time frame would be even more challenging. Other benefits of automated data collection are the accuracy of the data and efficiency of the method. In addition, it allows easy and flexible output of multiple parameters (*13*). Therefore, the implementation of an automated data collection method ensures a more comprehensive database, which is the key element of a pavement management system.

Boettcher explains that an automated method of data collection is needed to "meet evolving management requirements" (*14*). That is, with the development of new technologies, more advanced techniques for PMS will be employed, and thus better methods of data collection will be developed.

Equipment

There are several types of automated equipment for collecting pavement surface distress and/or pavement structural strength information. Some of the equipment that are currently being used by state DOTs include:

- **Prolfilometer / Rutbar Van** – collects ride quality data using lasers and transverse profile data using ultra-sonic sensors,

- **Skid Systems** – measures surface friction characteristics,

- **Falling Weight Deflectometer (FWD)** – collects pavement surface deflection data,

- **Ground Penetrating Radar Van (GPR)** – collects data, in a non-destructive manner, to determine the upper surface layer thickness, and presence of certain types of pavement anomalies such as stripping or presence of moisture (*15*),

- **Multi-Function Vehicle (MFV)** – collects video images of the pavement surface and the right-of-way, and other information, such as ride quality, rut depth, and global positioning system (GPS) data, and

- **Automatic Road Analyzer Vehicle (ARAN®)** – collects longitudinal and transverse profiles, grades, cross-slope, pavement texture, pavement distress, GPS coordinates, and video images of the right-of-way and pavement in a single pass (*16*).

The practice is to combine some of these equipment to gather the pavement surface distress and pavement structural strength information in order to obtain a complete set of data.

Data Integration

Data integration refers to the issues related to obtaining more information from the data collected (*17*). The general issues include data collection technologies and practices. Technologies are combined to improve the quantity and/or quality of the data collected. For example, GPS coordinates are being used to link the data collected from the ARAN® vehicles to state highway locations to reproduce the location of "link points" between the two data sets (*18*). Likewise, different agencies use the data collected from pavements in a variety of combinations and circumstances.

The use of new technology as a consequence of the need for more accurate and reliable pavement data has raised questions about how much data should be collected; that is, since computing power and memory are no longer significant constraints, other criteria must be used to determine the type and frequency of data collection. Likewise, concerns related to the historical data in the PMS databases have also raised some questions, such as what type of data are being used for which decision-making processes, what patterns can be extracted from the data, and from which data can these patterns be extracted.

RESEARCH APPROACH

As part of the knowledge discovery process, data mining is applied to extract patterns of information in PMS databases, which can be used to predict the present serviceability ratings (PSR), as an indirect way of obtaining the remaining life of pavements. The process consists of building the data mining database by exploring the data and preparing it for the given applications, evaluating the different types of data mining applications, building the model, and evaluating the model. Figure 1 illustrates the knowledge discovery process as described in the paper.

Building Data Mining Database

Building the data mining database, together with exploring and preparing the data to be mined, takes anywhere from 50 percent to 90 percent of the time and effort in the entire knowledge discovery process. The process of building the database is usually divided in eight tasks (*7*):

1. *Data collection* – identifies the sources of the data to be mined. The data needed may not be available, in which case the process for collecting it must be arranged. Data collection usually includes items such as the source of data, owner, cost (if purchased), size of tables (i.e. records/attributes), size of file(s), physical storage, security requirements, restrictions on use, and privacy requirements.

2. *Data description* – describes the content of each file. In the description of data, a list of items, for example, number of records/attributes, number or percentage of records with missing values, and field names, is usually presented. For each field name, a list of descriptions is usually provided as well. Some of the items in the list may be data type, definition, description, source of field, unit of measure, range of values, list of values, and time frame.

3. *Selection of data* – selects subsets of data to be mined. It is the "gross elimination" of irrelevant or unnecessary data, which is a very important task when limiting factors like space and time are a concern.

4. *Data quality assessment and data cleansing* – identifies characteristics of the data that will affect the quality of the model. Missing values or incorrect values account for many of the problems encountered while performing a data mining application, and these need to be fixed or accounted for in order to provide reliable results.

5. *Consolidation and integration* – combines data from different sources into a single mining database and requires the differences in data values to be reconciled. For example, U.S. dollars cannot be combined with Canadian dollars without a conversion, because there would be inconsistencies and the knowledge discovery process would be of a poor quality.

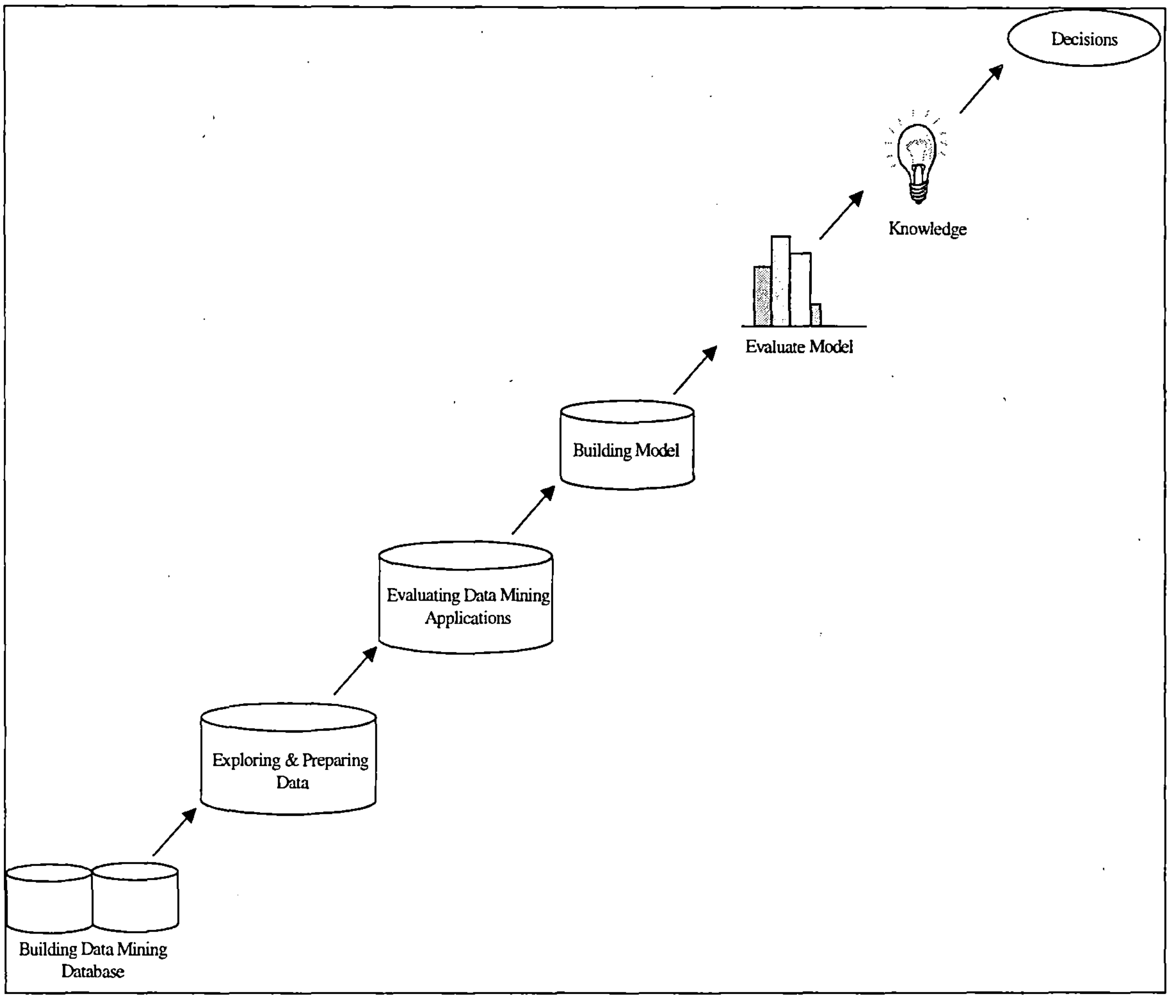

Figure 1. Knowledge Discovery Process

6. *Metadata construction* – provides information that will be used in the creation of the physical database as well as information that will be used by analysts in understanding the data and building the model. Kimball refers to metadata as having a "back-room that guides the extraction, cleaning, and loading processes, and a front-room that makes query tools and report writers function smoothly" (*19*).

7. *Load the data mining database* – stores the data in databases. Depending on the quantity and complexity of the data, it may require storage in a database management system (DBMS) as opposed to a flat file.

8. *Maintain the data mining database* – preserves/cares for the data, for example, making backups of parts of the process already completed and reorganizing the data to reclaim disk storage or to improve performance.

Exploration of the Data

Exploring the data to be mined is an extremely important part of the process of building the data mining database. It consists of identifying the most important fields/attributes in predicting an outcome and determining which derived values may be useful. Since the data sets may contain hundreds of columns (fields), exploring the data can be as time consuming and labor intensive as it is informative. It is extremely important to have a good interface and fast computer response because the nature of the exploration will depend on the time it takes for results to be obtained. Depending on the system, this task can take anywhere from seconds to minutes or even hours.

Data exploration is basically used to discover and evaluate appropriate problems in the data, define solutions and implement strategies, and produce measurable results. It involves a series of stages, including exploring the problem space, exploring the solution space, specifying the implementation method, and mining the data. Each of the stages demands a certain amount of time, and usually the importance of these stages to success is inversely proportional to the time consumption during the exploration (*7*). That is, by just exploring the data set one can determine if the data are "noisy"; this process might take only minutes, but the advantages of knowing the data quality will be enormous.

Preparation of the Data

The process of preparing the data is frequently confused with the process of exploring the data. Even though one process is needed to obtain the other (that is, it is hard to define how to extract value from the data mining activities that follow without identifying the problem to solve), these are two very different processes (*20*). The preparation of data involves the selection of variables and rows, the construction of new variables, and the transformation of variables with the purpose of manipulating and transforming raw data into data that are more easily accessible. Variables and rows are selected carefully to minimize the time it takes to build a model and to optimize the output obtained from the model. The construction of variables is useful since some variables have little effect when used alone and might need to be combined with other variables. One way of constructing variables is by using algebraic operations, like addition, subtraction, and ratios. Likewise, the transformation of variables is useful to represent the data according to the tool one chooses; for example, variables may be scaled to fall within limited ranges, such as 0 to 10. Another example of variable transformation would be the variable classification one gives to certain fields; for example, the records in a field of *last year worked* might need to be classified as categorical data in order for the application to understand the value of the record as a year.

Building the Model

Model building is an iterative process, in which alternative models are explored to find the most useful model for solving the problem in question. Models replicate some useful features of the original object in some "more-convenient-to-manipulate" way. Furthermore, models can be computationally manipulated to answer questions posed about the model's behavior, and thus, about the behavior of the real world (*20*). By manipulating the model, engineers, scientists, and economists can determine how well the proposed explanation "works".

The first part of building the model is deciding what type of data mining task is needed, that is, descriptive or predictive. The next step is to choose a model type for making the corresponding task. There are several model types, such as decision trees, neural networks, and rule induction, and the model type chosen will influence the kind of data preparation that must be conducted.

The two requirements for building a predictive model are training and testing the data mining model. Training and testing the data is usually referred to as supervised learning. It involves training or estimating the model on a portion of data, and then testing and validating it on the remainder of the data. The resulting accuracy rate of the test database is used to estimate how the model will perform on future databases that are similar to the training and test databases. It does not guarantee that the model is correct, but rather that if the same techniques were used on a series of databases with similar training and test data, the average accuracy would be close to the one obtained. Training stops when the accuracy rates on the test database no longer improve with additional iterations.

Evaluation of the Model

Once the model is built and validated, it can be used by analysts to recommend actions based on its results or to apply the model to different data sets. Remembering that the accuracy rate applies only to the data on which the model was built, it is important to learn more about the type of errors found and the costs associated with them. By evaluating and interpreting the model one can learn about the types of error as well as the accuracy rate of the model.

CASE STUDY

The methods described are being applied to a large pavement condition data provided by the Missouri Department of Transportation (MoDOT). The objective is to predict the PSR value as an indirect approach to obtaining the remaining life of a pavement. For PSR values, a pavement with a threshold of 24 is considered for replacement (21). Pavement condition data from 1995 to 1999 from MoDOT was provided for the study. MoDOT gathered a set of data from two different databases, data collected with the ARAN® van and structural data from a separate database, to present a better set of data given the objectives of the study. The data were provided in a database format consisting of 28,231 records and 49 fields.

Modeling

The first step in building the model was to convert the database file to an Excel® file in order to facilitate the preparation and exploration process of the data. Table 1 shows the field names, definitions, and types of data present in each field. The data types are related to the appropriate scales of measurement, for example, *gender* would be the measurement and *categorical* would be the scale type for that measurement. During the exploration of the data set, several problems were identified. It had three extremely noisy columns, NHS, LSHLDTYPE, and RSHLDTYPE, and missing data for every other year from the RIDE, CRACKING, RUTTING, PSR, and CONDITION fields. The three noisy columns were not considered relevant for the objective of the study, and therefore they were not selected for the model. A first attempt at restoring the missing data was made by using the values from previous years in the place of missing values. A second attempt will be made by using the average of previous and subsequent years of the same fields. In addition, 54 percent of the whole data was found to be a better representation for the model. That is, from the five-year condition data reported, only the 1999 PSR data was fully complete and the second most complete set of data was the 1998 PSR data. Therefore, the selection of records was based on the amount of complete data for 1998. The advantage of making this selection was to obtain a higher percentage of good data to build the model.

Table 1. Pavement Condition Data from 1995 to 1999

Field	Meaning	Type
DIST	District	Numeric
CNTY_NO	County number	Numeric
RTE_DESIG	Route designation	Categorical
ROUTE	Route number	Categorical
DIR	Direction	Categorical
CONT_BLOG	Mileage from the beginning of county	Numeric
CONT_ELOG	Mileage from the end of county	Numeric
AREA	Area (i.e., urban/rural)	Categorical
ST_SYS	Street system	Categorical
FC	Functional classification	Categorical
LANES	Number of lanes	Categorical
TRAV_WAY	Width of travel way	Numeric
ORIG_SURF	Original surface	Categorical
OVERLAYS	Quantity of overlays	Numeric
SURF_TYPE	Surface type	Categorical
YR_LSTWK	Year last worked	Categorical
AADT	Average annual daily traffic	Numeric
RIDE (95 to 99)	Ride quality	Numeric
CRACKING (95 to 99)	Cracking	Numeric
PSR (95 to 99)	Present serviceability rating	Numeric
RUT (95 to 99)	Rutting	Numeric
CONDITION (95 to 99)	Condition	Numeric
DESGN_CLS	Design classification	Categorical
DIVCLS	Division classification (i.e., divided/undivided)	Categorical
LSHLDWDTH	Left shoulder width	Numeric
RSHLDWDTH	Right shoulder width	Numeric
NHS	National highway system	No Data
LSHLDTYPE	Left shoulder type	Categorical
RSHLDTYPE	Right shoulder type	Categorical

As part of the data preparation process, the software tools to be used were selected. The IBM Intelligent Miner for Data (Intelligent Miner) was selected according to its connectivity features and data mining characteristics. The features of the IBM Intelligent Miner for Data provide the use of large sets of relational data to be mined. The data sources that the software accepts are ASCII text, DBase, Oracle, and Sybase. The software is able to perform several data mining applications, such as prediction, data preprocessing, regression, classification, clustering, and associations. In addition, it uses decision trees and neural networks for its discovery methodology (*22*).

Based on the features provided by Intelligent Miner, the data for the model were converted to a text file. The text file was then corrected so that it would not include any tabs; this was an extensive part of the process given the number of records and the irregularities of each field in the data. Once the tabs were eliminated, the data set was ready for the applications of the Intelligent Miner.

The three data mining applications considered for the research are association, neural clustering, and tree classification. The basic understanding of the data mining applications used in the study is that they should be able to produce a description summarizing the characteristics of pavements, that have a PSR greater than 24, for example; and that they should be able to compare two groups of pavements, such as those which have PSR greater than 24 and those which have PSR less than 24 (23).

Given that associations are used to show attribute-value conditions that occur frequently together in a given set of data, this method will be used to find the PSR value of a pavement given certain characteristics of a pavement, such as YR_LSTWK, CONDITION, RUTTING, RIDE quality, number of OVERLAYS, AADT, SURF_TYPE, etc. Neural clustering applications are used to find centers for each cluster once the records are grouped together due to similar characteristics. This type of data mining application will be used to find the PSR of a pavement given the particular characteristics of such pavement and the similarities it has with a given cluster, i.e. a new pavement can be distributed to the cluster whose center is the most similar.

Since tree classification applications create a model based on known data, this method will be used to classify pavements as belonging to a "vulnerable" group (i.e. PSR > 24) or not belonging to a "vulnerable" group (i.e. PSR < 24). The process for the tree classification is divided into three parts, training mode, testing mode, and application mode. In training mode, a mining run learns the fields of each of the defined pavement "vulnerable" classes. Testing mode will then be used to test the accuracy of the model created in the training mode by applying this model to test data with known "vulnerable" pavement classes, and application mode will then be used to predict which pavement will obtain a high PSR value in the future.

Preliminary Results

Due to the extensive process of data preparation, the results obtained in this paper are from 32 percent of the portion of data selected for the completion of the research from the condition data collected in 1999. Even though the results obtained here are from a small segment of the entire data set provided by MoDOT, it is still a good representation of the condition of pavements for the State of Missouri since it consists of 4816 records.

The results indicate that 89 percent of the pavements were in "vulnerable" conditions in 1999; that is, having PSR values greater than or equal to 24. The tree classification method was used to study the data. PSR was used as the class label, and RUT, RIDE, COND, CRCK, YR_LSTWK, AADT, OVERLAYS, ORIG_SURF, and SURF_TYPE were used as active fields. Table 2 shows the preliminary results obtain from the condition data collected by MoDOT in 1999.

The rest of the parameters, YR_LSTWK, AADT, OVERLAYS, ORIG_SURF, and SURF_TYPE, need further study to obtain better information.

The results shown here are preliminary results that are intended to demonstrate the type of information expected, further research is being conducted in which results from the whole set of data selected for building the model will be presented and interpreted in order to obtain a better view of the condition of pavements in Missouri.

Table 2. Preliminary Results from the Condition Data Collected in 1999

		PSR > 24	PSR < 24
		Range of Values	
Rut	**Min.**	0.255 – 0.295	0.195 – 0.285
	Max.	0.295 – 0.345	0.325 – 0.335
Ride	**Min.**	4.435*	2.295*
	Max.	6.15 – 7.49	4.435*
Condition	**Min.**	18.45*	18.45*
	Max.	**	**
Cracking	**Min.**	3.445 – 3.805	3.285*
	Max.	3.805 – 4.050	3.75 – 4.050

* Constant values
** No data was found

The methods for restoring the missing values in the data are expected to cause some problems. It is likely that the replacement of missing values with values from previous years will not show the corresponding behavior of the deterioration of pavements, but it is a good way of avoiding inconsistencies in the results obtained from performing the data mining applications. Likewise, the replacement of missing values with the average of values obtain from previous and subsequent years will probably not show the corresponding behavior either, but again it is a way of avoiding inconsistencies in the results obtain. The results obtained from these two approaches will be compared to the results obtained from leaving the data just as it is in order to learn from the percentage of errors found as well as to point out the need for the collection of pavement data.

CONCLUSIONS

The collection of pavement data is an extensive process that is done both manually and automatically. The data collected are currently being used by state agencies to assess the decision-making process for the maintenance and rehabilitation (M&R) of our nations highways. However, the amount of data stored in the PMS databases today is immense, which generates the need for knowledge in order to use more of the data for any M&R decisions.

The KDD process is being applied to a set of data provided by MoDOT as an initial step towards the goal of expanding the use of pavement management data. The process here described is part of an ongoing study. Among the parameters used for the assessment of the pavements by means of the KDD process and data mining applications, the most relevant were the present serviceability rating, rutting, ride quality, condition, cracking, year the pavement was last worked, AADT, overlays, original surface, and surface type. However, for future work, parameters like the percentage of trucks and passenger vehicles, design life of each pavement, type of pavement (i.e. concrete/asphalt, and if concrete, reinforced/unreinforced or continuously reinforced), and environment, should also be considered in order to extract better information, thus make better decisions.

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AIRBAG: IS IT AN EFFECTIVE OCCUPANT PROTECTION SYSTEM?

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ABSTRACT

The role of airbag as an occupant protection system is very significant. In case of serious frontal crashes, airbags can perform well enough to provide additional protection. Airbags save and/or kill people involved in a crash depending on the way they deploy and the position of the passenger when the crash occurred. Hence, understanding the process of airbag deployment is very crucial. Some significant facts related to air bag deployment are discussed in this paper. Many factors influence the effectiveness of airbags. Hence, the effectiveness of airbags is discussed at length and the variation in the effectiveness due to different factors and in different applicable conditions is presented. One of the more recent problems being widely discussed is the child-airbag interaction. A few facts regarding this issue are outlined and the possibilities for a positive interaction between children and airbags are discussed. The concept of future airbags (smart airbags and side airbags) has been introduced. It has been concluded that airbags are an efficient occupant protection system, if used with necessary precautions.

INTRODUCTION

There have been 3.8 million airbag deployments from the late 1980s to October 1999, with driver side deployments accounting to 3.3 million and passenger side deployments constituting 0.5 million. As of October 01, 1999, 89 million vehicles are equipped with airbags, out of which 57 million are cars (about 45 percent of cars on the road) and 32 million light trucks (about 41 percent of light trucks) (*1*).

An Overview

Airbags have reduced driver deaths by 14 percent and passenger deaths by 11 percent. The most widely accepted method of statistical analysis, called the double pair comparison studies are employed, for calculation of lives saved. The process involves a mathematical analysis of the real-world fatality experience of vehicles with airbags compared with vehicles without airbags (*1*).

Statistics of airbags

As of March 01, 2000, the estimated gross number of lives saved by airbags is in the range of 4496-5303 drivers and 807 right-front passengers (2). The estimate of airbag benefits from late 1980s through March 2000 is shown in Figure 1.

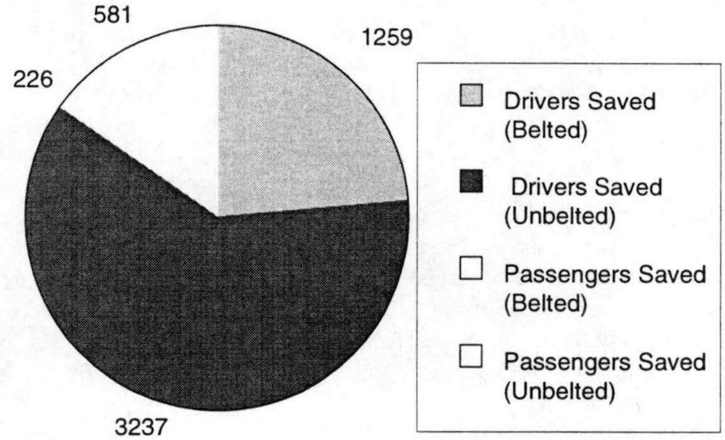

FIGURE 1 Estimated airbag benefits (*1*)

Although relatively rare, inflating airbags have also caused deaths and serious injuries. In the last decade, the total number of fatals involving airbags is 194 and the total number of seriously injured drivers, passengers, and children are 128 (2). The estimated airbag deaths have been shown in Figure 2.

Purpose

The focus of this paper is to emphasize the effectiveness of airbags. The child-airbag interaction has been an issue of concern in the last decade and hence an emphasis has been placed on this issue. Airbag on/off switches and their installation has also been a controversial issue recently. A discussion on smarter airbags and side airbags has been included. The overall purpose is to reiterate the fact that airbags are an efficient occupant protection system if used properly with due considerations to the risk involved.

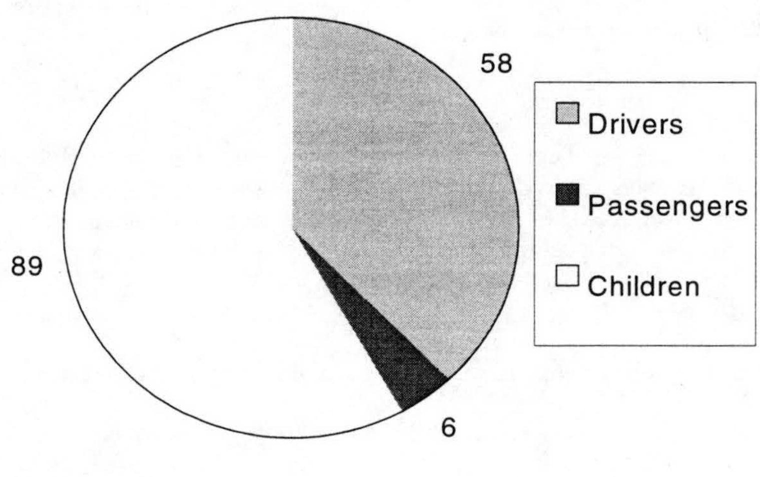

FIGURE 2 Estimated airbag deaths (*2*)

An airbag is not a soft, billowy pillow. An airbag inflates, immediately after a serious crash, in a fraction of a second and becomes an energy-absorbing buffer between people in the vehicle and the hard interior surfaces of vehicles (*3*).

To do its important job of protection, an airbag comes out of the dashboard at up to 200 miles per hour faster than the blink of an eye (4). Air bags are mounted in the steering wheel and the right front instrument panel and are designed to protect people in serious frontal crashes. The air bags help people by preventing their heads and chests from hitting the steering wheel, instrument panel, or windshield. In case of frontal crashes, even the occupants wearing seat belts move forward towards the steering wheel, instrument panel, or windshield. Airbags provide additional protection to occupants wearing seat belts by preventing them from hitting the hard interior surfaces. The design of most of the airbags is such that they inflate in crashes equivalent to hitting a solid barrier at 10-12 mph (*3*).

Present Scenario of Airbags

According to a study of real-world crashes conducted by the National Highway Safety Administration (NHTSA), the combination of seat belts and airbags is 75 percent effective in preventing serious head injuries and 66 percent effective in preventing serious chest injuries. As mentioned earlier the airbag provides a cushion and keeps the occupant's head, neck and chest from hitting the hard interiors of a vehicle. In order to perform well, an airbag must deploy quickly and forcefully. After the airbag bursts through its cover and begins to inflate, the force is the greatest in the first 2-3 inches. Those first 2-3 inches are the 'risk zone'. The force of the airbag decreases as the bag inflates farther (*5*).

Positioning too close to the airbag or failure to use proper restraints may result in serious injury or death. The risk is the same for both children and adults (5). In the first few milliseconds of inflation the force is very high and can seriously injure anyone struck by the inflating airbag. In most of the deaths caused by airbags, people involved were unbelted or improperly belted which allowed them to move on top of, or extremely close to the inflating airbags. Most of the adults (drivers and right-front passengers) killed by airbags were not properly restrained. Most of the deaths related to inflating airbags have been children. All older children killed by airbags were either unbelted or improperly belted and most of the infants were in rear-facing restraints in the front-seat (*3*).

A driver of any size and age needs to buckle up and sit at least 10 inches away from the steering wheel to avoid serious injury risk from an inflating airbag. On the passenger side, there's no significant airbag injury risk for belted adults sitting back in the seat (*6*). The back is the safest for children of any age. A recent study by IIHS found that children in back are 36 percent less likely to be killed in crashes (*3*).

FACTORS INFLUENCING THE EFFECTIVENESS OF AIRBAGS

The effectiveness of airbags is influenced by type of crash, type of occupant protection system employed along with airbag. The effectiveness of airbags is different when used alone and when used in conjunction with an active occupant protection. Also, the characteristics of drivers and front-seated passengers such as age, and height have a significant impact on the effectiveness of airbags.

Type of Crash

Airbags significantly reduce the likelihood of fatal injury in the most severe type of crashes, both purely frontal and head-on collisions (*7*). Airbags are so designed that they do not deploy in side impact crashes and rollover crashes. The airbags have little effect in case of non-frontal or side impact crashes (5). The description of a frontal and a non-frontal crash is explained with respect to Figure 3.

Head on Collision

A crash is considered head-on or purely frontal if the point of impact is 12:00 (Figure 3) (*7*).

Side Impact

If the initial point of impact is in between 10:00 and 2:00 (Figure 3), the crash is considered a non-frontal crash (*7*).

Airbag Plus Active Occupant Protection System

The most common active occupant protection system in use is the seat belt.

Seat Belts

When occupants are belted, the airbag offers increased additional protection to occupants from hitting the hard interiors of the vehicle in case of a frontal crash. Airbags in conjunction with lap-shoulder belts are proven lifesavers for adults in case frontal crashes (*7*).

Driver

The factors to be considered are age and height (*7*).

Age

Younger drivers (age14-50) seem to experience more benefits from airbags than older drivers (50 or older) (*7*).

Height

Shorter drivers (especially short women) have occasionally exhibited problems in maintaining the safe distance of 10 inches between the breastbone and the cover of the airbag (7).

Front-Seated Passenger

The most significant factor is the age of the passenger in front (*7*).

Age

Children (age 0-12) are proven to be at a great risk of injury or death from airbags if seated in the front-seat with a passenger airbag (*7*).

Conclusions

Based on real-world crash experience, it is now clear that the frequency of fatal airbag induced injuries is not inconsequential. A review of the validity of early life saving forecasts for airbags states that

- In the case of airbag effectiveness the early analyses did not include the truth within the uncertainty intervals. It should have been realized that technologies often perform differently in controlled settings than in uncontrolled ones.

- In case of early estimates of airbag effectiveness, no attempt was made to quantify the number of occupants who might be killed by adding deployment energy to crashes.

- In case of airbags, it is now obvious that welfare of children and elderly were unintentionally overshadowed by preoccupation with the protection of the presumed sensitive group (unbelted adult males).

- Based on real-world injuries caused by airbags, NHTSA now recognizes that it is not sufficient to require airbags and expect that this "passive" device will work optimally. It is recognized that the so-called passive safety devices may have diminished effectiveness and unexpected risks, when human behavioral complications are ignored.

- As technology changes, estimates of risk and benefit should be subjected to refinement and validation over time (8).

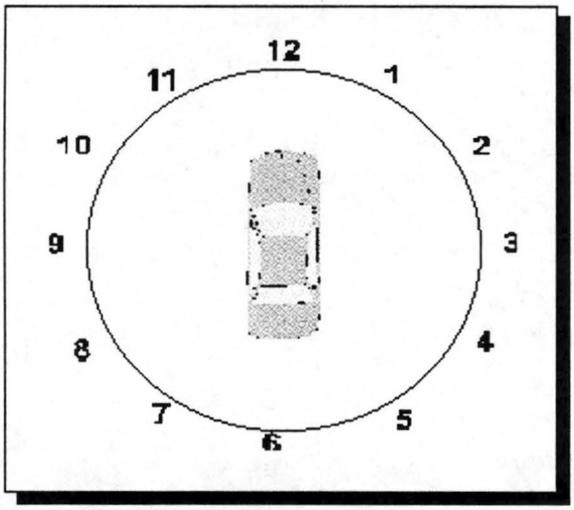

FIGURE 3 Description of point of impact (7)

CHILD-AIRBAG INTERACTION

Most deaths from inflating airbags have been children (*3*). The unintended adverse effects of an inflating airbag pose the greatest risk to infants in rear-facing restraints and unbelted/unrestrained children in the front seats of vehicles with passenger airbags (*9*).

The Problem

Most deaths from inflating airbags have been children (*3*). The unintended adverse effects of an inflating airbag pose the greatest risk to infants in rear-facing restraints and unbelted or unrestrained children in the front seats of vehicles with passenger airbags (*9*). When an infant in a rear-facing restraint is placed in the front seat with a passenger airbag, the infant's head is too close to the bag. The enormous speed, with which the airbag inflates, exerts force against the back of the restraint and can cause serious, even fatal, head injuries to the infant (*3*).

Most of the older children killed by airbags are either unbelted or improperly belted. Even though belted, when a child leans forward, for example, to fiddle with the radio, the child's head can be too close to the airbag. Hence even belted children can be at risk in the front seat with a passenger airbag (*3*).

Figure 11 illustrates the risk to which the infants in rear restraints are subjected to by an inflating airbag, when placed in a front seat with passenger airbag.

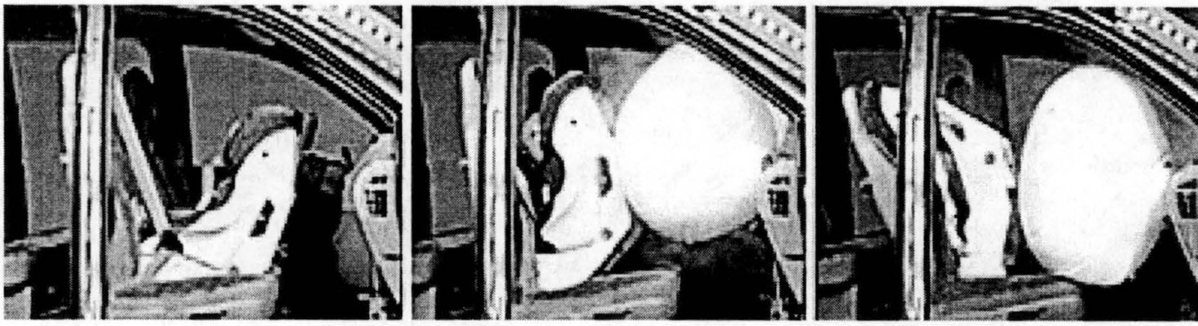

FIGURE 11 An illustration of hazard to a child in rear facing restraint seat placed in the front seat with a passenger airbag (9)

Possible Solutions

Steps that can avoid the risk of death/injury from an airbag

- Infants in rear facing restraints should never ride in the front seat of a vehicle with a passenger side airbag.
- Small children should ride in a rear seat in child safety seats approved for their age and size.
- Children 12 and under should ride buckled up in a rear seat.
- If a child over one year and under 13 should ride in the front seat with a passenger side airbag, the child must be placed in a front facing safety seat, a booster seat, or a correct fitting lap/shoulder belt and the seat be moved as far back as possible (*4*).

The back seat has always been the safest for children even before airbags and now this is even more important. A recent IIHS study found that children riding in back are 36 percent less likely to be killed. Only when transporting too many small children and not possible to place all of them in the back, should a child ride upfront. Then it should be made sure that the seat is all the way back and the child is securely buckled up in a lap/shoulder and sitting all the way back in the seat (*3*).

FUTURE AIRBAGS

Smart airbags and side airbags are the significant developments in airbag technology in recent years. The smart airbags are designed keeping in mind the problem posed by present airbags to children and also to short adults. The side airbags are intended to improve the effectiveness of airbags in side impact crashes.

Smart Airbags

It was predicted that the smart airbag systems would arrive in phases in the mid-'98 model year. Phase one was likely to include a weight sensor, variable crash severity sensor, seat buckle sensor, dual/variable output inflator and rear-facing child detection. Phase two was likely to add complete occupant sensing to determine exact position, weight, and presence of a child seat (*10*).

The Need

Recently released statistics from the NHTSA predict that currently designed passenger side airbags in the U.S. will kill one child a week. The immediate solution is wearing a seat belt and staying as far away as possible from the airbag. The short-term solutions studied by both the government safety groups and automakers include on/off switches and high/low power inflators. But the long-term solution for which both the government and automakers agree is "smart" airbags (*10*).

The Technology

A design change that will lower injury risk would be to reduce the airbag inflation energy. This won't eliminate the risks, particularly for children (3). These are called "depowered" airbags as they deploy with less force than the current airbags (5). By definition, a true, smart airbag is one that can detect the size and position of the occupant it is designed to protect. It can also detect the severity of the crash that deploys it. Using that information, the airbag provides maximum protection by adapting its inflation rate according to the prevalent conditions. It will also be smart enough not to deploy at all if it detects the presence of a child (10).

Considerations

According to Bill Eagelson, manager of occupant protection and impact dynamics at Ford's Environmental and Safety engineering, "There is a big difference between something being concept ready and its actual implementation and adoption into mass production". Both automakers and regulators should realize the time constraint. For any hope of getting a safer airbag into cars, the decision must be made now. If the automakers don't have time to perform tests, and suppliers don't have time to tool-up for production, the 1998 models may fulfill NHTSA's grim prophecy of losing a child each week to airbag deployment. Regardless of the good intentions of the regulators, a restraint should be exercised to avoid pushing a technology into production that promises something it isn't ready to deliver (10).

Side Airbags

Significant supplemental safety benefits can be provided to adults, by side airbags, in side impact crashes (11). These are designed to produce energy-absorbing buffers between people and the vehicle doors that can be driven into them in side impact crashes. These are smaller than frontal ones. Most side airbags are designed to protect people's chests, and are likely to provide some head protection also. They can be mounted in doors, seats, and roof rails. Side airbags are expected to offer increased protection from the fronts of striking vehicles and also intruding objects such as trees, poles (3).

CONCLUSIONS AND RECOMMENDATIONS

Regardless of size or age, anyone close to or on top of an inflating airbag is at risk. Hence the position of the occupant is very important. An airbag is effective and will do its job properly only when the occupant is at least 10 inches from the airbag when it is inflating.

The following ABCs will allow airbags to perform as an effective occupant protection system:

- **A**lways slide the seat back as far as possible and sit back
- **B**uckle everyone
- **C**hildren 12 and under ride properly restrained in the back seat

Before going into mass production, the regulators and automakers should do a thorough investigation into the performance of smart airbags. A technology that isn't yet ready to deliver should not be pushed into production. The initial estimates of airbag benefits were overestimated by overconfidence and the same should not be repeated with the so-called "smart" airbags.

Airbags aren't alternatives to safety belts, but are designed to work with belts and provide additional protection in serious frontal crashes. Airbags and belts work together as a system, and one without the other isn't as effective (11).

Very few people need an on/off switch. By turning off an airbag, one is forgoing the important protection provided by the airbag in case of a serious frontal crash. After knowing the facts, it becomes clear that leaving the airbags intact is almost always the best (11).

The final conclusion of K. M. Thompson, et al. is taken as one of the conclusions of this paper. From the perspective of risk-analysis, a consideration to the variability in the population's susceptibility to risk and benefit as well as the degree of uncertainty in the estimates of risk and benefit is very critical to the engineer. This has been revealed by the airbag case study. Hence technologies that result in maximizing benefits, minimizing risks, and promoting warranted public confidence should be aimed at by reducing bias and overconfidence in estimation (8).

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PEDESTRIAN ARRIVALS AT SIGNALIZED INTERSECTIONS IN CENTRAL BUSINESS DISTRICTS

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ABSTRACT

Pedestrian flow pattern in coordinated signal networks may not be random or uniform. Statistical analysis of the data collected in downtown St.Louis along 7 routes supports the above hypothesis. A methodology was used to prove that there exist two significantly different flows within a cycle. The results from this study indicate that, the equation used to calculate the delay at signalized intersection might have to be improved. And also suggests, that consideration of pedestrian progression might be a viable alternative to improve the quality of pedestrian flow in the cities.

PROBLEM STATEMENT

Pedestrian walkways are an important mode of transportation in Central Business Districts(CBDs). Understanding the characteristics of the pedestrian movement along these walkways would be helpful in planning for better mobility of pedestrians.

Pedestrians are assumed to arrive randomly at signals (*1*). This may not be true when pedestrians are traveling in coordinated signal network. Pedestrians arriving randomly at an intersection will move in a group after the signal turns green and might continue as a significant group towards the downstream signal. The objective of this study is to test the validity of the random arrival characteristics of pedestrians in a coordinated signal network.

BACKGROUND INFORMATION

Virkler has done related research in Australia (*2*) but to author's knowledge little or no work was done in America in this area. The equations used in this study are stated as follows

$$\text{Rate of flow} = \frac{\text{Number of pedestrians in the period (high or low)}}{\text{(\# Cycles) *(Proportion of the flow (high or low))}} \quad (1)$$

$$\text{Pooled Standard Deviation}(3) = \sqrt{\frac{S_1^2 + S_2^2}{N_1 + N_2}} \quad (2)$$

S_1,S_2 are standard deviations for the high and low flow data

N_1,N_2 are the number of cycle lengths the data was collected.

Pedestrian delay for random arrivals at an intersection can be derived (*4*) as

$$d_u = \frac{r^2}{2C} = \frac{(C-g)^2}{2C} \quad (3)$$

where: d_u = averaged stopped delay per pedestrian

r = effective red time

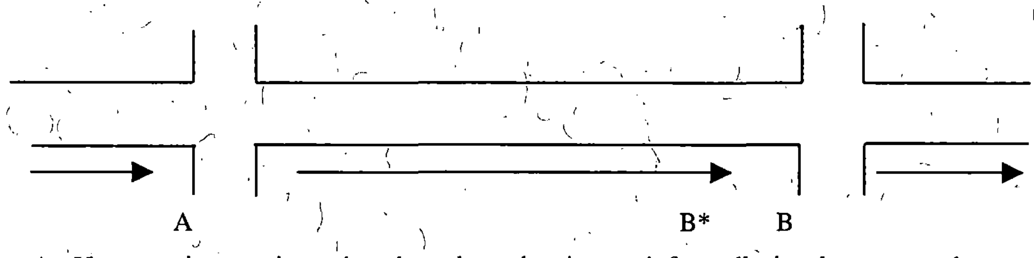
C = cycle length

g = effective green time

METHODOLOGY

The Transportation professionals in and around St. Louis were contacted for best possible walkways carrying significant amount of pedestrians. The input from them and personal judgement was used in identifying the routes for this study. The following Methodology was used to conduct the study.

Number of pedestrians arriving at a signalized intersection (B or B* in Figure1) from an upstream-signalized intersection (A in Figure1) were counted for a period equivalent to 15 minutes or approximately 15 minutes, so that the cycle length will be a factor of the time period. The data collected were then statistically analyzed.



A - Upstream intersection point where the pedestrians wait for walk signal to step on the crosswalk
B* - Possible Queuing point where pedestrians are likely to join the waiting queue
B - Downstream intersection

FIGURE 1 Pedestrian Walkway Segment.

FIELD STUDY

The data for this study was collected in St.Louis CBD. The signals in the area of study were coordinated and pre timed. Each intersection had a cycle length of 60 seconds. The walkways lengths ranged from 200ft to 300ft and the walk times ranged from 6to 25 seconds.

Data collection was performed on walkways, which had major pedestrian influx from offices and Metro Link Stations. A total of 12 data sets were collected on 7 different routes. On each route the number pedestrians arriving at the downstream intersection from the upstream intersection were counted in 5-second increments, for a period of 15minutes that was incidentally equal to 15 cycle lengths. The data were collected in the form similar to the one shown in Table 1.

Results and Discussion

Table 2 helps in illustrating the statistical analysis procedure to be used for the analysis of the data.

1. The number of pedestrians was summed in all the cycle lengths for each 5-second increment.
2. By observing the flow within the cycle, the cycle length was divided into high flow and low flow periods.
3. Flow rates in these two periods were calculated using the equation 1.

TABLE 1 Platooning Study Form

Route: Along Pine From 8th street to 7th street							Weather :Sunny					
Cycle Length:60 sec at both intersections							Date:8-18-2000					
Min\Sec	5	10	15	20	25	30	35	40	45	50	55	60
0	3	0	0	0	0	0	0	0	0	1	0	2
1	2	1	0	1	0	0	3	0	0	1	0	1
2	0	0	3	4	0	0	0	0	0	0	0	0
3	0	0	0	0	3	2	0	0	0	0	0	0
4	1	0	0	0	0	0	0	0	0	0	0	0
5	0	0	0	1	0	0	0	0	0	2	1	2
6	0	0	0	3	0	0	0	0	0	0	2	0
7	0	0	0	0	0	0	0	4	3	4	0	1
8	1	1	0	0	0	0	2	0	0	1	3	1
9	0	0	0	0	0	4	0	2	0	1	0	0
10	4	2	0	0	0	0	0	1	0	0	0	0
11	4	0	0	1	0	0	0	0	0	1	2	3
12	4	0	0	0	0	0	0	1	0	2	2	0
13	0	2	2	0	0	0	0	0	0	0	0	0
14	0	0	0	0	0	0	0	0	0	0	1	0
Sum	19	6	5	10	3	6	5	8	3	13	11	10

4. The standard deviation among each flow period data was calculated.
5. The two standard deviations should be pooled using the equation 2.
6. Significant differences in flow rates were tested using the T-statistic.

Analyzing the twelve data sets using the above procedure indicated that high and low flow rates existed within a cycle length at a significant value less than 0.005. This supports the hypothesis "arrivals in coordinated signal networks are not random".

TABLE 2 Results Table

Direction	Total # Of Peds	High Flow Flow rate Peds/min	High Flow Std dev	# Of Peds	Low Flow Flow rate	Low Flow Std dev	Pooled Std	Calculated T-stat	Pvalue
P: 8-->7	103	9.4	2.2	56	5.6	3.7	1.11	3	<. 005
P: 8-->7	99	10.6	4.0	46	4.6	2.0	1.15	5	<. 001
P: 7-->8	73	8.6	3.9	30	3	4.8	1.60	4	<. 001
P: 8-->9	103	10	3.7	53	5.3	3.1	1.25	4	<. 001
O: 7-->8	46	5.8	5.3	17	1.7	1.4	1.42	3	<. 005

CONCLUSIONS AND RECOMMENDATIONS

The knowledge of pedestrian arrivals would be very useful in designing a better-coordinated signal system. This study indicates that the delay equation used to calculate the pedestrian delay at signalized intersection might not be correct, as the arrivals are not random. Further research should be done to suggest appropriate equation to estimate the delay. Considering pedestrian progression in the CBD's might be useful in understanding the characteristics of pedestrian flow, as it will enhance the quality of flow.

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MAP INTERFACE FOR IOWA DEPARTMENT OF TRANSPORTATION'S ACCESS-ALAS

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ABSTRACT

This paper discusses the limitations of Iowa Department of Transportation's (DOT) Access-Accident Location and Analysis System (ALAS) and also explains the working of the map interface, a software program designed to overcome these limitations. The paper also includes discussion on other crash analysis systems designed and/or deployed in other states. All features/capabilities of the map interface are explained and some of them are compared to those of Access-ALAS to focus on the enhanced performance of map interface.

INTRODUCTION

State departments of transportation, insurance agencies and transportation consultants do crash analyses to evaluate roadway safety. Various software programs are used by these agencies to perform crash analyses. The Iowa Department of Transportation (DOT) uses Geographic Information Systems (GIS)- Accident Location and Analysis System (ALAS) and Access-ALAS to perform system-wide crash analyses. ALAS is used to retrieve accident data and generate customized reports that are used to improve engineering (highway design), safety and enforcement in transportation. This paper focuses on enhancing the capabilities of Access-ALAS and improving access to the data for crash analysis by designing a map interface. The map interface will ease the process of selecting crashes by providing a user-friendly Graphical User Interface (GUI). A GUI helps the safety analyst perform crash analysis faster and thereby the analyst can spend more time in giving better solutions to transportation problems and making the transportation system safer. Therefore, enhancing Access-ALAS with a map interface will have an indirect impact on transportation safety.

The main purpose of a crash analysis is to improve safety by identifying crash patterns and mitigating the crash severity or/and reducing the number of crashes by adopting suitable countermeasures. The effectiveness of these countermeasures in improving safety depends upon the accuracy of the crash analysis. The accuracy of the results of crash analysis depends upon the nature (spatial or textual) and accuracy of crash data input into the crash analysis system. Therefore, crash data have a considerable effect on roadway safety. Figure 1 shows the relationship between crash data and roadway safety.

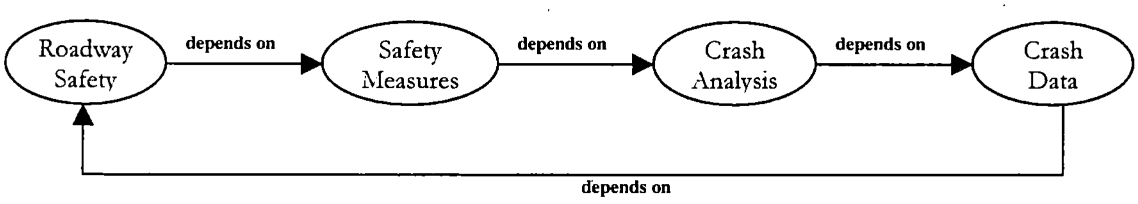

FIGURE 1 Graphical Representation of the Relationship between Roadway Safety and Crash Data

Crash data can be in spatial or textual form. If spatial data are used, the analyst will have a choice to select specific crashes and decide whether the crash is intersection-related or access-related or curve-related and so on. On the other hand if textual data are used, the safety analyst has to use some criteria such as an intersection ID to query crash data and the results will be based upon the data entered into the crash table. Therefore, spatial data allows the analyst greater discretion during the crash selection process whereas textual data limits analyst's choice to discrete ranges of attribute values.

OTHER CRASH ANALYSIS SYSTEMS

Analogous to the Iowa Department of Transportation's Access-ALAS, Washington State DOT has developed a Collision Records System (CRS) that helps the local agencies improve traffic safety on their street systems. CRS stores the collision data in a Microsoft Access database. Microsoft Access helps in generating specialized reports such as location report, high collision location report and corridor report (*1*).

The Kentucky State police uses CRASH, the primary goal of which was to design and implement a single, uniform, cost-effective system to accurately capture, analyze, and report traffic collision data. Kentucky State Police, federal and local law enforcement agencies throughout the Commonwealth, use the CRASH System (*2*).

The state of Alabama stores and manages detailed information about vehicle crashes including the estimated location where a crash occurred. Crash locations are estimated and hand recorded by highway patrol officers at the scene of a crash. These locations are typically placed at identifiable points along the roadway, such as mileposts and intersections (*3*).

Iowa DOT's TraCS software can provide crash data to the safety analysts as quickly as eight hours from the time of crash occurrence (*4*). This helps in getting the most recent crash data. Automated collision database and reporting system developed for Nashville, Tennessee will make possible the near real-time entry and analysis of collision records. This system will allow for enhanced manipulation of collision data, thereby increasing time efficiency and improving the overall level of analytical capability (*5*).

PROBLEM DEFINITION

Access-ALAS uses node numbers to run crash-location queries. Nodes include intersections and points on roadways with significant change in alignment. A unique number is assigned to each one of these nodes. When new roads are added to the existing road network, new node numbers representing the new intersections are added to the system, thereby making the node-numbering system non-uniform.

The Access-ALAS user requires node maps to perform crash analysis. The user has to manually read these node maps to determine the node numbers for the intersections, which can make the crash data retrieval process time-consuming.

As most of the Access-ALAS queries are based on nodes, and thereby node numbers, the analyst does not have a chance to select crashes between any two points on the roadway except between those points with designated node numbers.

APPLICATION OF MAP INTERFACE

The queries in the map interface are based on real world x-y coordinates unlike the existing Access-ALAS system and therefore, unlike Access-ALAS, the map interface does not inherit the limitations of a node-based system. The safety analyst can select crashes based on different criteria using the map interface software program. The most commonly used criteria featured in the map interface are as follows.

1. County-wide crashes
2. City-wide crashes
3. Crashes in a user-selected area (a rectangular area)
4. Crashes along a road between any two points
5. Crashes between intersections with intersection names as the input
6. Any selected crash

In the existing system, the user has to enter the node numbers to analyze crashes on either a particular roadway or between intersections. Figure 2 shows a node-input screen to analyze crashes between two pre-defined nodes. The node number 1 is for the intersection of Hyland Avenue and Lincoln Way, and node number 2 is for the intersection of Hayward Avenue and Lincoln Way. These node number values are obtained from the node maps provided by Iowa DOT.

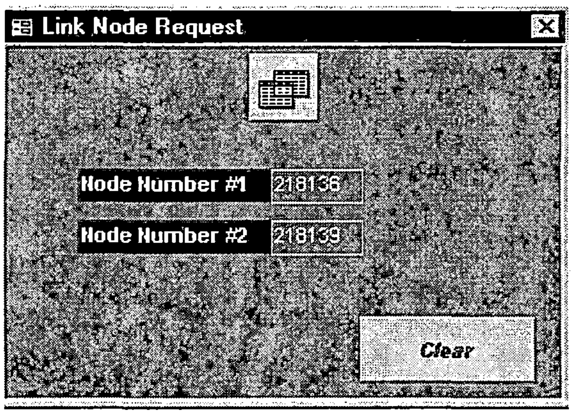

FIGURE 2 A Node-Entry Screen in the Existing Version of Access-ALAS

Determining node numbers for all the intersections required to be queried is a time-consuming process. The map interface allows the user to select crashes from a map that consists of roads with road names, crashes, county boundaries, county names and other features such as rivers and railroads. With the map interface, the user can perform the same query mentioned above with just two mouse clicks. This not only saves the analyst's time but also gives him a chance to spend time in doing more analysis. Figure 3 shows a map interface screen with crashes between the intersection of Hyland Avenue and Lincoln Way, and intersection of Hayward Avenue and Lincoln Way selected.

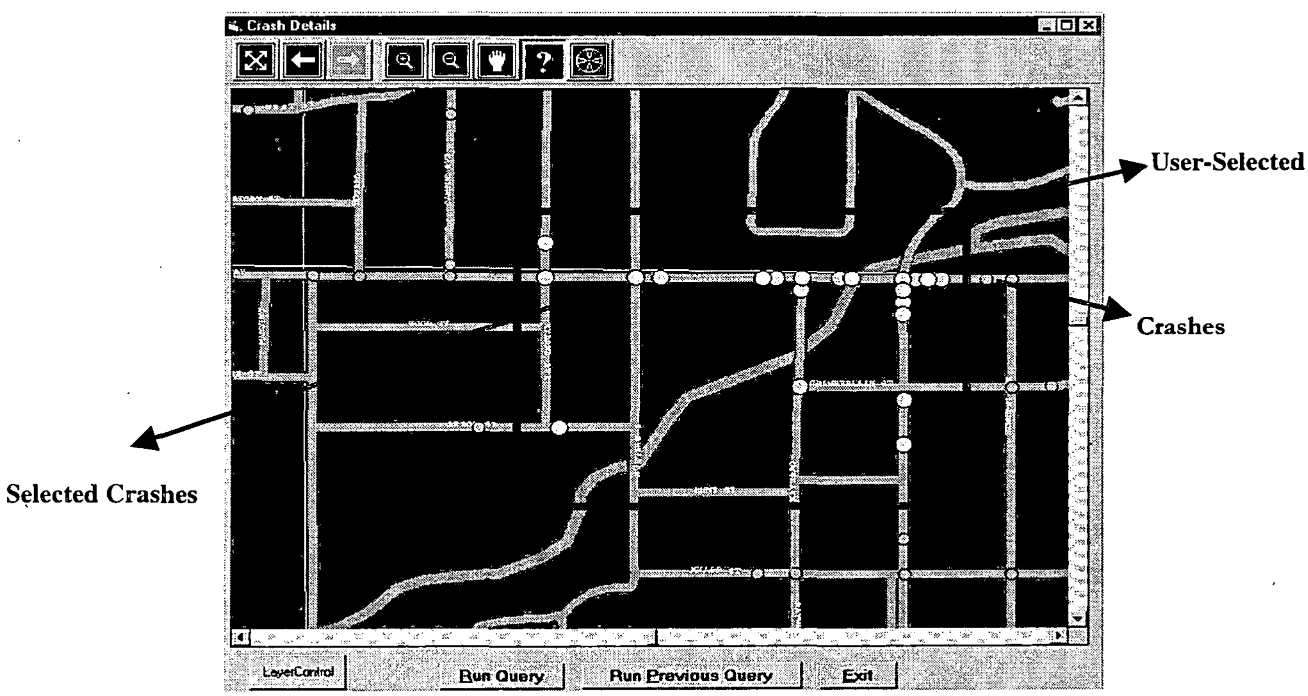

FIGURE 3 Map Interface Crash Selection Screen

One of the other features of map interface lets the user select crashes in a particular area by drawing a rectangle around the crashes. Figure 4 illustrates the area-wide selection of crashes.

Care is taken to protect the functionality of Access-ALAS, thereby providing the user a choice to select either the traditional node-based system or the map interface. In future, the crashes will not be referenced using node numbers any more.

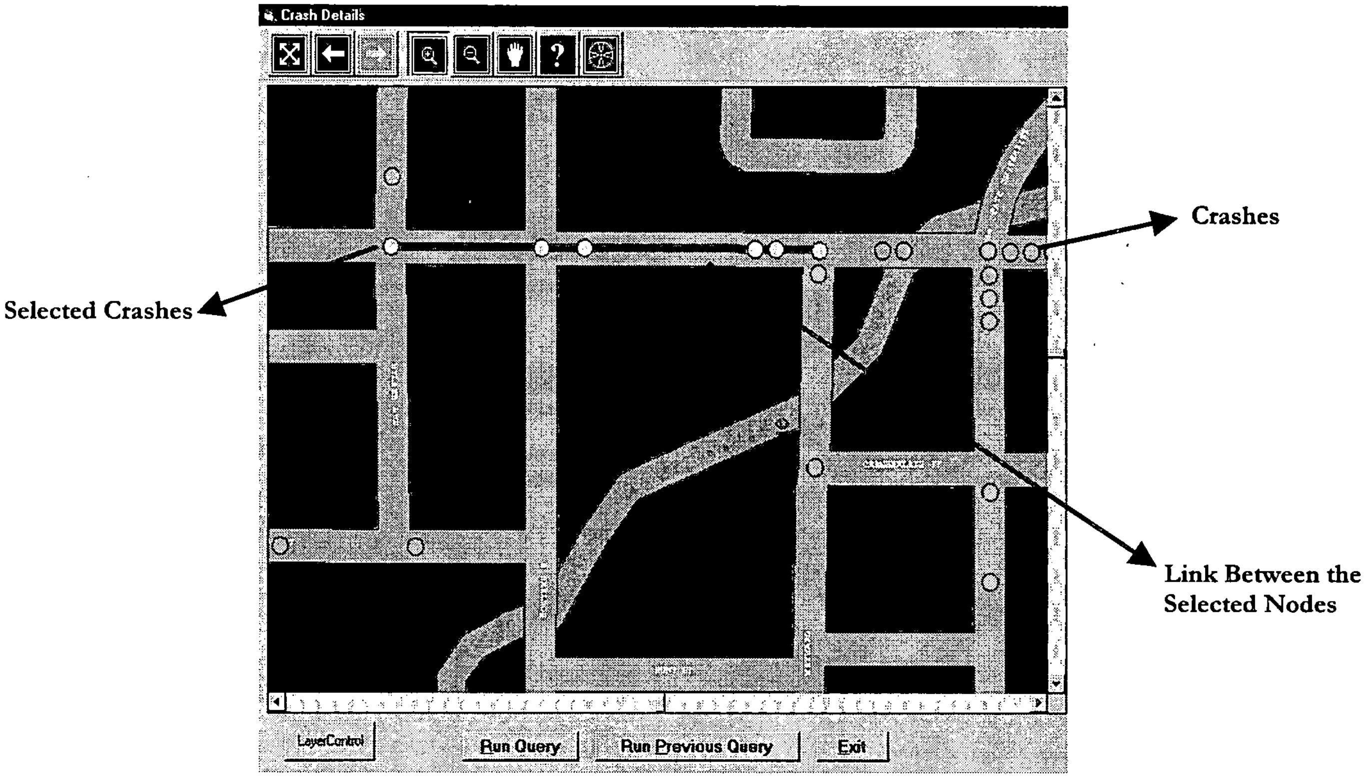

FIGURE 4 Area-Wide Crash Selection Using Map Interface

SOFTWARE DESIGN AND DEVELOPMENT

A model showing how the map interface works has been developed. The model shows every detail of the internal workings of the map interface, thereby providing the possibility to easily modify the system in future. For example, if a crash is to be selected on a map, the system needs input from the user. The input can be a crash number, a segment on a roadway or a polygon on the map. The system takes the input, processes it and generates the output, which in this case is a crash/set of crashes. Every detail of how the system works and the logic involved is documented using the Unified Modeling Language (UML). UML is a language for visualizing, specifying, constructing, and documenting the artifacts of a software-intensive system (*6*). Figure 5 shows a high level view of user interaction with the map interface.

The code required to build the map interface is written in Microsoft Visual Basic 6.0 and MapObjects 2.0. Visual Basic is a high-level programming language and supports all windows-based computers.

This program is in development stages and will be provided to users next year. An Access-ALAS user survey will be done in November 2000 to determine whether map interface features everything the user needs and the software program will be modified accordingly.

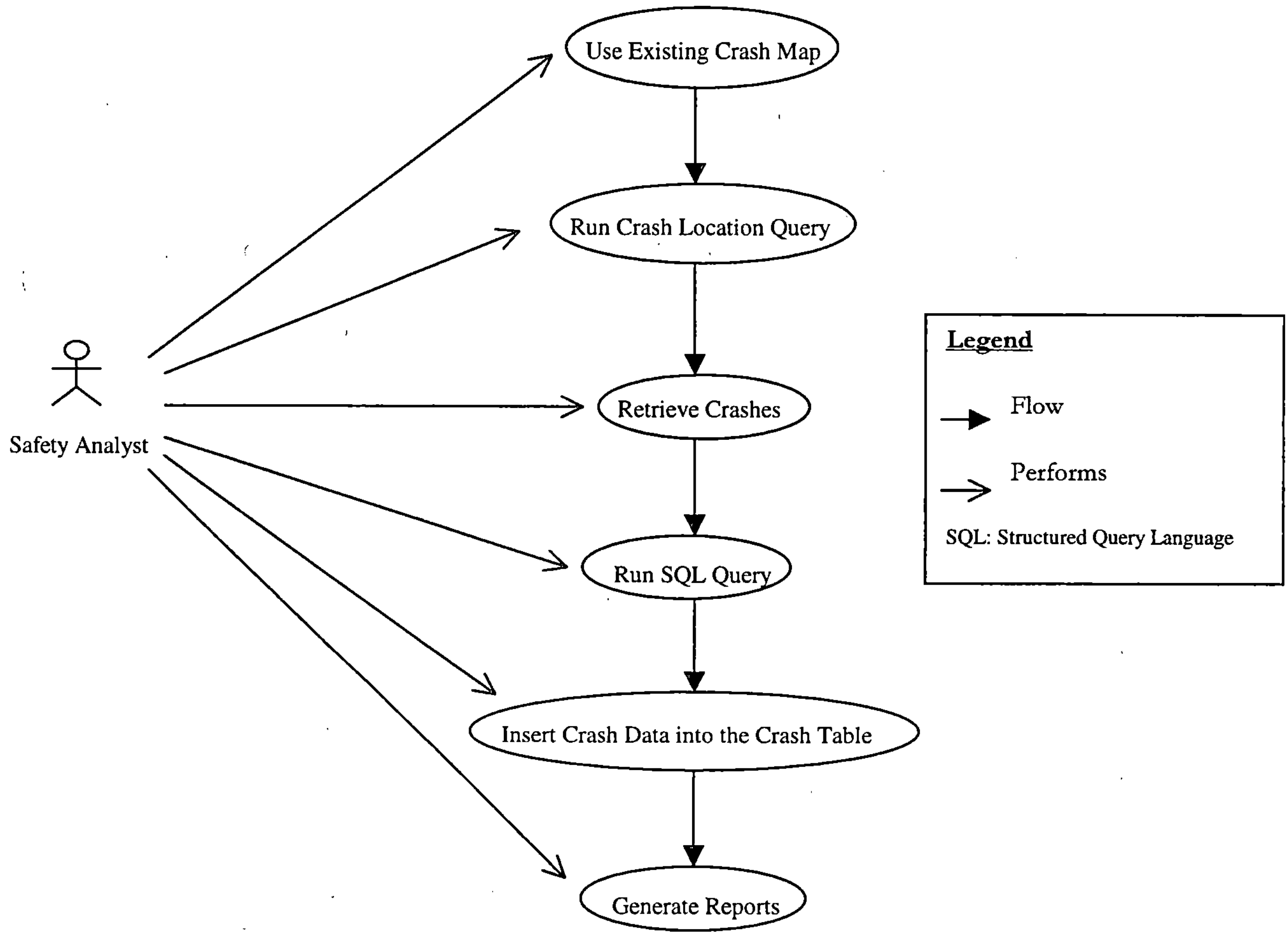

FIGURE 5 Working of Access-ALAS Map Interface

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APPLICATION OF INFLUENCE DIAGRAM AND SEQUENTIAL HYPOTHESIS TESTING METHOD IN BRIDGE MAINTENANCE DECISION MAKING

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ABSTRACT

Bridge inspection is important in making maintenance or reconstruction decisions. Sometimes a single inspection fails to accurately reflect the actual condition, and additional inspections are needed. The paper addresses this issue from the perspective of minimizing the potential loss, that is, balancing the financial consequences of inspecting to obtain additional information and the financial consequences of making a decision with inadequate information.

An influence diagram is a tool used for decision support. The research described uses the framework of an influence diagram to build a bridge inspection and maintenance model. The sequential hypothesis testing method is a statistical approach used to estimate the expected cost of the whole decision process. In this paper, it is used to calculate the potential loss for the influence diagram model.

INTRODUCTION

There are more than 575,000 bridges over 20 feet in length in the United States recorded in the existing National Bridge Inventory (NBI) (1). Inspecting, maintaining, and repairing these bridges can be time-consuming and costly. More importantly, if an agency cannot detect an unsafe condition or does not make good decisions, the resulting price is perhaps not merely a waste of funds, but the loss of life.

The detection of construction deficiencies, deterioration, and subsequent damage depends on the inspection devices and methods used. The emergence of new technologies and the invention of better detection implements will narrow the difference between the detected condition and the actual condition. Yet, making good decisions on the time intervals of maintenance and rehabilitation (M&R) activities and the distribution of funds, which is much more complex, needs both expert experience and good data.

Errors during data collection are inevitable. Some errors are caused by inexperienced inspectors, while others are caused by inspection devices. Although researchers and engineers have developed some methods for dealing with the effects of uncertainty in data collection, one hundred percent accuracy of the data cannot be guaranteed. Inadequate data can cause cumulative effects in a series of decisions. However, too much data collection can result in the waste of money in bridge inspection and additional efforts spent on data processing. Therefore, determining the appropriate amount of data collection is an urgent task, and solving the problem can lead to improved decisions.

For example, assume an agency has used some data to determine whether a particular repair action should be applied to a bridge. The agency is then faced with the question of whether to collect a little more data to confirm the accuracy of the data already collected, or whether to apply the data in hand directly to decision making. In the first case, the agency should calculate the funds needed for additional data collection; in the latter case, the agency should estimate the cost of the risk for direct application of the existing data. Then, a comparison is made, and the lower cost alternative is the optimal choice. Such is a general illustration of the procedure for solving the problem

The research described uses influence diagrams to represent the bridge inspection and maintenance decision process. The sequential hypothesis testing method is applied, within the context of influence diagrams, to address the question of how much data to collect before making a maintenance or rehabilitation (M&R) decision. The paper describes bridge management, maintenance, and data collection practices, the combining of influence diagrams and the sequential hypothesis testing method, and the model developed.

DATA MANAGEMENT AND DATA NEEDS FOR DECISION MAKING

Bridge Management Systems

A bridge management system (BMS) is a tool that can help the agencies responsible for bridges make accurate decisions. BMS typically include inventory and condition data, and decision support algorithms. The National Bridge Inventory (NBI) contains information about the status of the nation's bridges and magnitude of the funding needs. Yet, it cannot be classified as a BMS because it does not provide decision support. However, the NBI information can be used as the foundation for a BMS.

BMS at various levels of detail and completeness have been developed by a number of agencies. PONTIS and BRIDGIT are two commonly used BMS. PONTIS is designed to help agencies optimize network-level budgets and programs for maintaining and improving bridges (2). The approach PONTIS uses is "top-down". Both budgets and standards are used to optimize policies, which are then used to select corresponding maintenance and rehabilitation (M&R) projects. BRIDGIT, which performs similar functions, was developed under the National Cooperative Highway Research Program (NCHRP) (3). BRIDGIT follows a "bottom-up" approach. In this approach, standards are used to plan projects, and these plans are used to estimate costs. The costs are compared with the budgets, and the result of the comparison is used to modify the original standards.

Bridge Maintenance Decision-making

In making bridge maintenance decisions, the bridge management agencies must decide how to allocate funds for M&R activities among a network of bridges and over time to maximize the funds' use. The common question concerning M&R decisions is: "which M&R activities should be performed on which bridges in the network?"

To answer the question, information on the current condition, which is provided by bridge inspection, is needed. In addition, the decision-maker needs forecast information on future condition, provided by a performance prediction model corresponding to specific bridges. The current condition information is used to support the decisions made on M&R activities. Therefore, the answer is not merely saying "yes" or "no" to the funds distribution, but also an explanation of "why" and a specification of "how".

In addition to deciding what M&R activities to take, the agency must also determine the frequency of M&R activities. While M&R frequencies can be defined at fixed intervals, sometimes M&R activities must be done in real time or at shorter intervals. For example, Cook and Lytton (4) addressed the issue of timing pavement M&R activities using a variable trigger point. Another group of researchers, Frangopol et al. (5), present the relationship between time and the cumulative maintenance cost, under different maintenance strategies.

The Federal Highway Administration uses the NBI to evaluate projects eligible for federal funds and to allocate these funds to the states. The current program is designed to target replacement activity as well as bridge rehabilitation activity, and the type of activity is based on the bridge rating obtained by inspection. The Highway Bridge Repair and Replacement Program (HBRRP) uses a discretionary bridge rating to allocate funds (6).

Data Collection

To obtain the information for the above appraisal, M&R decision–making, and prediction of bridge deterioration, data collection is essential. Before collecting data, the agency must determine what kind of data to collect. Therefore, the inspection should have a definite purpose. Depending on this purpose, data may or may not be recorded.

Different approaches have been used to collect different types of data. For example, visual inspection is used to collect surface data such as deck cracks; non-destructive testing or evaluation (NDT/NDE) is used to examine materials for structural integrity; and some destructive evaluation, such as burning, drilling, and grinding, is used to test chloride contamination level. For further discussion of these issues see (7).

Currently, BMS research is focused primarily on data collection methods and algorithm development to support the goal of optimal decisions for bridge maintenance and rehabilitation. For example, Turner and Richardson (8) describe bridge data needs and identify what data have the greatest impact on management decisions. Sanford and McNeil (9) address the broader issues in data collection, such as the role of geographic information systems (GIS) in linking BMS data with other data spatially, and the use of NDE in bridge inspection. Unfortunately, research on data collection and on decision support algorithms for management has been separate; that is, the algorithms assume the data collected are complete and error free. It is de facto not such a case. Therefore, further studies should concentrate on the integration of the two areas of research.

DATA REQUIREMENTS FOR BRIDGE MANAGEMENT

Managing data involves storing, filtering, and manipulating data throughout its life cycle. It is a series of processes to convert a large amount of raw data into formalized and useful information, so that the information can be used as the basis for appropriate decision-making. Components of data management for the decision support process include data collection, data preparation, data interpretation (analysis), and data application (Figure 1). Data requirements analysis determines whether there is a need to perform data collection. Generally speaking, most of the data in the BMS are renewed at 2-year intervals. But when some "special cases" occur, intermediate inspection is needed. Such "special cases" include a fracture found in the main structure of the bridge, or a scour critical bridge in a river with increasing water volume (10).

As stated above, manual and automated inspection are the two main methods for data collection. Because data collection can be costly, it is necessary for the decision-maker to determine what kind of data should be collected.

As data are obtained initially, they may be "noisy" or "dirty". Therefore, data preparation is used to filter it. Filtering technologies include regression algorithms and neural networks. The former has the advantage of easy calculation, especially for linear modeling. It also has the disadvantage of imprecision. The major strengths of the latter are the ability to model complex and non-linear relationships, and the unrivaled ability at low-level pattern recognition. Its major weakness is the complexity in calculation (11).

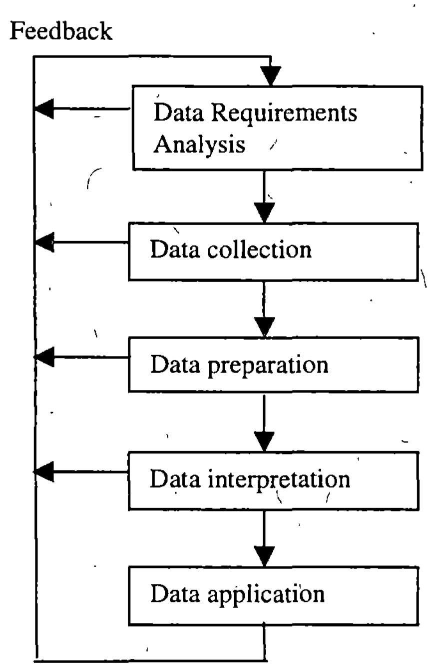


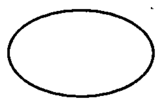
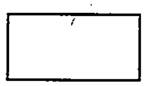
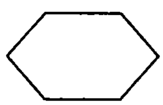
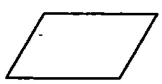
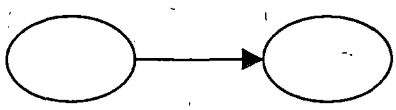
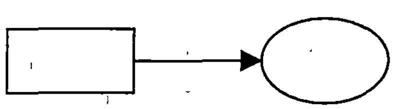
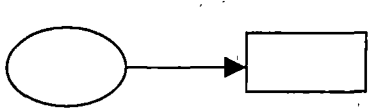
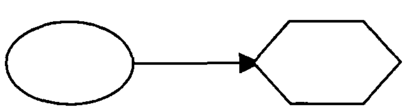
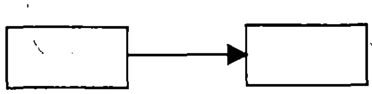
FIGURE 1. Data Management for Decision Support Process

Data integration utilizes methodology and technology to combine different types of data, or data in different periods, to derive more information. For instance, the determination of fatigue crack level could be integrated with Average Annual Daily Traffic (AADT), environmental factors, and material quality. Fatigue cracking is measured using NDE methods, which provide a quantitative measure of condition. However, current BMS are primarily designed to accept subjective condition ratings rather than measured data (9), and this leads to a flexibility or uncertainty in categorizing other types of data. Thus, as data integration provides tools to obtain information from BMS, it should focus on investigating opportunities for constructing future BMS to use more quantitative data as inputs (10).

An influence diagram, which represents a decision process at a particular point in time, can be used to determine the types of data that should be collected, based on the value the data provide. Nodes and arrows are the main components of an influence diagram (Table 1) (12, 13). In the diagram, three types of nodes are connected by arrows: chance nodes, decision nodes, and outcome nodes.

Influence diagrams clearly show the dependencies between data and the state of knowledge at a decision point, and they enable the evaluation of the outcome under different scenarios.

TABLE 1. Influence Diagram Components

Nodes and Arrow	Explanation
	Chance node indicates that an event could happen; its occurrence is beyond the control of the decision maker.
	Decision node indicates that a decision is made; it is under the control of the decision node.
	Outcome (also called objective in ANLYTICA) node indicates what the final result.
	Index node is used to help create table dimensions.
	The previous chance node affects the probability of the subsequent chance.
	The decision affects the probability of the subsequent chance.
	The decision is made knowing the probability of the chance occurrence.
	The occurrence of the outcome is contingent on the chance probability.
	The previous decision is made before the subsequent one.

SEQUENTIAL HYPOTHESIS TESTING METHOD

The sequential hypothesis testing method is a statistical tool used to derive the optimal scheme from several alternatives. Each alternative is represented by a hypothesis. Because a decision in one time period may affect the decision in the next time period, the choice of the optimal alternative for the whole time period is based on a Markov decision process. In each time period, the optimal value for different decisions is calculated and used as the input for the next time period. When the process terminates, the final optimal value is obtained, and the sequential decisions are determined. The core of Markov decision process is a Finite State Markov Chain which evolves over time. See (14) for additional details.

Research in this area has generated optimal strategies for bridge inspection and prediction of bridge deterioration rates. For example, Madanat (15) addresses the issues of making optimal decisions about pavement M&R under uncertainty by means of decision tools such as a decision tree. Mauch and Madanat (16) describe the use of survival modeling techniques to develop deterioration models.

As discussed, the inspection of a bridge deck is not error free. Therefore, all inspection and maintenance decisions should be coordinated so that the potential loss for the decisions is the minimum. Potential loss is the possible cost resulting from an incorrect decision. This can be accounted for by using a hypothesis testing model. The following formulation is adapted from (17).

In a sequential hypothesis testing model, after each measurement, the inspector is given the choice of either accepting one of the two hypotheses, or collecting an additional measurement prior to making a decision. That is, the inspector can

- accept the null hypothesis H_0 : the measurements are generated by f_0 ;
- accept the alternative hypothesis H_1 : the measurements are generated by f_1 ; or
- delay the decision until one more measurement is taken.

The distributions f_0 and f_1 describe a sequence of observations, which in this application represent the measurements of deterioration at different points on the bridge deck, in the substructure, or other part of the bridge.

The state of the information can be represented by the probability of H_0 being true, which is denoted by p_k. (It could also be based on the probability of it being false, because the two formulations are symmetric). Using Bayesian method:

$$p_{k+1} = \frac{p_k f_0(Z_{k+1})}{p_k f_0(Z_{k+1}) + (1 - p_k) f_1(Z_{k+1})}, \quad k=0,1,\ldots N-1, \tag{1}$$

and

$$p_0 = \frac{p f_0(Z_0)}{p f_0(Z_0) + (1 - p) f_1(Z_0)} \tag{2}$$

where: p is the initial assessment of the probability of H_0 being true,

k represents the time period, and

Z_k represents the measurement of deterioration at different points on the bridge.

The term "loss" refers to the negative consequences of making an "incorrect" decision. Expected losses related to the selection among competing hypotheses are associated with two types of errors: type I error (reject the null hypothesis H_0 when it is true) will lead to loss L_0; type II error (accept the null hypothesis H_0 when it is false) will lead to loss L_1. Comparing these losses, and the cost of taking additional measurements to make a decision, the recursive expression for calculating the minimum expected cost ($\overline{J}_k(p_k)$) from the kth period until the end of the inspection process is:

$$\overline{J}_k(p_k) = \min[(1-p_k)L_0, p_k L_1, C + E\left\{\overline{J}_{k+1}\left[\frac{p_k f_0(Z_{k+1})}{p_k f_0(Z_{k+1}) + (1-p_k) f_1(Z_{k+1})}\right]\right\}] \tag{3}$$

where: k=0,1,...N-2,

E is the expected value,

and

$$\overline{J}_{N-1}(p_{N-1}) = \min[(1-p_{N-1})L_0, p_{N-1}L_1] \tag{4}$$

where: N is the time period, and

C is the inspection cost.

Therefore, the decision rule for the last period (N-1) is:

If (1- p_{N-1}) $L_0 \leq p_{N-1}L_1 \Rightarrow p_{N-1} \geq \frac{L_0}{L_0 + L_1}$, then accept f_0.

If $p_{N-1} \leq \frac{L_0}{L_0 + L_1}$, then accept f_1.

For the previous periods k=0...N-2, the optimal policy is also a threshold policy, given by:

If $p_k \geq \alpha_k$, then accept f_0.

If $p_k \leq \beta_k$, then accept f_1.

If $\beta_k \leq p_k \leq \alpha_k$, then take one more observation.

where α_k, β_k are determined from:

$$\beta_k L_1 = C + E\{\overline{J}_{k+1}(p_{k+1})\} \tag{5}$$

$$(1-\alpha_k)L_0 = C + E\{\overline{J}_{k+1}(p_{k+1})\} \tag{6}$$

Furthermore, as k$\rightarrow \infty$, α and β will converge to $\overline{\alpha}$ and $\overline{\beta}$. Therefore, the optimal policy can be approximated by the stationary (time invariant) policy:

If $p_k \geq \overline{\alpha}$, then accept f_0.

If $p_k \leq \overline{\beta}$, then accept f_1.

If $\overline{\beta} \leq p_k \leq \overline{\alpha}$, then take one more observation.

To apply to the problem addressed by the paper, p corresponds to the overestimation probability; L_0 is the potential loss of skipping necessary M&R; L_1 is the potential loss of doing unnecessary M&R; C is the inspection cost; and f_0 and f_1 are functions that reflect the deterioration level under overestimation and underestimation, respectively. This is because as more inspections are made, the judgment should more accurately reflect the actual condition, so that the overestimation or underestimation probability is reduced.

RESEARCH METHODS AND TECHNIQUES

Influence Diagram for Bridge Inspection

An influence diagram has been constructed using ANALYTICA®, a tool that visually and quantitatively simulates decision making. The application software has the advantage of easily creating and displaying the structure of influence diagram, as well as doing mathematical calculation and analysis for each node. Figures 2, 3, 4, and 5 illustrate the decision model built in ANALYTICA, and Table 2 lists the node definitions.

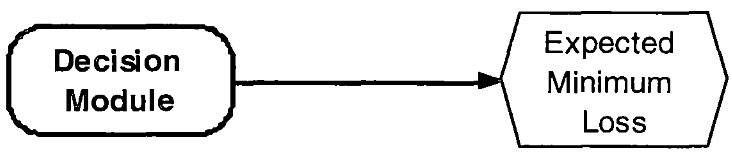


FIGURE 2. Top level of Influence Diagram Module

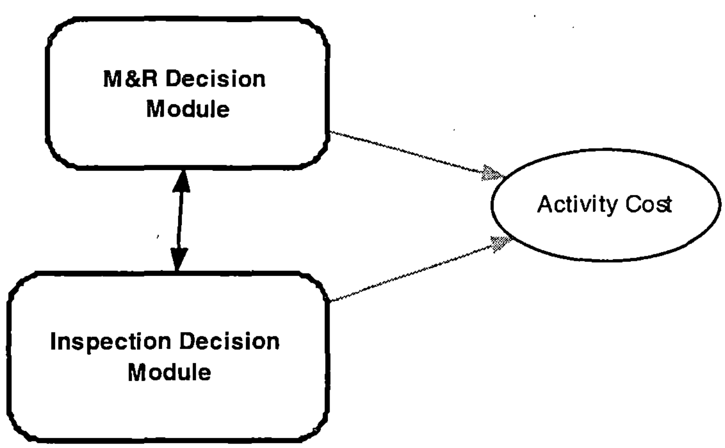


FIGURE 3. Middle level of Influence Diagram Module

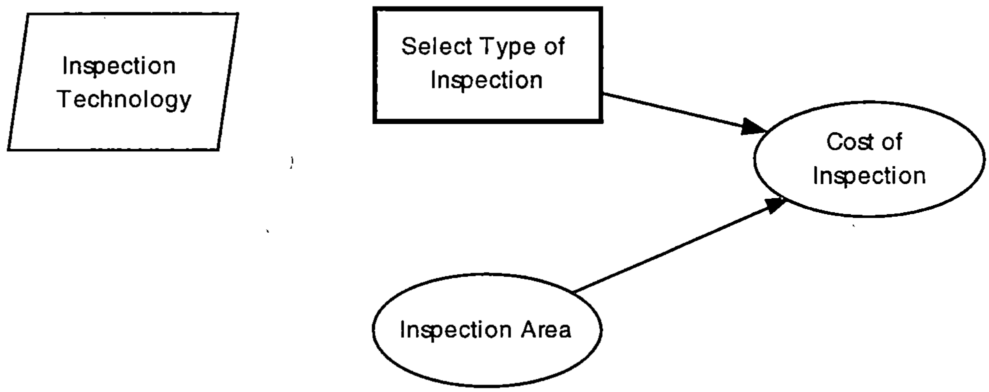


FIGURE 4. Lower level – Inspection Module

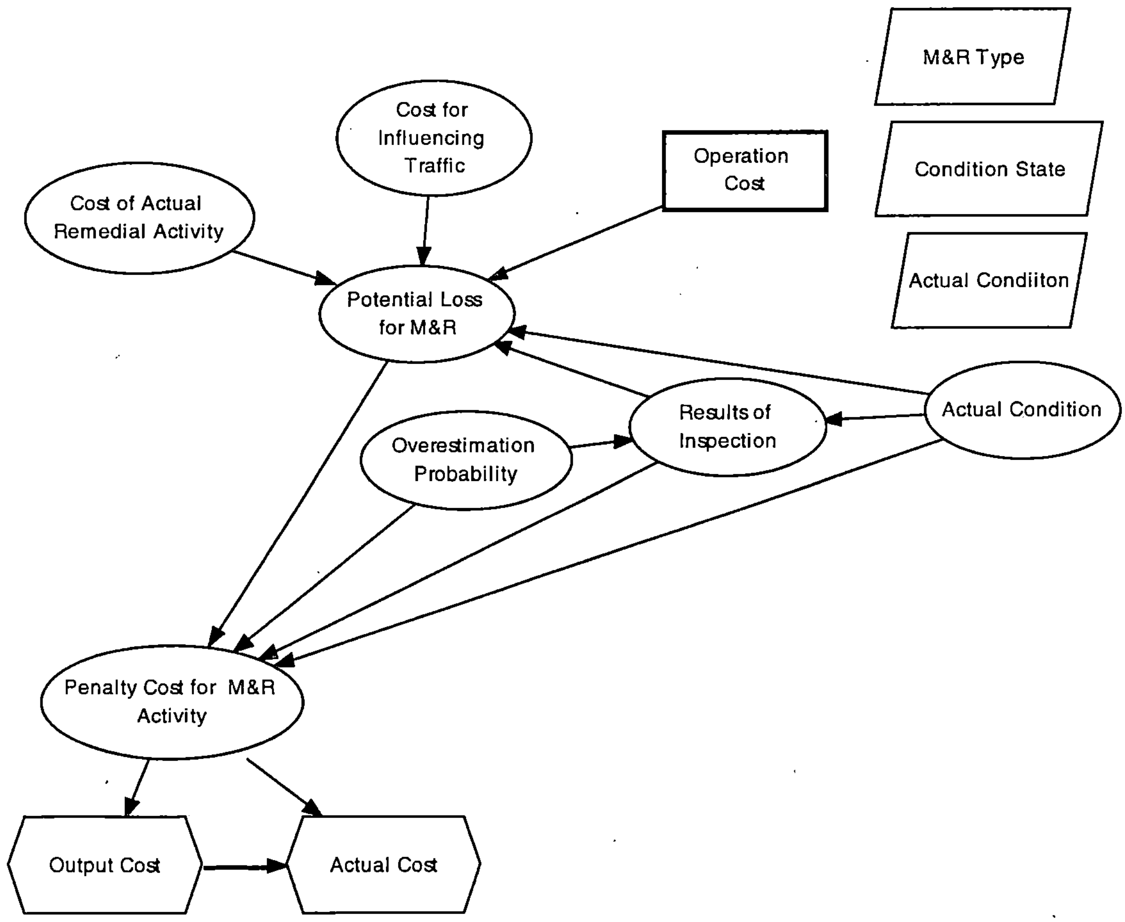


FIGURE 5. Lower level – Maintenance Module

TABLE 2. Influence Diagram Node Definitions

Node	Definition
Activity Cost (Objective)	The accumulating cost for each time period.
Actual Condition (Chance)	A condition state used by PONTIS.
Actual Condition (Index)	Combined with the condition states used by PONTIS, it gives the probability of how close the inspection results are to actual condition.
Actual Cost (Objective)	Cost value used in calculating activity cost.
Condition State (Index)	The bridge condition defined by PONTIS.
Cost for Influencing Traffic (Chance)	The cost because of traffic delays, obstructions, or detours. It can also be affected by the M&R action chosen or the inspection activity.
Cost of Actual Remedial Activity (Chance)	The cost for appropriate remedial action
Cost of Inspection (Chance)	The present inspection cost, which is determined by the type of inspection.
Expected Minimum Loss (Objective)	The calculated results of the optimal expected loss for this time period.
Inspection Area (Chance)	The area that the engineers inspect.
Inspection Technology (Index)	The inspection technology available in the module.
M&R Type (Index)	The types of maintenance that could be conducted.
Operation Cost (Chance)	The cost for different levels of maintenance.
Output Cost (Objective)	Combined with inspection cost, it is used to compare with previous M&R cost.
Overestimation Probability (Chance)	The probability of overestimating the actual bridge condition, i.e., the actual condition is worse than the inspection result.
Penalty Cost for M&R Activity (Chance)	The expected loss caused by doing maintenance and rehabilitation due to overestimating or underestimating the bridge condition.
Potential Loss for M&R (Chance)	The cost for M&R actions, no matter whether it is actually unnecessary because of overestimating or put off because of underestimating.
Results of Inspection (Chance)	The condition state provided by inspection engineers based on PONTIS rating standard.
Select Type of Inspection (Chance)	Determine what kind of inspection (visual inspection or NDE) should be conducted. Types of inspection include visual inspection and NDE evaluation.

The model consists of three levels of modules. The first level is the *decision module* (Figure 2). Its output is the *expected minimum loss*. The second level consists of two modules — *M&R decision module* and *inspection decision module* (Figure 3). Their output, *activity cost*, calculates the cumulative cost or potential loss for each step, i.e., the cost for inspection or the potential loss for making M&R decisions. The third level is the calculation of the *cost of inspection* and M&R potential loss (Figures 4 and 5). For the former, it is the multiplication of inspection area and the inspection cost per unit area, which is determined by the technology used. For the latter, it is more complex to determine the M&R potential loss. The core is the node *potential loss for M&R*. Before obtaining the expected penalty cost, the *actual condition* and *results of inspection* are compared. If the actual condition is poorer than the *results of inspection* indicate, that is, if the condition is overestimated, the penalty cost should be the potential loss for skipping necessary M&R multiplied by its *overestimation probability*. On the other hand, if the *actual condition* is better than the *results of inspection* indicate, that is, if the condition is underestimated, the penalty cost should be the *potential loss for doing unnecessary M&R* multiplied its underestimation probability, which is equal to one minus the *overestimation probability*. If the results of inspection match the actual condition, no penalty cost is generated. Two objective nodes are drawn from the penalty cost node. The *output cost* node is used for comparison, and the *actual cost* is used for computation.

Explanation of Models

Figure 6 shows the algorithm for inspection and maintenance decision making, and Table 3 illustrates the cost estimation process.

```
While not make M&R decisions
    If time=1 then
    Do regular inspection;
    Activity cost[time]=cost of inspection;
    Else
       If cost of inspection[time-1]+output cost[time]>actual cost[time-1] then
          Do additional inspection;
          Activity cost[time]=activity cost[time-1]+cost of inspection[time];
       Else
          Reduce one inspection and do M&R; (*)
          Activity cost[time]=activity cost[time-1]-cost of inspection[time-1]
                               +actual cost[time-1];
          cost of inspection=0;
          actual cost=0;
       endif
    endif
    time=time+1;
endwhile
```

* one inspection is deducted because of the assumption that inspection is made before the M&R action is done.

FIGURE 6. Inspection and Maintenance Decision Algorithm

For the example in Table 3 (assuming that the inspection results are worse than the actual condition), when time equals one, the initial values for inspection, output cost, and actual cost are simulated to be set to, respectively, 10, 90, 90. Because the assumption is that inspection is always done before maintenance, the activity cost is set to be equal to the inspection cost 10. When time equals 2, because an inspection is done in the first time, the values for the three nodes should be 10, 64, 64. The output cost is obtained using the potential loss for M&R (the value is set to 450) times overestimation probability (initial value is set to 0.2), which is calculated recursively using formula (2). That is, as more inspection occurs, the potential loss should be deducted. Because the previous inspection cost plus the output cost is less than the previous actual cost (10+64<90), the activity cost is 10+10, which is 20. That means the decision to perform the initial inspection is correct. The second 10 is also the assumption that in the second time, an inspection will be performed before maintenance activity is done. Similarly, the third and fourth activity costs are 30 and 40, respectively. For the fifth time, because the previous inspection cost plus the output cost is no less than the previous actual cost (10+21>=31), the activity cost should be 40-10+31=60. This means that the assumption that inspection should be performed in the fourth time is incorrect; instead, maintenance should be done at the time. After that, all the inspection cost and actual cost is set to 0. After the process is over, the final activity cost is 61, and the activities performed are: inspection, inspection, inspection, and M&R. Figure 6 illustrates the simulation activity cost at each time step.

TABLE 3. Simulation of Influence Diagram Model

Time	Inspection cost	Output cost	Actual cost	Activity cost	Comments
1	10	90	90	10	Assume inspection is usually done in the first time
2	10	64	64	10+10=20	10+64<90
3	10	45	45	20+10=30	10+45<64
4	10	31	31	30+10=40	10+31<45
5	0	21	21	40-10+31=61	10+21>=31
6	0	14	0	61+0=61	0+14>0
7	0	10	0	61+0=61	0+10>0
8	0	6	0	61+0=61	0+6>0
9	0	4	0	61+0=61	0+4>0
10	0	3	0	61+0=61	0+3>0
Results	Therefore, the expected minimum loss is 61. Because the time latent reason, that is, we assume inspection each time, an inspection should be deducted. That means we need three inspections before we make final decisions.				

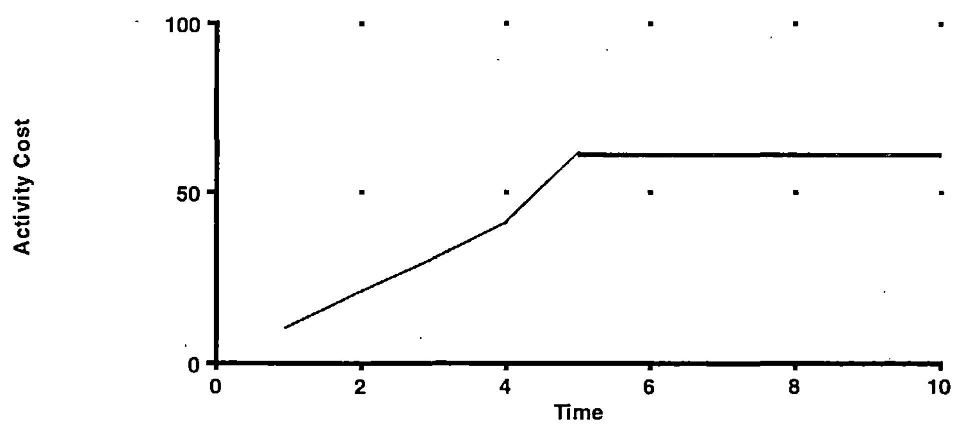


FIGURE 7. Model Simulation Results

CONCLUSION

The combination of influence diagrams and the sequential hypothesis testing method provides an approach for predicting the optimal number of inspections and estimating the minimum potential loss for the decisions. The model is appropriate for bridge maintenance decision making at the project level, i.e., the potential loss analysis for each bridge should be calculated individually. One problem to be noted in the model is that because of functional limitations of the software, the total number of inspections is always one more than the actual number of inspections.

Significant work remains. For instance, how can the definitions of the diagram nodes be modified so that they better reflect the actual decision process? In applying the approach to the network level of bridge inspection, what changes should be made to the influence diagram? How can a cost estimation model be converted into a bridge maintenance decision support system? And how could such a procedure be combined with existing BMS, such as PONTIS or BRIDGIT? Solving these problems will help to improve the bridge inspection and maintenance decision making process.

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ACCESS MANAGEMENT: MEDIAN TREATMENTS AND THEIR EFFECTS ON REDUCING CRASHES ON IOWA'S URBAN ARTERIALS

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ABSTRACT

Safety on our nations roadways is becoming of great concern with an increasing amount of vehicles on the roads resulting in higher vehicle miles traveled per year. Mitigating efforts to minimize crashes have been heavily studied in recent years. With almost 3.4 million people injured in 1997, the increased concern of safety measures has spurred the investigation of access management techniques. Specifically, the use of medians in the State of Iowa as a mitigating effort to reduce the occurrence and severity of crashes on arterial roadways will be investigated. This paper attempts to justify the use of medians on arterials as a function of overall reduction in economic loss due to injury and property damage costs. This justification will investigate crashes on several segments of arterials across the State of Iowa. Selected segments will be compared in terms of overall crash costs as a function of median usage and this paper will attempt to justify improvements based on cost comparisons.

INTRODUCTION

"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." *Federal Highway Administration* (1)

According to the National Highway Traffic Safety Administration, in 1997, 42,013 persons were killed in automobile crashes and 3,399,000 were injured (2). This fact alone has urged me to investigate mitigating efforts to reduce traffic crashes in the State of Iowa. The purpose of this report is to give a justification for the increased use of access management techniques in Iowa and the United States.

The State of Iowa currently has a comprehensive Access Management Awareness Program, a program sponsored by the Iowa Department of Transportation, underway and this paper will expand upon this report by focusing on levels of access management. Specifically this report will evaluate the effectiveness of medians on lowering the number of crashes on roadways resulting in injury and property damage and whether the economic savings pays for the cost of improvements on existing or new infrastructure. This paper will expand upon recent literature, and the findings that access management techniques have greatly reduced the number of crashes on U.S. roadways and particularly across the State of Iowa. This report will address the following questions:

- What are anticipated and actual goals of access management programs?
- How significant do medians contribute to lowering crash rates on urban arterials in Iowa?
- Does access management provide a justification for improvements on our existing infrastructure?

Benefits of Access Management

There are a number of social and economic benefits that are received from implementing access management strategies. The most important benefit from access management is that motorists experience fewer crashes. Access management can reduce crashes as much as 50% while increasing travel speeds by as much as 40% (3). A reduction of congestion is experienced resulting in the efficient flow of vehicles through the road networks and improved travel time on roads with good access controls. The Colorado DOT reported that access-related crashes cost the State approximately $900 million in 1994 (1).

Fewer delays and crashes reduce transportation costs for individuals, businesses, and taxpayers. Fewer delays allow businesses to ship and receive more products, thereby increasing their output. Property damage and personal injury are obvious results of crashes. A reduction in crashes helps to control the increasing insurance costs relative to property, health, and life, and reduces the loss of work time, benefiting both employees and employers.

Costs of Crashes

"While it is unpleasant to think of human life and injury in monetary terms, this is precisely what must be done if safety improvements are to be taken into account...when highway investment decisions are made" (3). Motor vehicle crashes cost society millions of dollars as well in lost wages, insurance costs, etc. According to 1994 statistics, the economic cost of crashes in the State of Iowa totaled $1.4 billion dollars (4). According to the Iowa Department of Transportation (IDOT), cost estimates related to motor vehicle crashes exist for fatal injuries and personal injuries. This data is represented below in Table 1.

TABLE 1 Injury Classification Estimated Associated Costs

Fatal Injury	$800,000
Major Injury	$120,000
Minor Injury	$8,000
Possible Injury	$2,000

Although there exists discrepancies between agencies to these exact dollar values, for the purposes of evaluation, IDOT values will be used. According to the U.S. 20 Corridor Study, crash rates by hundred million vehicle miles of travel on different levels of highway type exist (3). According to this study, for example, if an existing 2-lane highway is to be rebuilt as a 4-lane expressway, the average fatality rate on the highway might be expected to decline from 2.34 to 1.20 crashes per million vehicle miles traveled. I suggest that if this same analysis were applied to urban arterials using raised medians to those that do not, we may expect to find similar conclusions. I would also suggest that if we were to lower crash rates, the difference in the indexed "costs" of personal injuries and fatalities, combined with property damage, would in turn pay for the differences in construction costs for implementing raised medians on arterial corridors versus not using median control techniques. For example, the construction costs for a 1-mile section of arterial without a raised median is approximately $1 million. The same arterial could be built with a raised median for approximately $1.1 million. The costs of right-of-way can only be handled on a case-by-case basis; therefore will not be taken into account in this report.

There are many types of costs associated with crashes. The two costs used in this report include: property damage and an indexed injury costs per crash. Not only do crashes cost the persons involved in the crash, but can be translated into costs covered by taxpayers and employers. Costs covered by employers resulting from motor vehicle crashes fall into three categories.

- Health fringe benefit costs are the costs of fringe benefits paid because of illness and injury. This covers a wide range of costs including, but not limited too, Worker's Compensation, sick leave, medial insurance, etc.

- Non-fringe costs are incurred to employers when crashes happen when employed including: legal expenses, damage to property, health insurance, and loss of worker productivity.

- Wage premiums; the wages paid to employees who work in high-risk environments and are considered payment in advance for possible injuries.

These costs amounted to $18.3 billion dollars in health fringe benefits costs, $24.8 billion in non-fringe costs, and $11.6 billion in wage-risk premiums for a total of $54.7 billion in employer costs nationwide in 1995 dollars (6). This report will compare only the costs of property damage and the indexed costs of injuries for analysis purposes.

ACCESS MANAGEMENT TECHNIQUES

Access management requires careful planning considerations of land use and of the current transportation system to be an effective technique for reducing crashes. The following techniques exemplify current trends in access management techniques (1).

- Regulate driveway spacing, corner clearance, and sight distance.
- Increase minimum lot frontage and setback requirements along thoroughfares and regulate lot width-to-depth.
- Restrict the number of driveways per existing parcel or lot and consolidate access wherever feasible.
- Treat properties under the same ownership and those consolidated for development as one property for the purposes of access control.
- The use of Continuous Two-way Left-turn lanes (TWLTL).
- Utilization of medians.
- Driveway Spacing and Regulation.

The regulation of a minimum spacing between driveways is a regulatory method to reduce the number of driveways, driveway density, on major arterial streets. This technique, diagramed in Figure 1, can be implemented at existing locations or during driveway permit stages. Driveway location and design greatly affects the ability to safely enter and exit a parcel of property. If not properly spaced, drivers may be unable to see or anticipate traffic on the road, and traveling motorists on the road may not have adequate time to slow or yield to entering vehicles. Reasonable spacing between driveways is important to the safety and capacity of roadways, as well as the appearance of a corridor. Aesthetically, corridors can be become more pleasing to the eye by limiting the number of access points thereby increasing the amount of open/green space that can be used by businesses. By promoting consolidation of driveways for adjacent parcel owners, numerous property owners are connected with common entrances. This allows vehicles to circulate between businesses without entering the major arterials (4). "This technique indirectly reduces the frequency of conflicts by separating adjacent, basic conflict areas and limiting the number of basic conflict points per length of highway" (7). A conflict point occurs where the path of two traffic movements intersect. This technique is expected to reduce the severity of rear-end collisions as it allows more deceleration distance for motorists.

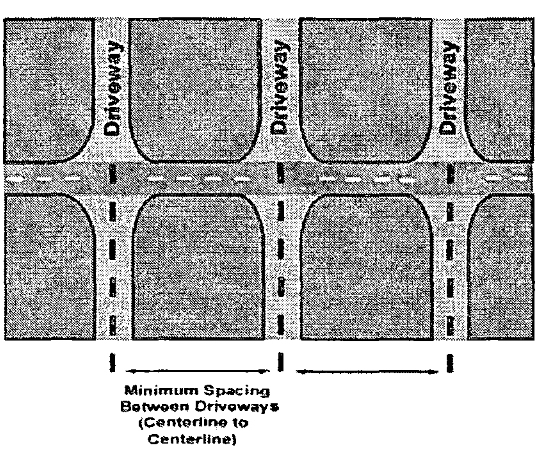


FIGURE 1 Driveway Spacing Regulations

The use of a continuous two-way-left-turning-lane (TWLTL) in Figure 2, utilizes a middle lane designated as a left-turning lane only. These lanes allow turning traffic to be removed from thru traffic lanes, thereby decreasing the risk of rear-end crashes. Traffic turning left into a drive is removed from the overall traffic flow of the system and these vehicles must yield to all other traffic before exiting the roadway onto a parcel of property.

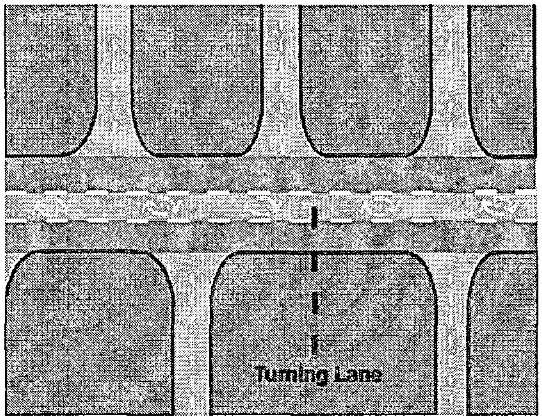


FIGURE 2 Continuous Two-Way-Left-Turning-Lane

MEDIAN TREATMENTS

AASHTO's Green Book defines a raised median as "...the portion of the highway separating the traveled way for traffic in opposing directions," and that, "A median is highly desirable on arterials carrying four or more lanes" (8). The utilization of medians, raised or grass, physically separates two or more lanes of traffic and limits turning and cross-traffic. Medians greatly reduce the occurrences of crashes by restricting motorists from making excessive left-turns, by providing freedom from interference of opposing traffic, by limiting the number of motorists crossing over lanes of oncoming traffic, and by reducing headlight glare (1). Raised medians may also greatly affect the aesthetics of a corridor by allowing limited landscaping to be done.

The use of medians and the median type to be used depends on many factors of the roadway and land development. First, the type of highway that is being planned, (i.e., two-lane or four-lane,) and the existing or predicted traffic volumes that would warrant the use and type of median technique, dictate whether a median will be used or not. Second, several land use considerations have to be made when determining the use of medians including: development of property adjacent to the road, (i.e., commercial or industrial), the average annual daily traffic (AADT) these land uses will produce, and the availability of right-of-way. Thirdly, the speeds and existing types of access control may warrant the treatment of medians (9).

There are four basic types of medians recommended for use by the Iowa DOT. These are depressed, raised, painted, and closed.

1. Depressed medians are used with divided highways, which have a common ditch, usually considered grassy medians

2. Raised medians are typically used in urban areas where curbs are used on the outside of the pavement. Raised medians are typically constructed of concrete or may be a concrete-framed raised earth median, which is occasionally landscaped.

3. Painted medians are yellow paint-striped and thus provide no physical barrier other than distance between opposing traffic lanes. Their efficiency relies solely on a driver's ability to readily perceive the painted indication (7).

4. Closed medians are normally used on freeways or expressways in locations with very high traffic volumes or where the amount of separation between opposing directions of travel is minimal or being reduced (such as when inside lanes are being added in the existing median).

Potential Problems Mitigated by Medians

Potential safety problems exist at every intersection or driveway along highway corridors. What effect does median implementation have on the likelihood of crashes? These potential crash points, known as conflicts, can be minimized using the aforementioned median designs. The following diagrams illustrate these conflict points on major arterial roadways. A normal intersection without medians results in 32 conflict points relevant to intersection traffic flow. With the use of a raised (or in some instances a painted median), these conflict points decrease to 18. These techniques prevent some turning movements, while removing turning traffic from thru lanes. These types of medians can be used with or without traffic signaling thereby increasing the prevalent locations of their implementation.

The implementation of direction median openings, diagramed in Figure 3, is a technique that decreases the major conflict points to four. This technique does not allow thru traffic traveling north and south. This type of design offers one of the safest types of access management that can be applied to intersections, but is not widely used across the State of Iowa.

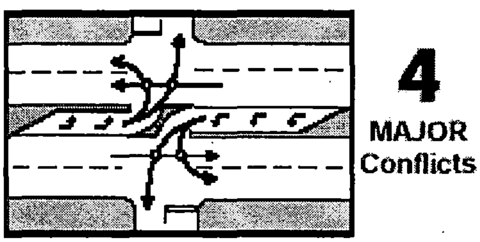


FIGURE 3 Directional Median Opening

The next intersection-controlling median is the left-in only, diagramed in Figure 4. This technique only allows for left-turns off major roadways and eliminates any other type of movement. Again, these are only sparsely used in the State, but minimize the major conflict points to two.

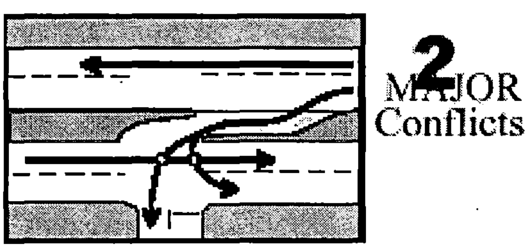


FIGURE 4 Left-In Only Median

The final type of median treatment that can be used on roadways is that of a full-center median. This treatment decreases the major conflict points to zero. A restrictive median is the safest type of access management technique, but does not allow left-turns to be made either onto a parcel of property or roadway. These techniques are commonly used in Iowa on arterials with high volumes of traffic. This is represented below in Figure 5.

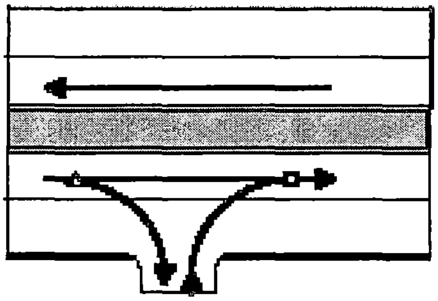


FIGURE 5 Full-Center Median

A REVIEW OF MEDIANS AND THEIR EFFECTS

In one of the earliest studies conducted in 1968, indicated the safety advantages of nontraversable medians over traversable medians in urban areas by comparing the crash experience on two multi-lane streets in Springfield, Illinois. Crash rates on the painted (traversable) median were 2.63 times that of the curbed (nontraversable) median. Analysis also showed that the street having the curbed median had lower crash rates at all locations (i.e., intersections and mid-blocks other than driveways and private driveways) (10).

"It has been demonstrated that median access control results in a substantial reduction in the number of crashes together with a reduction in the associated social and economic costs of injuries, fatalities, and property damage" (3). A review of literature reveals a consensus that the undivided cross section is associated with more crashes than the TLWLT and raised-curb median (10).

According to Bonneson, et al. 1998, the raised-curb median treatment is associated with fewer crashes than the undivided cross section and TWLTL (10). This is based on a safety model represented by regression analysis that predicts the expected annual crash frequency for a street segment based on its length, average daily traffic demand, median treatment, adjacent land use and total access point density.

According to statistics from a retrofit project in Atlanta, Georgia, an existing TWLTL was replaced by a raised median on a 4.34-mile section of highway during 1989-1990. The results of a before and after study show a 37 percent reduction in total crashes and a 48 percent drop in injury rate (3).

Analysis of Jimmy Carter Blvd., also in Atlanta, Georgia shows that installation of the Jersey median resulted in a substantial reduction in the number of crashes and the crash rates. The total number of crashes decreased by 32 percent and the crash rates decreased by 27 percent on the north section and by 47 percent on the south section when the TWLTL was replaced with the Jersey Barrier (3).

Median width has proven to decrease the number, and severity, of crashes in Illinois and Utah. According to the FHWA, total crash rate appears to decline steadily with increasing median width from 0 to 110 ft. The rate of multi-vehicle crashes declines the most with increasing median width. Some grassy medians are used as recovery areas by out-of-control vehicles, and by vehicles avoiding crashes. This exemplifies how crash rates decrease as median widths increase; vehicles exiting a roadway into a median area allow out-of-control vehicles to avoid crashes (11).

According to case studies conducted by the Center for Transportation Research and Education, Iowa State University, of the six case studies, three used retrofit median treatments. These three cases located in Ankeny, Clive, and Mason City, showed an approximate average of 36 percent decrease in average annual total traffic crashes (1).

METHODOLOGY

The methods for testing whether median usage has an effect on crash occurrences and costs incurred by crashes will be discussed in the following section. It was determined that this study would be a "blind" test of several locations across Iowa. A blind test is one that is based solely on selection criterion rather than on first hand knowledge of a study area. There are limitations to this type of study, which will be discussed further in the limitations section. The blind selection of case study location and arterial segments are discussed further below, but were made using predetermined criterion.

Case study selection was made using a variety of criteria. Using Geographic Information Systems software (GIS), cities were selected using the following criteria. First, cities that had a population of over 30,000 persons were selected as primary location candidates. This criterion was used to limit the overall number of urban areas that would be used for analysis. Thus, the cities with the largest population in the State of Iowa were selected as urban case study areas, which makes extrapolating the analysis to larger metropolises, implementing access management, possible. Second, of these 11 cities, four cities were selected based on population, roadway criteria, and time and resource constraints.

The four cities included:

- Ames
- Des Moines
- Cedar Rapids
- Iowa City

Corridor Selection

The selection of corridors in the State of Iowa was based on several criteria. First, the corridors had to be within the city limits of one of the above four cities. Next, traffic corridors were selected based on the their AADT's. An AADT of equal to, or greater than 10,000 vehicles per day, was used as a baseline of daily traffic volume. These corridors were also selected based on their federal functional classification established by the Federal Highway Administration. The federal classifications that are used in the U.S. include the following:

- Interstate
- Freeway
- Expressway
- Arterial
- Arterial Connector
- Trunk
- Municipal Arterial
- Municipal Collector
- Municipal Service
- Trunk Collector
- Area service

The only Federal Classified Roads that were selected for analysis were those classified as arterials. This classification was selected on the basis that arterials are assumed to have the highest AADT's and crash rates in urban areas.

After initial selection of appropriate cities was completed using GIS, the actual roadway segments were selected. GIS software (ArcView 3.1/3.2) was used as the basic analytical software for this analysis. A Beta version of GIS-ALAS (Accident Location and Analysis System) was used to investigate crash incidents, rates, and locations based on 1997 data for the State of Iowa. Selection of roadway segments that met the following criteria were selected as case study segments:

- Segments were selected if their AADT was 10,000 or greater.
- Roadways were selected if they were federally classified as Arterials.
- Roadways utilizing medians were selected if they also met the above criteria.

The first criterion for selection was the level of access management (high v. low) employed on the existing traffic segments. The level of access management in this case refers simply to, *whether a segment utilizes a median versus one that does not.* This resulted in a number of case study candidates, which were then queried to determine eligible case study segments. Using ArcView, the final selection was based on segments with comparable levels of AADT's and median usage, which resulted in numerous segments per city; ones that used medians and ones that did not. Approximately equal segment lengths in each city were determined based on the available segment lengths resulting from the above selection process. From this, eight case study segments were identified that would be used in the final analysis, these are indicated in Table 2. These selected road segments vary in length, as it was very difficult to find segments using medians and those that did not, which matched the predetermined selection criterion. To alleviate this problem, averages were used in the analysis of crashes and costs associated with these crashes.

TABLE 2 Selected Road Segments

Location	Route	Median Usage	Length
Ames	Lincoln Way	Yes	2-Miles
Ames	Lincoln Way	No	2-Miles
Cedar Rapids	1st Avenue	Yes	1-Mile
Cedar Rapids	1st Avenue	No	.5-Mile
Des Moines	University Ave	Yes	3-Miles
Des Moines	University Ave	No	3-Miles
Iowa City	U.S. 6	Yes	3-Miles
Iowa City	U.S. 1	No	3-Miles

From these case studies, crash analysis techniques were used to determine:

1. Total number of crashes on each segment.
2. Injury severity of crashes on segments.
3. Calculated indexed injury cost per segment.
4. The total number of injured persons involved in each crash.
5. The total property damage sustained in each crash.

Summary crash and property data was obtained for each of the segments and is presented in Table 3. It is interesting that half of the cities had higher property damage on segments with medians and half had lower property damages. This data is contrary to what other researchers have found in the past.

TABLE 3 Summary Crash and Property Damage for Selected Segments

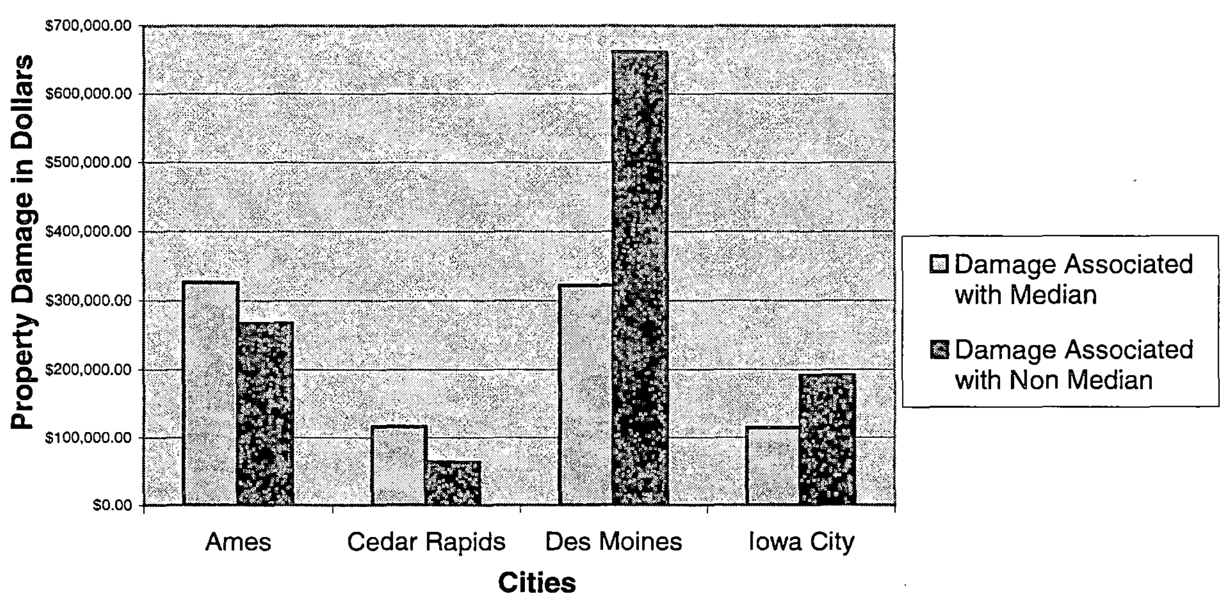
Location	Route	Median Usage	Total Crashes 1997	Property Damage
Ames	Lincoln Way	Yes	102	$326,638.00
Ames	Lincoln Way	No	130	$267,677.00
Cedar Rapids	1st Avenue	Yes	52	$116,714.00
Cedar Rapids	1st Avenue	No	37	$63,650.00
Des Moines	University Avenue	Yes	90	$322,239.00
Des Moines	University Avenue	No	209	$661,509.00
Iowa City	U.S. Hwy 1	Yes	47	$114,888.00
Iowa City	U.S. Hwy 6	No	61	$191,800.00

ANALYSIS

The following section constitutes the analysis of crashes on eight segments of arterials across the State of Iowa. There are two data sets that were analyzed: injury data and property damage data. Figure 6 represents the total costs of property damage in all cities. This table only illustrates the amount of property damage in comparison to segments, but does little to illustrate a difference in property damages per segments in relation to the number of crashes.

The mean property damage per crash was calculated to find deviations in damage for segments using a high level of access management versus segments that use a low level of access management. It was found that only Iowa City experienced, on average, a lower amount of property damage on segments using a high level of access management, while Ames, Cedar Rapids, and Des Moines experienced an increased amount of property damage on segments with a high level of access management. On average, it was found that property damage on segments with a raised median was $3,025.70 in comparison to an average of $2,710.84 per crash on segments not using a raised median. Although, the total amount of property damage for segments using raised medians was $880,479.00 compared to $1,184,636.00 on segments without a median. This is one of the only comparisons between segments that justifies the use of raised medians.

FIGURE 6 Property Damage in Select Cities



Injury costs were calculated using the Iowa DOT Indexed Costs for Crash Injuries. Using the personal injury values, total costs per injury could be calculated for each of the segments. In all but the Iowa City segments, total indexed injury costs were higher on segments using medians versus segments not using raised medians. This is very interesting to observe, as one would be led to believe that injury costs on segments using access management would be lower, as we would expect the total number of crashes to be lower. One of the explanations for this is the fact that there were more major injuries on segments using raised medians.

The last analysis of segments and crashes was comparing the injury rates to the level of access management. A calculated injury rate per crash showed that on average, the injury rate for crashes on arterials with raised medians was higher. Again, this is contrary to previous research on injuries and access management. Finally, the following table summarizes the total costs (property damage and indexed injury costs) for each of the cities. Although, only one of the cities shows a lower cost for the use of medians, a total savings for all cities is equivalent to $943,222.00.

TABLE 4 Total Costs of Crashes on Segments

City	Total Costs for Crashes-Median Usage	Total Costs for Crashes-Non Median Usage
Ames	$1,230,638.00	$693,677.00
Cedar Rapids	$270,714.00	$141,650.00
Des Moines	$1,476,239.00	$3,751,509.00
Iowa City	$1,302,888.00	$701,800.00

CONCLUSIONS AND LIMITATIONS

In summary, these case studies reveal that medians may not have the effect on crash reduction as theorized. Through analysis of segments, it was found that in most cases, injury rates and property damages actually increased on segments using raised medians. If we compare the previously discussed construction costs for arterials, we find that there are no cost savings, associated with crashes, between median and non-median usage. What do these results tell us? First, that the usage of medians as a crash reduction technique may not be warranted on arterial streets in Iowa. This may be a false conclusion, as there are many other factors, which will be discussed, that could affect the outcome of this analysis. Although, based on the methodology of selecting segments based on a blind technique, the analysis is valid. This leads to the shortcoming and limitations of this study.

First, the selection of segments was not made considering many important variables. Case study segments were based on a limited number of criteria. Had these segments been based on further criteria, such as access points, zoning of land, etc., the segments chosen would have inevitably been quite different. Finally, if more resources had been available, a more thorough study could have been completed taking into consideration these other variables. This study does not justify the initial argument because the data clearly shows that the treatment of arterial segments with medians, versus those not utilizing a type of median, has no significant effect on crash incidents, injury cost, or on property damage. Further research is needed to prove that the reduction in crashes will pay for the difference in construction/conversion costs on Iowa's arterials to improve our current transportation network. Referring back to the questions that would be addressed by this paper, we have failed to answer the following: "Does Access Management provide a justification for improvements on our existing infrastructure in terms of crash costs?"

Suggestions for Improvement

I suggest that this study should be repeated not using a bind selection technique. This technique allows one to do analysis to a certain extent, but has many limitations. Conclusions can be drawn from this report, but the data is not very useful or convincing to make valid and sound conclusions. This study would have to be repeated with first hand knowledge of a selected areas before conclusions can be drawn to the effectiveness of raised medians at limiting the number of crashes and severity. The severity of the crashes may in fact increase with the use of medians as this study has shown, but is that what is really happening on our transportation networks. Further research in this area is needed to determine that.

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ANALYTICAL MODELING OF GLUED LAMINATED GIRDER BRIDGES USING ANSYS

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ABSTRACT

This paper aims at developing a finite element model of glued laminated girder bridges that can predict accurately the analytical behavior of the bridges. Two models have been developed to study the characteristics of this type of bridge. Two case studies have been considered to validate the accuracy of the models. The first case study is a long span single-lane bridge, the Tuscaloosa Bridge, in Alabama. The second case study is a medium span two-lane bridge, the Cow Gulch Bridge, in Montana. The models show good correlation with the experimental data and hence, the models can be used to find the maximum deflections in the bridge.

INTRODUCTION

The age of wood spans human history. As a building material, wood is abundant, versatile, and easily obtainable. Although in the 20th century, concrete and steel replaced wood as the major materials for bridge construction, wood is still widely used for short and medium span bridges. Of the bridges in the United States with spans longer than 20 feet, approximately 12 percent of them are made of timber. In the US Department of Agriculture (USDA) Forest Service alone, approximately 7,500 timber bridges are in use, and more are built each year. The railroads have more than 1,500 miles of timber bridges and trestles in service *(1)*. In addition, timber bridges recently have attracted the attention of foreign countries like Canada, England, Japan, and Australia.

Timber's strength, lightweight, and energy-absorbing properties furnish features desirable for bridge construction. Timber has the capability to support short-term overloads and contrary to popular belief, provides good fire resistance qualities that meet or exceed those of other materials *(1)*. Timber is not damaged by continuous freezing and thawing and resists harmful effects of de-icing agents and hence, can be constructed in any weather conditions. Timber bridges do not require high skilled labor for construction. They also present a natural and aesthetically pleasing appearance, particularly in natural surroundings. Some examples of major American timber bridges are the First Bridge across the Mississippi River at Rock Island, IL, and William Howe's Connecticut River Bridge, at Springfield, MA.

GLUED LAMINATED GIRDER BRIDGES

Description of Glued Laminated Girder Bridges

Glued laminated (glulam) girder bridges are the most common type of timber bridges. The spans of these bridges range from 20 to 100 feet. In this type of bridges, the deck panels are laid transverse to the girders that run between supports. The deck panels consist of a series of laminated lumbers that are placed on edge and glued together on their wide faces. The panels are not interconnected and are normally about 4 feet in width and 5 to 7 inches in thickness. The girders are also glued laminated and are usually 5 to 12 inches in width, with depth to width ratios of 2 to 1 or greater. Lag bolts are used to connect the girders to the deck panels and this is responsible for the composite action between the deck and the girder. The bridge railing system consists of treated timber posts and a glued laminated rail, faced with a galvanized steel w-beam. The approach guardrail system is usually treated timber posts with galvanized steel w-beam. A typical glued laminated girder bridge is illustrated in Fig. 1.

Properties of Wood

Wood is an orthotropic material with unique and independent properties in different directions. Because of the orientation of the wood fibers, and the manner in which tree increases in diameter as it grows, properties vary along three mutually perpendicular axes: longitudinal, radial and tangential (Fig. 2). Since the differences in wood properties between the radial and tangential directions is minor compared to their mutual differences in the longitudinal direction, most wood properties for structural applications are given only for directions parallel to the grain (longitudinal) and perpendicular to the grain (radial and tangential).

The ANSYS finite element software denotes these material properties by associating them with the corresponding material axes (Fig. 2), as shown below:

Ex, y, z – Young's modulus in the longitudinal, tangential and radial directions respectively.

Gxy, yz, zx – Shear modulus in the x-y, y-z and z-x planes respectively.

νxy, yz, zx – Major Poisson's ratio in the x-y, y-z, and z-x planes respectively.

The ANSYS software relates the state of stress and strain in a body by the elasticity matrix, [D]. Fatal errors occur in the program if the inverse of the elasticity matrix [D]-1 is not positive definite. The [D]-1 matrix is also presumed to be symmetric. The use of Poisson's ratios for orthotropic materials causes confusion, so care should be taken in their use. To assure that the $[D]^{-1}$ matrix is positive definite and symmetric, the following relationship must be satisfied:

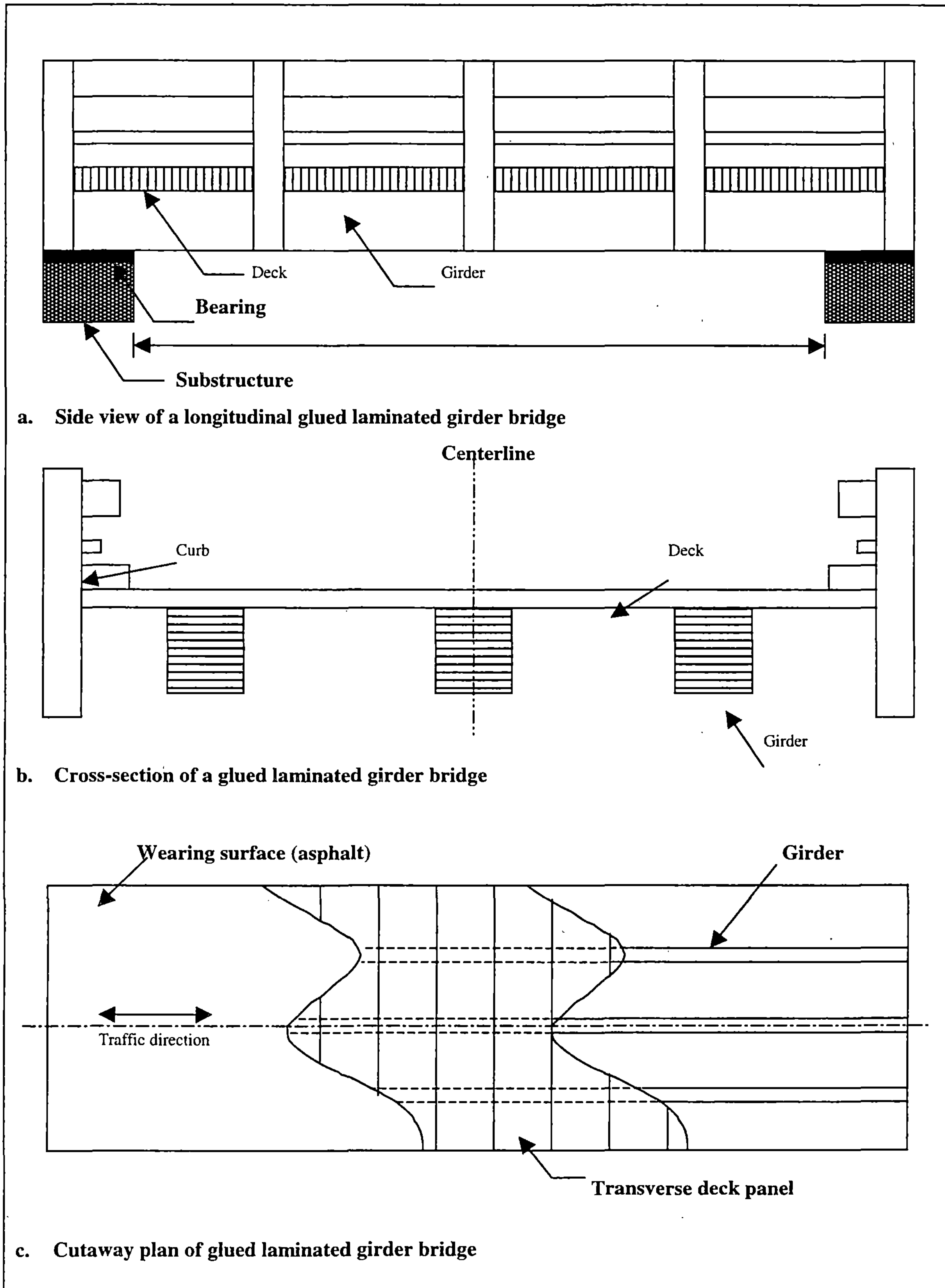
$$\nu_{ij} = \nu_{ji} E_i / E_j$$

where,

$i, j = x, y, z$, and $i \neq j$

$$G_{xy} \text{ (default)} = E_x E_y / (E_x - (1+2\nu_{xy}) E_y)$$

$$[D]^{-1} = \begin{pmatrix} 1/E_x & -\nu_{xy}/E_x & -\nu_{xy}/E_x & 0 & 0 & 0 \\ -\nu_{yx}/E_y & 1/E_y & -\nu_{yz}/E_y & 0 & 0 & 0 \\ -\nu_{zx}/E_z & -\nu_{xy}/E_z & 1/E_z & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/G_{xy} & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/G_{yz} & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/G_{xz} \end{pmatrix}$$



a. Side view of a longitudinal glued laminated girder bridge

b. Cross-section of a glued laminated girder bridge

c. Cutaway plan of glued laminated girder bridge

Fig. 1 – A typical glued laminated girder bridge

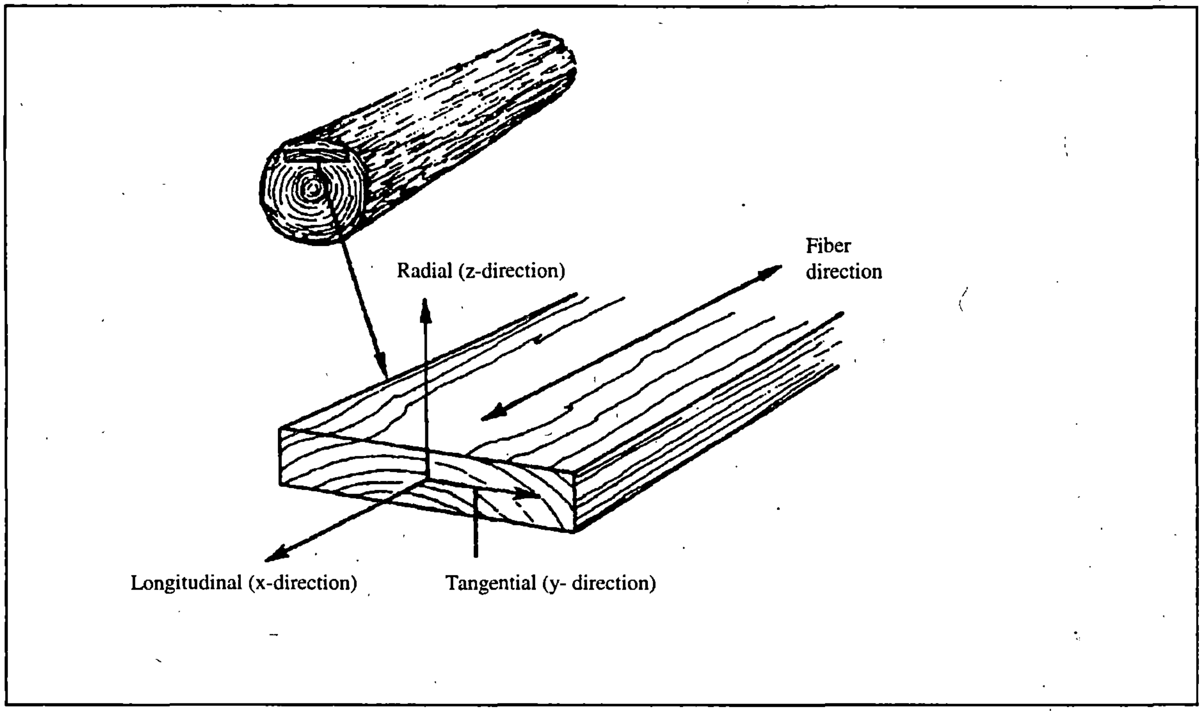


Fig. 2 – Three principal axes of wood with respect to grain direction and growth rings

ANALYTICAL MODELS OF BRIDGE

The ANSYS software (2) was used to describe the bridge behavior analytically because of its vast element library and powerful analysis techniques. The model was assembled by modeling the deck panels, girders, and curbs, if present. The panels were modeled using quadrilateral, elastic, and orthotropic shell elements. While in Model 1 the girders were modeled with 3-D elastic beam elements, in model 2 the girders were modeled with quadrilateral, elastic, and orthotropic shell elements. The curbs were modeled with 3-D elastic beam elements connected to the deck by rigid links. All these elements are present in the ANSYS element library.

Modeling the Deck Panels

The deck panels are laid out transverse to the girders. The deck panels are not interconnected and the assumption made in this regard is that the asphalt-wearing surface and friction between the adjacent panels contribute insignificantly towards continuity and load distribution between panels.

The four-noded shell element (SHELL63) (3) was chosen to model the deck panel. The element chosen has six degrees of freedom at each node: translation in the nodal x, y, and z directions and rotations about the nodal x, y, and z directions. This element can be used to model the orthotropic properties of wood and is defined by thickness, longitudinal and transverse moduli of elasticity, shear modulus and major or minor Poisson's ratio.

The longitudinal modulus of elasticity (parallel to grain of fiber) of the panels is substantially higher than the transverse modulus of elasticity (perpendicular to grain of fiber) and the modulus of elasticity tangential to the grain of fiber. *Fig. 3* shows the SHELL63 element used to model the deck panels.

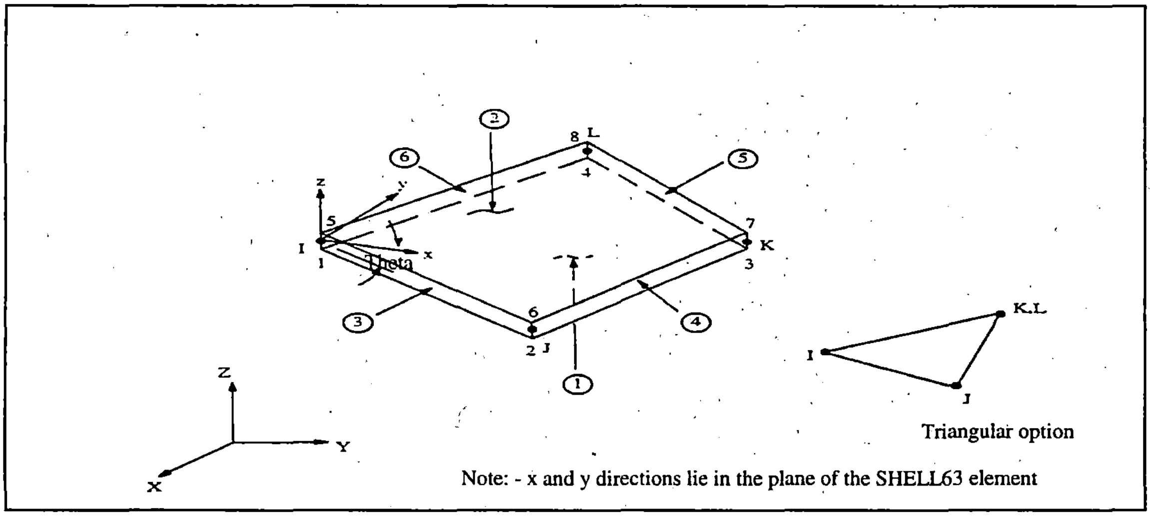


Fig. 3 – SHELL63 element used to model the deck panels

Modeling the Girders

In Model 1, the 3-D two-noded beam element (BEAM4) (3) was chosen to model the girders. The element has six degrees of freedom per node: translation about the x, y, and z directions and rotation about the x, y, and z directions. The element is defined by width, thickness, cross-sectional area, inertias about the x and y directions, longitudinal and transverse moduli of elasticity, shear modulus and major or minor Poisson's ratio. This however did not exactly represent the real situation on the field. The nodes of these beam elements are located halfway between the thicknesses of the girders. Hence, the abutment supports would have to be located at these nodes. In reality, however, the girder rests on the supports and hence, the supports should be located at the bottom of the girders. This led to the development of Model 2. Fig. 4 shows the BEAM4 element used to model the girders.

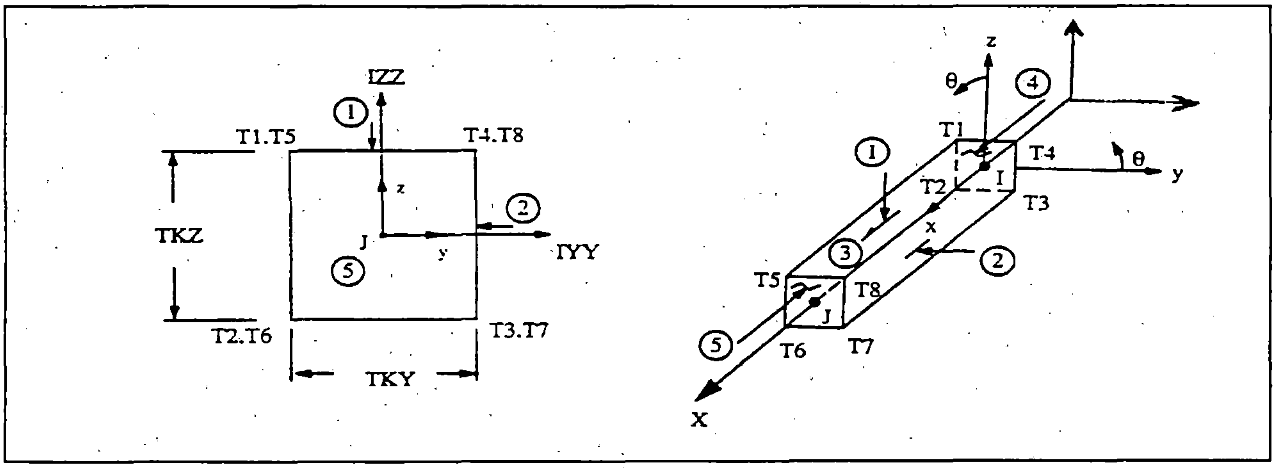


Fig. 4 – BEAM4 element used to model the girders

In Model 2, the girders were modeled with SHELL63 elements. This helped solve the problem of location of supports explained earlier. The girder was modeled as shell elements and the thickness of the shell elements was the width of the girder. Nodes were located at the bottom of the girders for addition of supports.

Bhari's research (4) showed that the assumption of full composite action between the girder and the deck was valid. Hence, in both the models explained above, full composite action was assumed. In Model 1, the composite action was idealized by rigid links (BEAM4 elements with very high flexural and axial stiffness) while in Model 2, the composite action was idealized by making the connecting nodes, between the deck panels and the girders, common nodes.

Modeling the Loads and Abutment Supports

The bridge was assumed to be simply supported since this assumption would be closer to the real situation and deflections would be conservative. The live load applied was truck wheel loads. The wheel contact areas were assumed to be small relative to the bridge and hence, were applied as concentrated point loads. Since the ANSYS software requires concentrated loads at nodes, the concentrated wheel loads were distributed to the nodes in the form of equivalent loads, since very rarely did the location of a wheel load correspond to the location of the panel node. Interpolation functions for rectangular elements (5) were used to distribute the concentrated wheel load to the four nodes of the shell element upon which the wheel load was located.

Note should be taken that the dead load or the permanent weight of all the structural and non-structural components of the bridges, including the roadway, sidewalks, railing, and wearing surface were not included in the load. Normally, girder bridges have curbs along their edges. These curbs stiffened the edges but have little or no contribution to the maximum deflections and stresses of the girders as we move away from the edge. The curbs (if present) were modeled as 3-D beams running along the edges of the bridge, connected to the deck by rigid links. This idealization was shown to be sufficient in the case studies explained later in the paper. Models of a typical glued laminated girder bridge are shown in Figs. 5 and 6.

CASE STUDIES

Two case studies were undertaken to test the model developed. The first case study was a medium span field bridge, the Cow Gulch Bridge, owned by Yellowstone County, Montana. This bridge was built from a grant received by the county from the Wood In Transportation program in 1996. The focus of the grant was to construct economical timber bridges, and to encourage involvement by a local timber laminating facility. The second case study was a long span, single lane field bridge, the Tuscaloosa County Bridge, in Alabama. This bridge consists of four simple spans but only the third span is analyzed in this paper. The material properties and field-test data for this bridge was obtained from Dlabola (5).

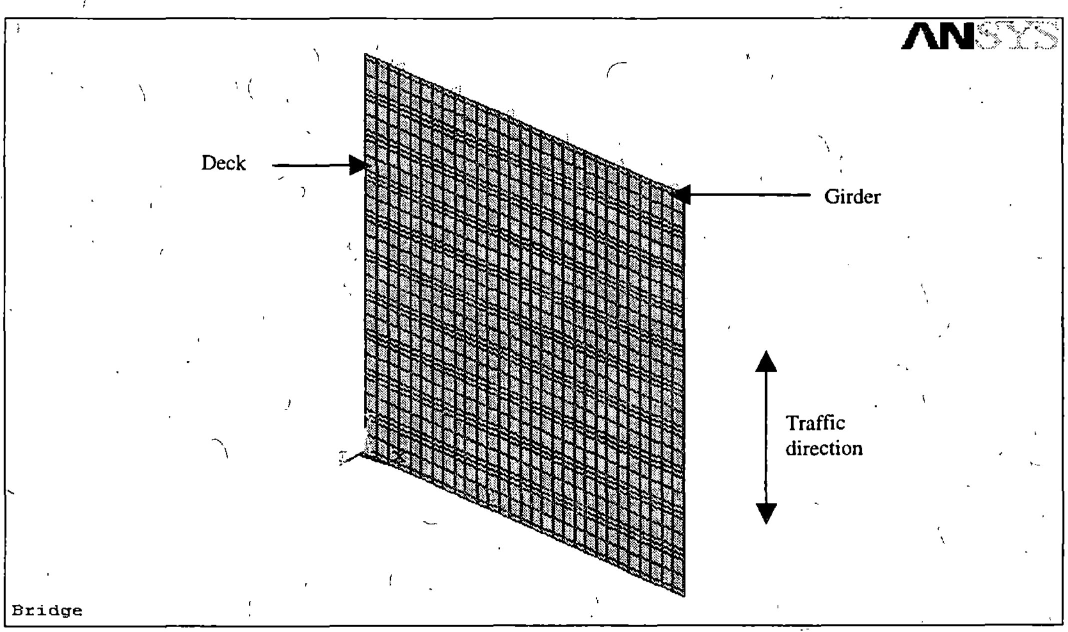


Fig. 5 – Model 1 of a typical glulam girder bridge

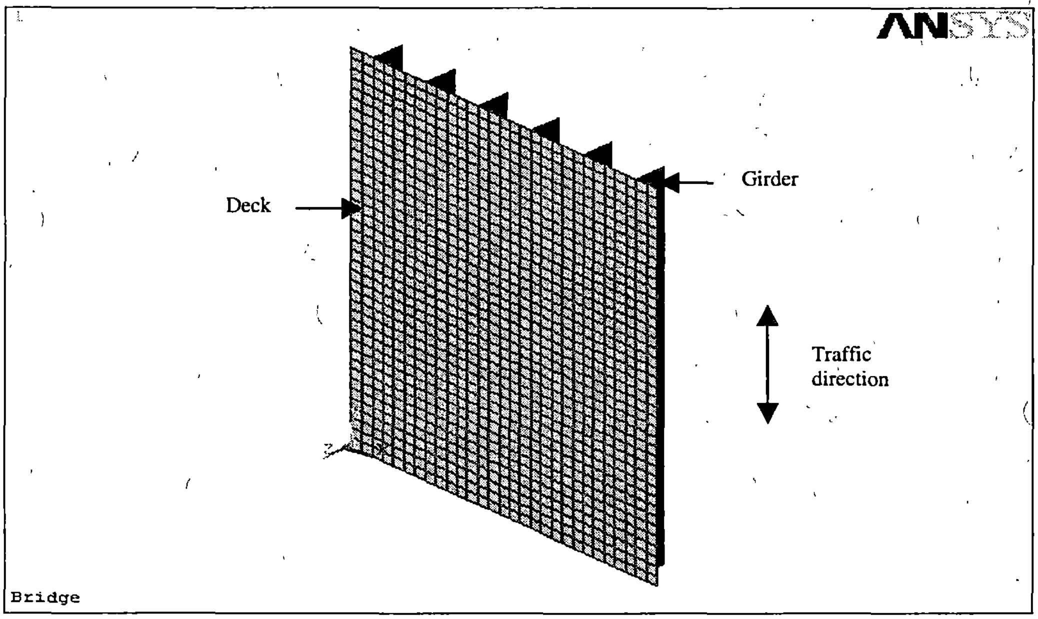


Fig. 6 – Model 2 for a typical glulam girder bridge

The Cow Gulch Bridge, Montana

The Cow Gulch Bridge is a glulam girder bridge made of Coast Douglas Fir, with six girders supporting the deck panels. The deck panels were about 4 feet in width, 28 feet in length, and 5.125 inches in thickness and were laid transversely on the girders. They were connected to the girders by lag bolts at 6 inches in from each edge of the panel. The bridge measured 38.5 feet in span, measured center-to-center of bearings. The girders had a nominal width of 8.75 inches, and a nominal thickness of 28.5 inches. Curbs with cross-section dimension 8 inches by 8 inches were present along the edges of the bridge. The deck panels had a longitudinal and transverse modulus of elasticity of 1800 kips-per-square inch (ksi) and 130 ksi respectively. The shear modulus of the deck panels was about 100 ksi. The girders had a longitudinal and transverse modulus of elasticity of 2000 ksi and 240 ksi respectively. The shear modulus of the girders was about 106 ksi (1).

Loading

The load case considered was a three-axle, fully loaded gravel truck with a gross vehicle weight of 54,000 pounds. The truck was located longitudinally on the bridge so that the rear axles were centered about the midspan of the bridge. It should be noted that the experimental deflections measured were only due to the live truckload. Since the load was placed such that the maximum deflections occurred at the midspan, the experimental deflections were measured at the midspan of each girder. The plan layout along with the loading case considered is shown in Fig. 7.

Results and Discussion

A comparison between experimental data and analytical results for the Cow Gulch Bridge is shown in Fig. 8. The graph shows the midspan deflections at each girder. The analytical results compare very well with the experimental values. The deflections of the girders based on the analytical model are on the average about 8% larger than the experimentally measured deflections. For a timber bridge with properties that vary in each direction, this is a very good correlation. The differences between the analytical results and the experimental values can be attributed to the assumed material properties.

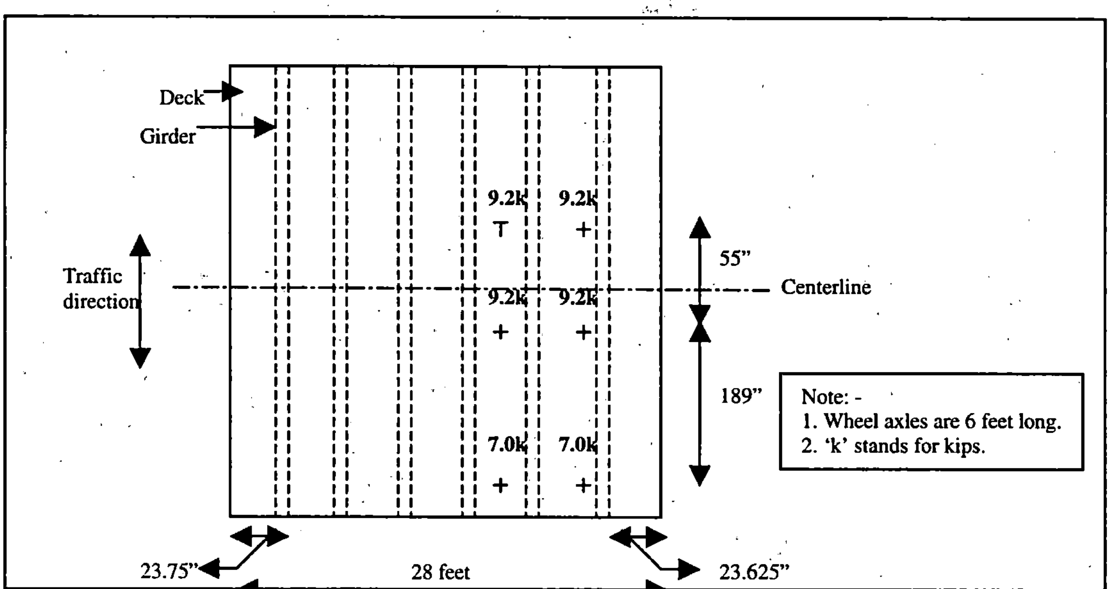


Fig. 7 – Plan layout of the Cow Gulch Bridge, Montana

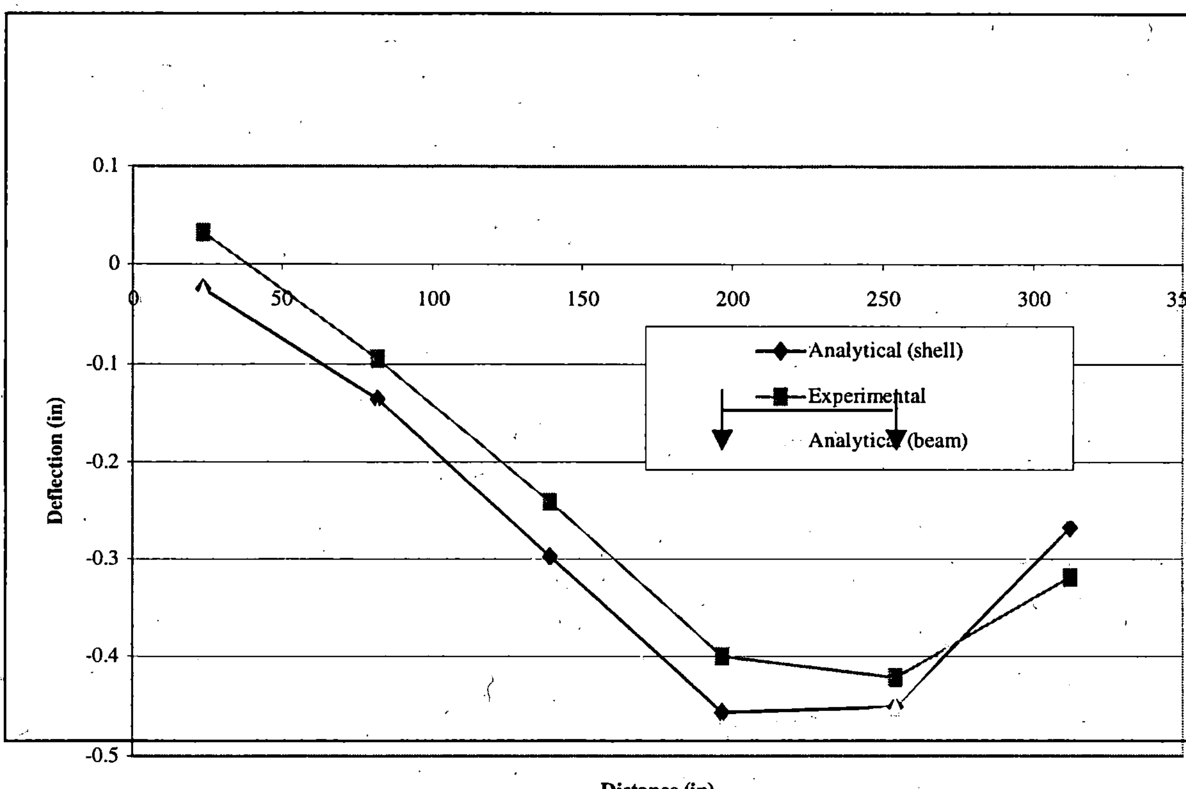


Fig. 8 – Panel deflections at midspan of girders – Cow Gulch Bridge

Model 1 and Model 2 compare very well with each other. There is more composite action between the deck and the girder in Model 2 and hence the deflections obtained from Model 2 are slightly lesser than the deflections from Model 1. The author also mentioned earlier about the problem of location of supports in Model 1. This did not seem to have any effect on Model 1. However, since Model 2 represented the real situation better, the author recommends Model 2 though Model 1 can be used as a good approximation.

A mesh sensitivity analysis on the analytical model showed that differences in deflections and bending stresses in the girders were smaller than 2% when the mesh size was 12 inches by 12 inches. The results discussed in the preceding paragraphs were obtained with a mesh size of 12 inches by 12 inches.

The Tuscaloosa Bridge, Alabama

The third span of the Tuscaloosa Bridge is a glulam girder bridge made of Southern Yellow Pine, with four girders supporting the deck panels. The deck panels were about 4 feet in width, 15.1 feet in length, and 5.125 inches in thickness and were laid transversely on the girders. They were connected to the girders by lag bolts at 6 inches in from the edge of each panel. The bridge measured 102.1 feet in span (center-to-center of bearings). The girders had a nominal width of 10.625 inches and a nominal thickness of 63.125 inches. The Tuscaloosa Bridge is a long span, single lane bridge with bigger girders than the Cow Gulch Bridge. The material properties for the analytical model were obtained from the National Design Supplement (7) for the particular combination of lumbers and grade of lamination. The deck panels had a longitudinal and transverse modulus of elasticity of 1930 ksi and 240 ksi respectively. The shear modulus of the deck panels was about106 ksi. The girders had a longitudinal and transverse modulus of elasticity of 1930 ksi and 240 ksi respectively. The shear modulus of the girders was about 106 ksi.

Loading

The load case considered was a three-axle dump truck with a gross vehicle weight of 55400 pounds. The wheel load in the front and the two rear axles was 16,860 pounds and 19,270 pounds respectively. To obtain maximum deflection caused by the truck, the centerline of the three axles of the vehicle was placed to coincide with the transverse centerline of the bridge. The longitudinal centerline of the truck was also placed to coincide with the longitudinal centerline of the bridge. The experimental deflections were measured at the midspan of each girder. The plan layout along with the loading case considered is shown in Fig. 9.

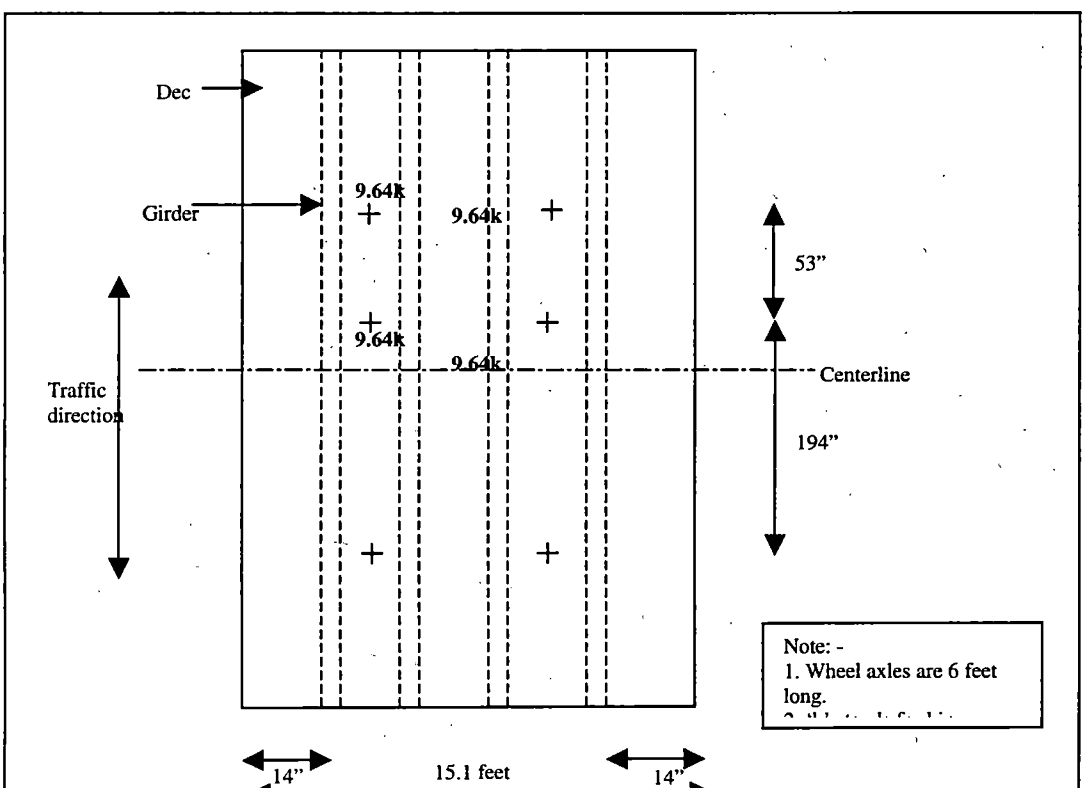


Fig. 9 – Plan layout of the Tuscaloosa Bridge, Alabama

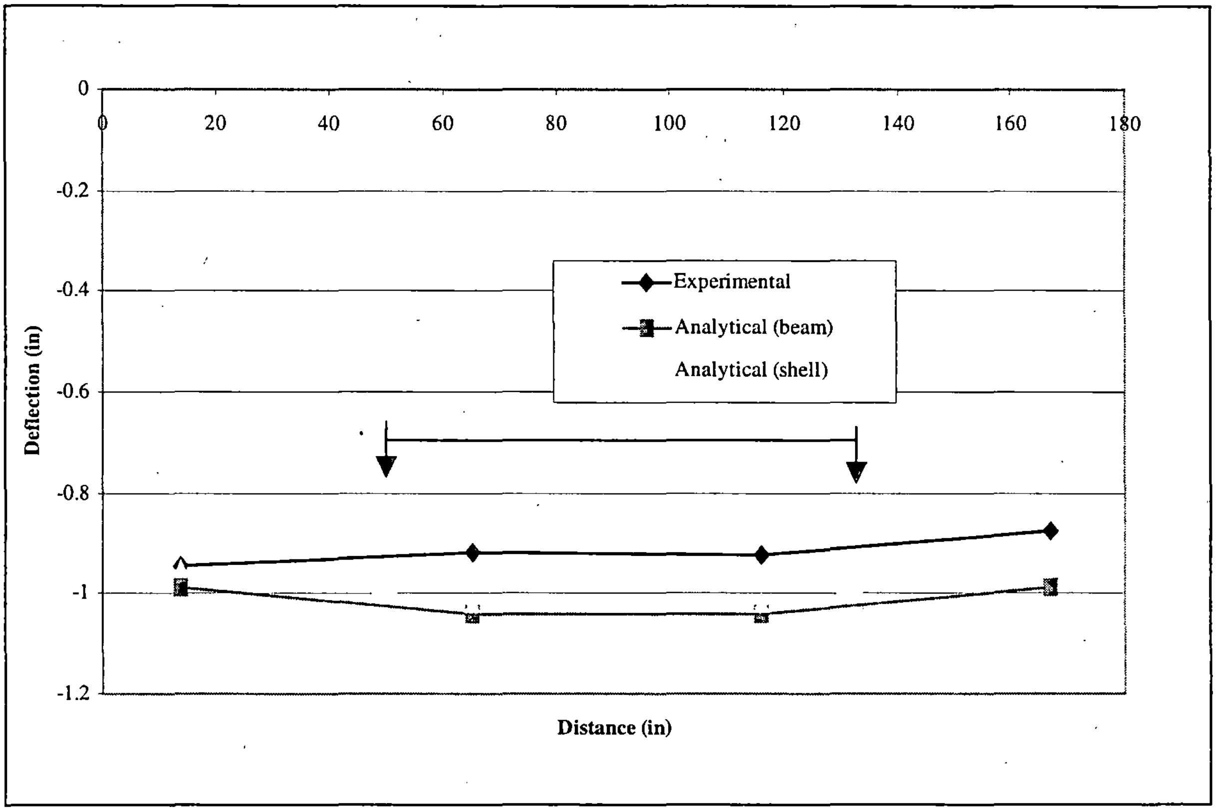


Fig. 10 – Panel deflections at midspan of girders – Tuscaloosa Bridge

Results and Discussion

A comparison between experimental data and analytical results for the Tuscaloosa Bridge is shown in Fig. 10. The graph shows the midspan deflection at each girder. The analytical results showed good correlation with the experimental values. The deflections of the girders based on the analytical model are about 12% larger than the experimentally measured deflections. The difference can be attributed to assumed material properties.

Model 1 and Model 2 compare very well with each other. As observed for the Cow Gulch Bridge, the deflections obtained from Model 2 are slightly lesser than the deflections from Model 1. This is attributed to the better simulation of composite action between the deck and the girder in Model 2 than in Model 1. The problem of location of supports did not seem to affect Model 1 and hence, it can be safely assumed that Model 1 is a very good approximation of the glued laminated girder bridge.

A mesh sensitivity analysis on the analytical bridge showed that differenced in deflection and bending stresses in the girders were smaller than 3% when the mesh size was 17 inches by 17 inches. The results discussed in the preceding paragraphs were obtained with a mesh size of 17 inches by 17 inches.

SUMMARY AND CONCLUSIONS

Two analytical models have been developed to analyze glued laminated girder bridges. Both models were validated by comparing the analytical results with experimental data from field bridges. The maximum difference between analytical deflections and experimental deflections was about 12% and this can be attributed to the assumed material properties in the analytical model. The following conclusions were drawn from the comparison of the analytical models to the field bridges:

1. The analytical models developed accurately predict the behavior of the girder bridge.

2. Experimental results support the conservatism of the analytical model.

3. Since Model 2 represents the real situation better than Model 1, the author recommends Model 2 to be the best idealization of a glued laminated girder bridge. Model 1, however, can be used as a good approximation to Model 2.

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LTPP-DISTRESS DUE TO ENVIRONMENT

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ABSTRACT

The paper titled 'LTPP - Distress due to Environment' presents the deterioration and distress in the pavement due to environment. Long Term Pavement Performance (LTPP) deals with various experiments in General Pavement Studies (GPS) and Specific Pavement Studies (SPS). The environmental effects are dealt in SPS8 experiments. Data Pave is the software available to extract the history data from the LTPP database for the experiments. The paper tries to analyze the pavements subjected to different environmental conditions. Data is extracted from the database for different states whose environments are significantly different. The International Roughness Index (IRI) values for the experimental sections in the different states are collected and analyzed in relation with the temperature. The linear regression on this data reveals the effect on the pavements due to the change in the temperature. The performance of the pavement is predicted based on the results obtained. The results can be implemented in the design of new pavements.

INTRODUCTION

Long Term Pavement Performance program is initiated as part of Strategic Highway Research Program (SHRP) in 1987 and is monitored by the Federal Highway Administration (FHWA). As part of the program 2,500 asphalt and Portland cement concrete pavement test sections are monitored and tested through many experiments. The basic idea is to understand why some pavements perform better than the others do *(1)*. As part of the program data relating to elements that may influence pavement performance is collected. Data for IRI, pavement thickness, annual and monthly precipitation totals and equivalent single-axle loads (ESALs) is collected in the past decade on the experimental sections and stored in the LTPP database. The data will be collected for the 20 year span and the database will be updated to help the highway engineers to get the information in ready to use format. The experiments and their results help the highway engineer to understand the pavement performance in the long run and design the new highways to perform effectively and for longer life periods.

The LTPP program essentially is initiated to effectively use the resources with in the available amount of funds. This study is going to become the primary source of performance indicator for the North American Highway Community. The data for LTPP sections is collected extensively for the inventory, material testing, pavement performance monitoring, climatic, traffic, maintenance, rehabilitation, and seasonal testing modules. The data is managed through an Information management System (IMS) called the LTPP database, which is the world's largest pavement performance database *(3)*.

LITERATURE REVIEW

As part of the LTPP program the entire US is divided into four regions, North Central, Western, Southern and North Atlantic. The regions are shown in the *Figure 1.*

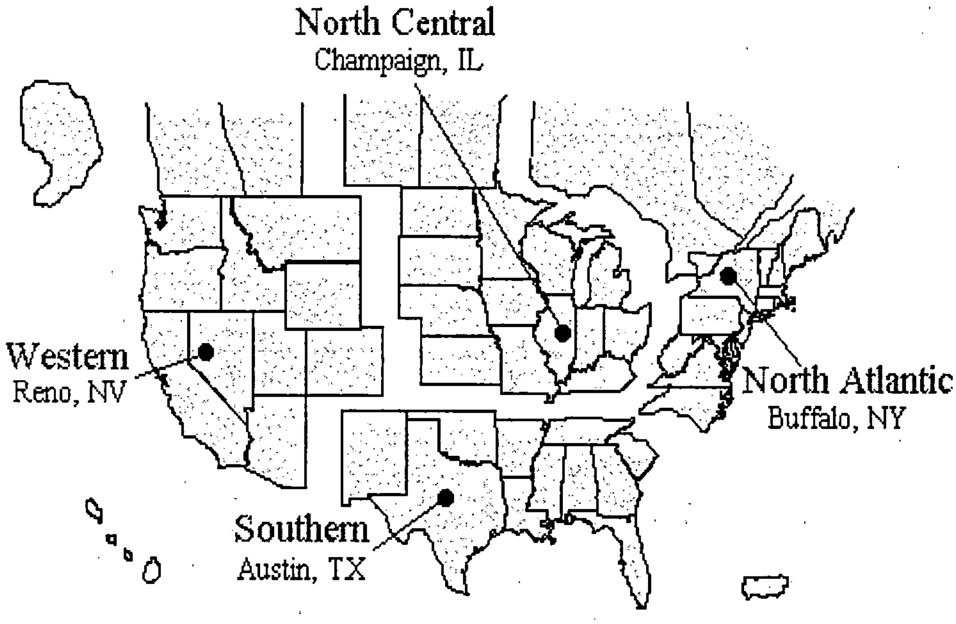

FIGURE 1 Regions and Monitoring Locations for LTPP *(2).*

Each region is having a monitoring center to monitor the experiments performed on the sections with in each zone. There are two types of studies performed as part of LTPP, General Pavement Studies (GPS) and Specific Pavement Studies (SPS).

The GPS experiments focused primarily on commonly used structural designs for pavements. Eight types of existing in-service pavements — in either original or rehabilitated condition — are being monitored throughout North America. The performance levels of structural designs are tested against an array of climatic, geologic, maintenance, rehabilitation, traffic, and other service conditions. Each GPS site has a single test section *(4).*

SPS test sections are specially constructed to investigate certain pavement engineering factors. Critical design factors are controlled by special construction and performance to be monitored from the initial date of construction. Each test site has multiple test sections, each with a different set of design factors. This makes it possible to compare the performance of different design factors, both within and between sites. The results will provide a better understanding of how selected maintenance, rehabilitation, and design factors affect pavement performance *(4).*

As of October 1999 there were 2500 asphalt and Portland cement concrete pavement test sections throughout the US and Canada. Of these sections 791 test sections are in GPS category and the remaining 1,714 are in the SPS category. The SPS test sections are divided into 9 sub categories, the description of each and the participating states are described in Table 1. *(4,6).*

ENVIRONMENTAL EFFECTS

The paper mainly concentrates on SPS-8 sections, which describe the 'Study of Environmental Effects in the Absence of Heavy Loads'. The effects of climatic factors, sub grade type on pavement sections having different designs of flexible and rigid pavements, under limited traffic are studied as part of SPS8 experiments. The test sections may be at the same or different locations. Currently there are 16 SPS8 projects. The International Roughness Index (IRI) values are affected by the change in the weather conditions like the average annual temperature, number of high temperature days.

DATA PAVE SOFTWARE AND DATA DESCRIPTION

DataPave is new easy to use software to retrieve and present data from a CD-ROM. DataPave is one of the software to explore the LTPP data. This software is chosen for the data retrieval, for the current paper from the DataPave CD-ROM. The main objective of the DataPave software is to provide a user-friendly format for exploring and presenting the LTPP data to provide easy-to-use presentation techniques which present the value of LTPP data *(3,6).*

DataPave provides the utilities to select the LTPP sections from the database through a Geographical Information System (GIS) based map or through specification of the criteria. After this step section specific information like the location and experiment type is provided through the Presentation module. Individual experimental data can then be extracted and exported to convenient formats like Excel-97 or delimited text formats. DataPave also has a powerful graphics engine to present data in graphs. In the future everyone in the field of design, construction and maintenance is to benefit through the products and research made possible by the DataPave software.

TABLE 1 Description of SPS Experimental Sections and the Participating States ***(4,5)***

SPS No.	Description & Participating States
SPS 1	**Strategic Study of Structural Factors for Flexible Pavements:** Alabama, Arizona, Arkansas, Delaware, Florida, Iowa, Kansas, Louisiana, Michigan, Montana, Nebraska, Nevada, New Mexico, Ohio, Oklahoma, Texas, Virginia, and Wisconsin.
SPS 2	**Strategic Study Of Structural Factors For Rigid Pavements :** Arizona, Arkansas, California, Colorado, Delaware, Iowa, Kansas, Michigan, Nevada, North Carolina, North Dakota, Ohio, Washington, And Wisconsin.
SPS 3	**Preventive Maintenance Effectiveness Of Flexible Pavements:** Alabama, Arizona, Arkansas, California, Colorado, Florida, Idaho, Illinois, Indiana, Iowa, Kansas, Kentucky, Manitoba, Maryland, Michigan, Minnesota, Mississippi, Missouri, Montana, Nebraska, Nevada, New York, Oklahoma, Ontario, Pennsylvania, Quebec, Saskatchewan, Tennessee, Texas, Utah, Virginia, Washington, And Wyoming.
SPS 4	**Preventive Maintenance Effectiveness of Rigid Pavements:** Arizona, Arkansas, California, Colorado, Illinois, Indiana, Iowa, Kansas, Kentucky, Minnesota, Mississippi, Missouri, Nebraska, Nevada, Ohio, Oklahoma, Pennsylvania, Quebec, South Dakota, Texas, and Utah.
SPS 5	**Rehabilitation of Asphalt Concrete Pavements:** Alabama, Alberta, Arizona, California, Colorado, Florida, Georgia, Maine, Manitoba, Maryland, Minnesota, Mississippi, Missouri, Montana, New Jersey, New Mexico, Oklahoma, and Texas.
SPS 6	**Rehabilitation of Jointed Portland Cement Concrete Pavements:** Alabama, Arizona, Arkansas, California, Illinois, Indiana, Iowa, Michigan, Missouri, Oklahoma, Pennsylvania, South Dakota, and Tennessee.
SPS 7	**Bonded Portland Cement Concrete Overlays on Concrete Pavements:** Iowa, Louisiana, Minnesota, and Missouri.
SPS 8	**Study of Environmental Effects in the Absence of Heavy Loads:** Arkansas, California, Colorado, Mississippi, Missouri, Montana, New Jersey, New Mexico, New York, North Carolina, Ohio, South Dakota, Texas, Utah, Washington, and Wisconsin.
SPS 9	**Validation of SHRP Asphalt Specification and Mix Design (Superpave):** Alberta, Arizona, Arkansas, Connecticut, Florida, Indiana, Kansas, Maryland, Michigan, Minnesota, Mississippi, Missouri, Montana, Nebraska, New Jersey, New Mexico, North Carolina, Ohio, Ontario, Quebec, Saskatchewan, Texas, and Wisconsin.

DATA EXTRACTION

The data is extracted form the database supplied along with the DataPave software. The first step is to select the region in consideration. The north central and southern LTPP regions are selected for the case study. LTPP experiments are sub grouped into climate, general, inventory, monitoring, maintenance, test sections, SPS sections, and traffic. Data from the climate, inventory, monitoring and SPS8 sections is extracted and exported to Excel 97 format. The required parameters like average annual temperature, IRI average value, number of days the temperature is above 32 degrees centigrade is extracted from Excel file and is presented here in Table 2. The data is collected for the states of Arkansas, Ohio, Texas, South Dakota, New Mexico, and Mississippi.

DATA

The data collected for the analysis is presented in Tables 2 & 3. Metadata is provided in Table 4.

TABLE 2 Experimental Data

STATE_CODE	SHRP_ID	AWS_ID	YEAR	AVG_ANN_TEMP	PAVE_TYPE	ABOVE_32	IRI_AVG
5	0803	050803	1998	17.00	1	64	1.14
5	0804	050804	1998	17.00	1	64	1.39
5	0809	050809	1998	17.00	0	64	1.70
5	0810	050810	1998	17.00	0	64	1.71
28	0805	280805	1997	16.50	1	52	0.97
28	0805	280805	1999	17.00	1	52	1.45
28	0806	280806	1997	16.50	1	52	0.82
28	0806	280806	1999	17.00	1	52	0.98
35	0801	350801	1997	14.64	1	83	1.06
35	0802	350802	1997	14.64	1	83	0.91
39	0803	390803	1994	10.40	1	10	1.23
39	0803	390803	1996	10.80	1	10	3.19
39	0803	390803	1997	9.48	1	10	1.15
39	0803	390803	1998	11.51	1	10	1.24
39	0804	390804	1994	10.40	1	10	1.18
39	0804	390804	1996	10.80	1	10	2.68
39	0804	390804	1997	9.48	1	10	0.91
39	0804	390804	1998	11.51	1	10	0.97
39	0809	390809	1994	10.40	1	10	1.93
39	0809	390809	1996	10.80	1	10	1.92
39	0809	390809	1997	9.48	1	10	2.02
39	0809	390809	1998	11.51	1	10	2.03
39	0810	390810	1994	10.40	0	10	1.63
39	0810	390810	1996	10.80	0	10	1.76
39	0810	390810	1997	9.48	0	10	1.79

TABLE 3 Experimental Data

STATE_CODE	SHRP_ID	AWS_ID	YEAR	AVG_ANN_TEMP	PAVE_TYPE	ABOVE_32	IRI_AVG
46	0803	460803	1993	5.5	1	25	0.81
46	0803	460803	1994	6.1	1	25	0.73
46	0803	460803	1995	6.2	1	25	0.76
46	0803	460803	1996	4.7	1	25	0.77
46	0803	460803	1997	6.24	1	25	0.88
46	0803	460803	1998	7.83	1	25	0.92
46	0804	460804	1993	5.5	1	25	0.82
46	0804	460804	1994	6.1	1	25	0.83
46	0804	460804	1995	6.2	1	25	0.82
46	0804	460804	1997	6.24	1	25	0.85
46	0804	460804	1998	7.83	1	25	0.89
48	0801	480801	1997	19.31	1	99	0.77
48	0802	480802	1997	19.31	1	99	1.05

TABLE 4 Metadata for Terms in Experimental Data

Term	Description & Units
STATE_CODE	Unique code identifying the state in which the section is
SHRP_ID	Unique code for the test section
AWS_ID	Code for the Automated Weather Station, combination of STATE_CODE
YEAR	Year of IRI data collection
AVG_ANN_TEMP	Average annual temperature in degrees centigrade
PAVE_TYPE	Pavement type in numerals. Asphalt=1and PCC=17
ABOVE_32	Number of days the temperature is above 32 degrees
IRI_AVG	IRI average values in mm/km

ANALYSIS & RESULTS

The effect of temperature and type of pavement on the IRI values for the test sections is analyzed through regression analysis. Multiple regression analysis of IRI is performed and the results are tabulated below.

TABLE 5 Results

Regression Statistics	
Multiple R	0.648
R Square	0.420
Adjusted R Square	0.369
Standard Error	0.453
Observations	38

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	1.029	0.324	3.179	0.00314
AVG_ANN_TEMP	0.096	0.0257	3.734	0.00069
PAVE_TYPE	-0.290	0.224	-1.294	0.20441
ABOVE_32	-0.017	0.0041	-4.322	0.00013

The best-fit lines equation for the multiple regression is given by

IRI = Intercept + β_1(AVG_ANN_TEMP)+ β_2 (PAVE_TYPE)+ β_3 (ABOVE_32)

IRI = 1.029 + 0.096(AVG_ANN_TEMP) - 0.29(PAVE_TYPE) - 0.017(ABOVE 32 C)

CONCLUSIONS

- The regression analysis resulted in the equation
- **IRI = 1.029 + 0.096(AVG_ANN_TEMP) - 0.29(PAVE_TYPE) - 0.017(ABOVE 32 C)**
- IRI values vary significantly with average annual temperature, pavement type and number of days the temperature is above 32 degrees centigrade in a year.
- Increase in average annual temperature increases the IRI values.
- Increase in number of days with temperature above 32 degrees centigrade decreases IRI values.
- IRI value is more for Asphalt pavements.
- States with high annual average temperature can design the pavements as flexible to reduce the IRI value. Example Texas
- States with low annual average temperature can design the pavements as rigid to reduce the IRI value. Example Wisconsin
- Further analysis of the LTPP data should be done to determine the accuracy of the current predicted model
-

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THE RELATIONSHIP BETWEEN URBAN DENSITY AND ROADWAY MAINTENANCE COSTS

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ABSTRACT

Urban land-use planning in the U.S. has traditionally been based on the Euclidean zoning model, the separation of land uses. The long-term impacts of this development pattern, often termed "urban sprawl", have been a continued source of controversy (1). This research determines if expert opinion regarding sprawl's negative effects, specifically, that regarding the public/ private capital and operating costs of communities, is applicable to the state of Iowa. These costs are defined as expenditures related to the construction of physical facilities in addition to their annual maintenance. This analysis investigates the relationship between transportation maintenance costs and population density; specifically, are the operating costs (the maintenance costs of streets) of local governments lower, if population density is higher? The study also employs a new measure of density, based on area proximate to streets and highways, developed using a geographic information system. The research indicates population density is not strongly linked to the costs of street maintenance; however, there is relatively little variation in density among Iowa communities. The statistical analyses demonstrate the street maintenance costs do not vary inversely with population density in Central Iowa, as one might expect.

INTRODUCTION

Although urban planning existed prior to the 1920's, the Standard Zoning Enabling Act (1922), the Standard City Planning Enabling Act (1928) and the legalization of zoning resulting from the 1926 *Euclid v. Amber Realty* Supreme Court decision facilitated its acceptance as American public policy. Consequently, the Euclidean zoning model of segregated land uses became the basis for land-use planning. In this model, residential land uses are considered the most sensitive to externalities; therefore, other land uses are excluded. Thus, the first suburbs were created to separate residential development from industrial and commercial sites to promote the public health, safety and welfare.

Urban Sprawl

During the post World War II era, rapid suburbanization occurred with the aid of federal policies. Post WW II suburban growth was a response to a number of social, economic, demographic, and technological factors. These factors are the postwar population boom, the increased availability of suburban housing, buyer preference for suburban sites, technological advancements, and the greater use of passenger cars. Federal housing policies contributed to suburban growth as the availability of federally insured low-cost mortgages facilitated suburban homeownership. Federal highway spending, particularly the 1956 Interstate Highway Act, financed the expansion of highways that gave consumers access to these suburban locations (2). Relatively few questioned this new configuration of settlement on the city edge or noticed its impact on urban centers (1). In this manner, low-density suburban residential development became part of the American urban landscape, subsequently becoming the most common form of "urban sprawl."

Other examples include any single use development excluding an attractive and functional mix of land uses i.e., industrial and commercial ribbon or strip, scattered, or leapfrog development (3).

Response to Sprawl

During the expanding economy era commencing in the 1960's, virtually all growth was considered inherently good. Concerns for the efficiency and costs of service allocation received less attention than their importance deserved. The rising costs of services, shortages of financing for capital improvements, and escalated costs of basic energy supplies have reversed these trends. Currently, the causes and effects of urban sprawl are being discussed, far more extensively than in the past.

Growth management is a range of policies designed to control, guide, or mitigate the effects of growth. These policies have become increasingly popular as a response to the ineffectiveness of local land use controls, one of the alleged causes of sprawl (3). Motivations for growth management are based on the principles of sustainability, energy and resource conservation, environmentalism, intergovernmental cooperation, coordination of planning with infrastructure provision, and to control the fiscal, functional, and aesthetic impacts of sprawl (3). Similarly, compact mixed-use developments have been proposed as alternative suburban designs to alleviate the problems of automobile dependence, growing vehicle miles traveled (VMT), and regional traffic congestion. These designs are referred to as neo-traditional communities, urban villages, pedestrian pockets, and transit-oriented developments. They share the characteristics of increased housing density, alternative travel modes, and a mix of commercial and residential uses (4). Their goal, as well as that of growth management, is a more compact and efficient urban form.

Iowa Context

As a follow-up to a 1976 study, the Institute for Design Research and Outreach (IDRO) at Iowa State University completed a survey (March 3, 2000) of local land use planning by Iowa's county and municipal governments (5, 6). The major findings of the census of land-use planning report, as stated by its authors in the executive summary, are as follows:

- The 1977 report noted the legal issues concerning the 100 cities having zoning ordinances without comprehensive plans, since these communities do not comply with the Iowa Code. This code requires zoning to be based on land-use plans. By 1999, this number had increased to 239.

- Among the cities with comprehensive plans, planning as an essential component to land-use management has decreased. Forty percent of the cities indicated they had revised their zoning ordinances without regard to their long-range plans.

- Within the last decade, More than 120 cities that lacked comprehensive plans annexed adjacent land.

- In general, the more urban (highly populated) the area, the more likely a city or county will adopt land-use planning and regulation; however, over 40 percent of the cities within metropolitan counties still lack long-range plans.

- County governments have increased planning and land-use management; twenty-five additional counties adopted comprehensive plans between 1976 and 1999. However, like cities, counties also have discrepancies between land-use plans and zoning ordinances i.e., only 44 percent of counties base revisions of their zoning statutes upon guidance provided by the comprehensive plan.

- Adoption of sign regulations, flood-plain regulations, and zoning ordinances increased significantly among counties between 1976 and 1999. The number of counties regulating the subdivision of land also increased during this period.

In summary, the condition of planning among Iowa's municipalities has deteriorated in the 23-year interim between studies; conversely, the level of planning activity at the county level has improved dramatically. Iowa Counties employ professional planning staffs more often than cities, primarily in the form of Councils of Government (5).

THE PROBLEM

A problem with unplanned growth is its impact on infrastructure costs and provision. Often, growth occurs in areas where the extension of physical utilities is less suitable (other sites may be less costly due to proximity) than other potential locations. The results are increased user fees in an average cost pricing system since the costs of extension are distributed on a per capita basis. This is claimed to be a negative fiscal impact of sprawl development.

In The Costs of Sprawl-Revisited, Robert Burchell et. al. define the public capital and operating costs of sprawl as those costs related to the construction of roads, water and sewer infrastructure, and public buildings, in addition to the annual costs of maintenance for these physical facilities (1). Higher infrastructure costs is alleged to be a negative impact of sprawl. The reasoning is that at lower urban densities or with inefficient urban form linear infrastructure serves a smaller portion of development than it could at higher densities or more efficient urban configurations (1, 7). The objective of this study is to discover if the maintenance costs of streets is lower, if population density is higher. In brief, are the two inversely related?

THE APPROACH

The objective of this Study is to discover, through case studies, if the literature regarding sprawl's negative effects accurately reflects conditions in the state of Iowa. The research investigates the relationship between population density, urban form, and transportation maintenance costs. The outcome is intended to aid the evaluation of urban growth in the state and its fiscal effect(s) on local governments. A prominent inverse relationship between maintenance costs and population density was expected. This would have been reflected by a declining 45-degree angle trend line; however, what we discovered in Central Iowa is illustrated below in figure 1.

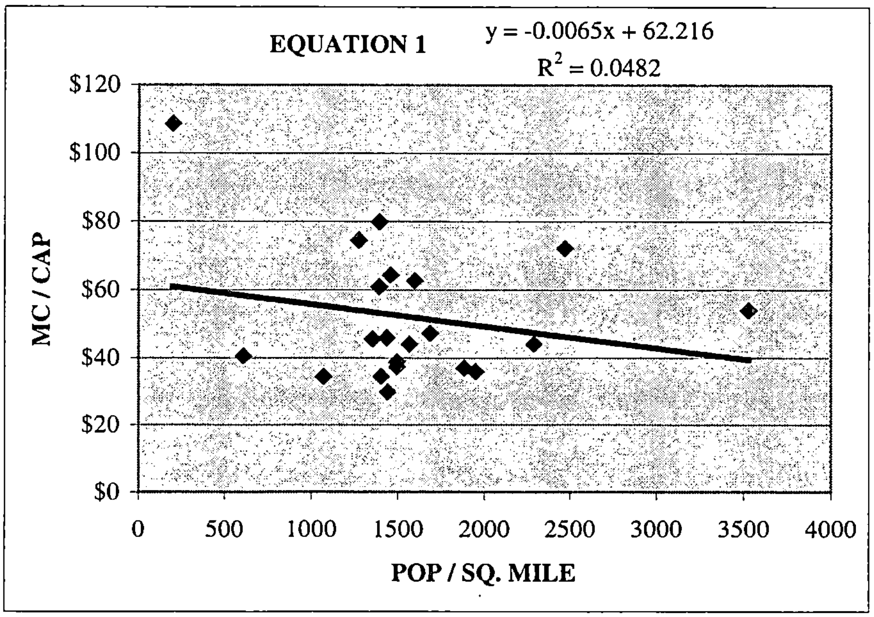

FIGURE 1: Maintenance cost and population density relationship in Central Iowa

The transportation maintenance expenditures of twenty-one selected cities within a nine-county region in central Iowa were studied (see figure 2 for the study area map). The study period is 1988-1998. Most of the selected cities were urban places (>5000 population) in 1998; however, in three cases, places under 5000 population were included. In Dallas and Madison Counties, the largest city is the county seat; therefore, Adel and Winterset have been included. The other exception is Pleasant Hill, which is included as part of the Contiguous Polk County metropolitan area. The selected cities and their respective 1998 populations are as follows:

Adel	3988
Altoona	9567
Ames	48415
Ankeny	25086
Boone	12754
Clive	11125
Des Moines	191293

Indianola	13023
Johnston	6906
Knoxville	8164
Marshalltown	25201
Nevada	6126
Newton	15371
Norwalk	6678

Pella	9525
Perry	7301
Pleasant Hill	4868
Urbandale	27907
West Des Moines	42333
Windsor Heights	4977
Winterset	4685

A longitudinal study of the twenty-one-selected cities was conducted to study the relationship between street maintenance costs, population density, and urban form over time. A categorical analysis of the twenty-one cities for a one-year period (1998) was performed to discover their individual compactness vs. cost relationships for maintenance costs per capita. This examination provides a context for evaluating the growth of Iowa cities and is intended to aid future research on Iowa land use planning.

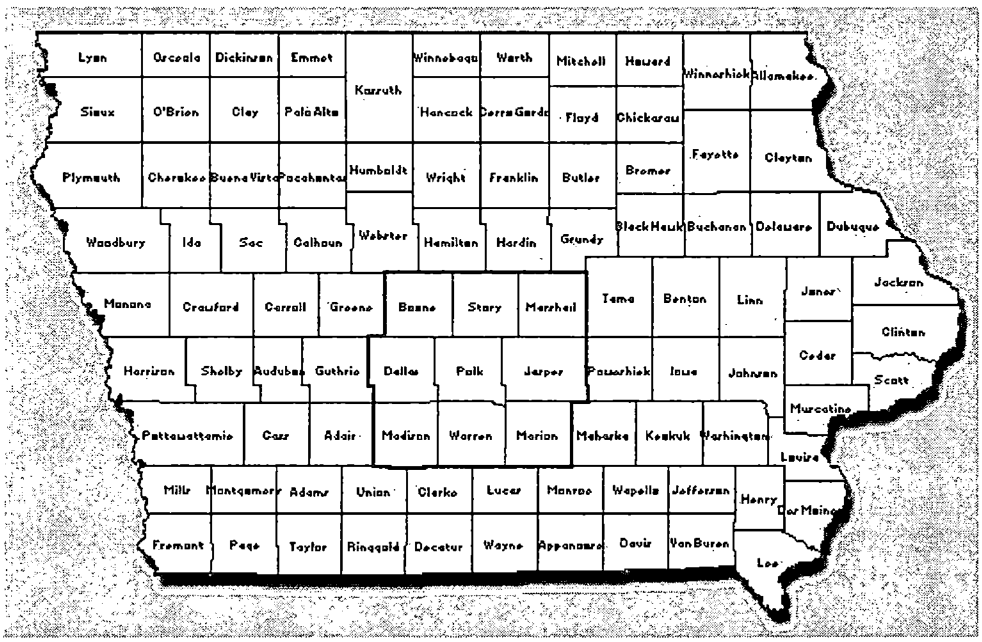

FIGURE 2: Nine-county study area in Central Iowa

Data

Data were collected on the population of cities, transportation maintenance expenditures for cities, and the number of lane miles for cities, from 1988-1998 (even years). The limitation to this decade reflects the availability of the cost data, Iowa DOT Street Finance Report RUT-2A:

Maintenance Costs for Street Purposes. These records are not available prior to 1988 or following 1998. The Iowa DOT also furnished the number of lane miles per city for the study period. Population estimates were obtained from the U.S. Census Bureau.

Standards of Measure

Data were analyzed to determine the rank of each city in maintenance costs per capita, and lane miles per capita, typically inversely related to population density. Low-density development typically requires a larger investment in transportation infrastructure. According to Burchell, the result should be higher maintenance costs. (1). The lane miles per capita measure is employed in this study. Other measures of population density utilized are the traditional population per square mile and a proposed measure, "Buffered Area Density". The new measure is insensitive to open space variability among cities and better reflects the actual housing density (lot sizes) in the study region. The shape index, a measure of city form efficiency, compares the city perimeter to its area. The most efficient form, that is, the one with the lowest ratio of area to perimeter, is the circle. For comparison, the circle has a shape index of 1; a square has a shape index of 1.13. For a given area, the more efficient the form, the less distance urban infrastructure is required to traverse, resulting in decreased costs. Differently stated, the shape index is a comparison of the measured perimeter to the minimum possible perimeter for the same area.

New Density Measure-Buffered Area Density

Street buffers were created in GIS Arc View version 3.2 based on the street networks obtained from the Iowa Department of Transportation. Four buffer sizes, 100 feet, 125 feet, 150 feet and 175 feet were applied to the street networks of nine sample cities and used as inputs to regression analysis. The 100-foot buffer produced the best results (R Square) and was subsequently applied to the remaining street networks. The goal of the buffering process was to capture all development adjacent to streets. According to the Ames City Assessor's Office, pre-1945 lots are approximately 1/8 acre or roughly 75-foot square in size. Most contemporary lots are between ¼ and ½ acre depending on land value, neighborhood, and city in Iowa. This equates to lot lines offset roughly 100-foot from the streets. Once the 100-foot buffers were constructed, the total buffer area for each city was calculated. The street area quantities were divided by the appropriate city population to determine population per buffered street area or buffered area density, the new density measure.

Data Manipulation

The Iowa DOT Street Finance Report RUT-2A contains municipal street maintenance cost data. These financial reports consist of five expenditure categories: Roadway Maintenance (the largest expense), Snow and Ice Removal, Storm Sewers, Traffic Services, and Street Cleaning. For this study, these expenses were combined to determine aggregate street maintenance costs per city on an annual basis for combination with lane mile and population figures. With these data, it was a simple matter to determine lane miles per capita (LMC) and maintenance costs per capita (MCC). For the study period 1988-1998, even years were analyzed to establish six data points for trend evaluation. Since RUT-2A cost data were unavailable for 1990, 1989 and 1991 were averaged on a categorical basis, and then totaled to determine aggregate 1990 costs. Population estimates for the off-census years were obtained from the Census Bureau. As the 2000 census data has yet to be released, the late 1990's estimates may be inaccurate.

1998 CATEGORICAL ANALYSIS RESULTS

Results are presented in two stages. First, the outcome of the 1998 categorical analysis is discussed. This discussion evaluates the utility of the three density measures, population per square mile, buffered area density, and lane miles per capita as well as the urban form efficiency measure (shape index) for the twenty-one selected cities. These results are then compared with maintenance costs per capita rankings for the same year (1998). Secondly, the longitudinal study of the selected cities compares maintenance costs with the city density and growth for the same period. The primary tool employed in this time series analysis is percent change for the study duration 1988-1998.

Population Per Square Mile

The categorical analysis did not reflect a strong relationship between maintenance costs and density; however, in individual examples, a link was observed using the population per square mile measure. The City of Johnston ranks as the least dense of the twenty-one-selected cities. It also has the highest expenditures in maintenance costs per capita. In this case, the results support the hypothesis that low-density development equates to higher maintenance costs. Refer to figure 3 for the results of the population per square mile density measure in comparison with other urban measures and figure 4 for maintenance costs per capita (MCC).

However, the most compact city (population per square mile), Windsor Heights, has higher per capita costs than half of all cities. If the hypothesis were correct, the order of cities in MCC would correspond inversely with that of the population per square mile density measure. In other words, the cities would appear in reverse order, the least dense being the most expensive. While this is true for the City of Johnston, the remaining cities do not follow this pattern. In general, the regression and plots indicate practically no relationship between population density and road maintenance cost.

CITY	POP/SQ. MI	POP/BUFAREA	LN MI / CAP	SHAPE INDEX
Adel	1276	7229	92.87	11.1619
Altoona	1394	5949	102.73	17.7423
Ames	2290	7957	124.54	34.5795
Ankeny	1497	5636	99.27	29.0736
Boone	1435	3739	63.70	22.9875
Clive	1689	5660	83.93	21.4416
Des Moines	2472	7925	97.68	71.6022
Indianola	1441	6134	100.92	20.4255
Johnston	202	6782	73.99	21.5133
Knoxville	1885	4278	77.89	15.7823
Marshalltown	1394	7501	74.68	30.8719
Nevada	1457	5717	72.43	14.8584
Newton	1498	5701	71.57	24.5267
Norwalk	1071	8784	107.69	14.3327
Pella	1403	6912	83.12	18.1862
Perry	1952	7408	72.81	15.4455
Pleasant Hill	611	7204	60.59	16.9495
Urbandale	1569	7978	91.50	30.2033
West Des Moines	1600	6308	99.50	36.0256
Windsor Heights	3529	5313	98.93	12.1312
Winterset	1353	4604	68.15	12.3625

FIGURE 3: Urban measure summary for selected cities

Buffered Area Density

The buffered area density measure reflects the predominant lot sizes of the city, including commercial, industrial and residential areas. For this study, these are treated in aggregate; moreover, no distinction is made between these three land-use types. The results indicate the City of Norwalk has the greatest concentration of population within the 100-foot buffered area. In the costs rankings, Norwalk has the second lowest maintenance costs per capita, which supports the hypothesis. The least dense city, Boone, is average in costs per capita. With the exception of Norwalk, the cities do not illustrate a logical pattern in their cost/ density relationship. Clearly population density, as measured by buffered area, has only a very weak relationship with maintenance costs.

Lane Miles Per Capita

Burchell contends that population density and lane miles are inversely related; as density decreases, lane miles increase, resulting in higher capital and maintenance costs for linear infrastructure. According to his thesis, cities with the lowest lane miles per capita values (higher densities) would have the lowest maintenance costs. This density measure does not support this conclusion. However, the city of Pleasant Hill has the least lane miles per capita yet roughly average maintenance costs per capita. Pleasant Hill should rank very near the bottom, assuming the Burchell hypothesis is correct. While the City of Johnston has the most expensive maintenance costs, it is above average in the lane miles per capita density measure. In other words, while it is more dense than average by this measure, it has the highest maintenance costs.

Similarly, Ames has the most lane miles per capita, equating to the least density; however, maintenance costs are average. Therefore, this density measure supplies little evidence supporting the hypothesis. Clearly, if there is a strong relationship between density and maintenance cost, other variables account for the cost fluctuation observed in the Iowa cities.

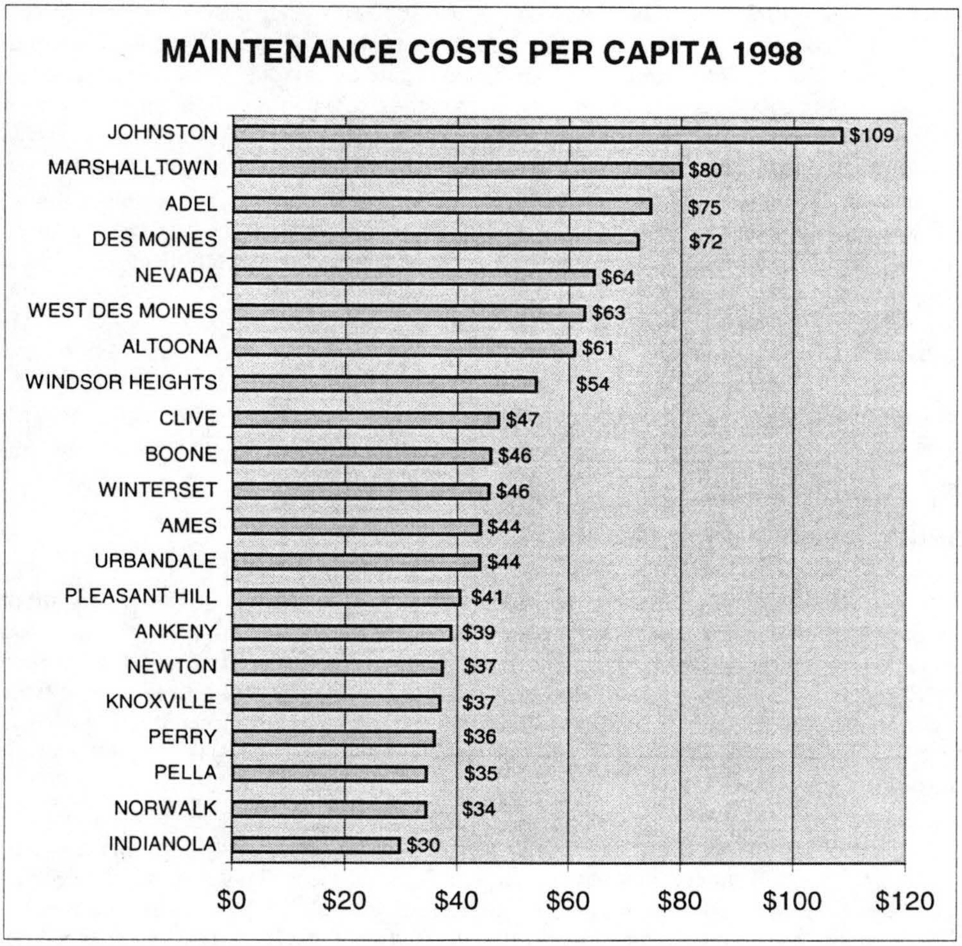

FIGURE 4: 1998 Maintenance costs per capita for selected cities

Shape Index

The measure of urban form efficiency, the shape index, is a calculation based on the measured perimeter to the minimum possible perimeter for the same area. It is independent of population. Ames is the least efficient by this measure, West Des Moines, the most. Des Moines is fourth most efficient. The City of Adel ranks third overall; Windsor Heights ranks second. In terms of urban efficiency, the Cities of West Des Moines, Windsor Heights, Adel and Des Moines should have the least expensive maintenance costs with Ames having the most. By referring to the maintenance costs chart, we see this is not the case. In MCC, Adel is third; Des Moines is fourth, West Des Moines is sixth and Ames is twelfth most expensive of the twenty-one total, all contrary to their urban configurations. The measure does not support the hypothesis. In fact, there appears to be a weak positive relationship between the shape index and street maintenance costs.

Regression Analysis

Initially, univariate regression was employed to determine the correlation (R square) between three dependent variables: maintenance cost per lane mile per capita, maintenance cost per lane mile, and maintenance cost per capita, with forty-eight independent variables in various combinations. Plots were created from the output of each of the forty-eight equations to view the relationships. Regression was then performed on nine sample cities: Adel, Ames, Ankeny, Boone, Johnston, Marshalltown, Newton, Norwalk, and Winterset. The goal-to determine which buffer size (100', 125', 150', and 175') was the most appropriate measure. Plots from the first regression were utilized to assist this selection. The first analysis is titled "regression," the second analysis, "regression 1". Regression 2 consisted of the remaining eleven cities omitted in the original sample. Regression 3 began the multivariate analyses and buffer size was restricted to buffer 100 as per results of the first three analyses. Regression 4 was multivariate, including the three dependent Y variables individually with all possible combinations of the independent X variables. Regression 5, a univariate, consisted of one Y variable, maintenance costs per capita, with four explanatory variables, the density and shape index calculations. Scatter plots were constructed for this final regression to determine the correlation of the density measures and form factor (shape index) with maintenance cost data. Hence, only maintenance cost per capita figures were used. At this point, a stepwise regression was conducted in statistical analysis software (SAS) to determine the optimum combination of the explanatory (X) variables with the response (Y) variables. The SAS procedure was labeled regression 6. The stepwise regression analysis, regression 7.xls, is a test of the stepwise regression completed in SAS. In regression 7, as well as regression 5, the model equation was utilized to estimate the dependent variable Y. In both cases, these estimates of response variables were compared with the actual maintenance costs on the "plots" pages. None of these seven regression analyses resulted in a model of predictive value. In this exercise, no evidence was discovered for the inverse relationship between maintenance costs and the urban measures employed. Although regressions 1, 3, 5, and 7 yield R squared results over 0.5, this phenomenon was caused by collinearity among the explanatory variables rather than a strong relationship per se. None of the density measures were strongly linked to maintenance costs in this study.

1988-1998 LONGITUDINAL STUDY RESULTS

A longitudinal study was conducted to compare the maintenance costs changes (adjusted for inflation) with those in two density measures, population per square mile and lane miles per capita. Physical growth during the period is expressed as changes in urban population and city area in square miles. Categorical shifts are presented as percent change for the study period 1988-1998. Since 1988 population is a census bureau estimate, the city area growth and percent change in population per square mile utilize 1990 census totals as a starting point to improve accuracy.

1990-1998 Population Per Square Mile

For the study period, the majority of cities decreased in population density. Only five of the twenty-one selected cities increased density by a substantial amount (see figure 5 for percent change in population per square mile). These cities, in descending order of percent change, are as follows: Adel, Clive, Indianola, Ankeny, and Boone. Adel, which did not annex land during the study period, increased its population density by 21%. Clive, Indianola, Ankeny, and Boone increased by 8%, 6%, 6%, and 2% respectively. The remaining 15 cities decreased in population density; Johnston had the greatest decline at 40 percent linked with the greatest expansion, 20.3 square miles area increase (refer to figure 6 for area growth during study period. Depending on the accuracy of the 1998 estimates, the trend is toward decreased density resulting in more land area consumed per capita.

1988-1998 Lane Miles Per Capita

The results of this density measure are contrary to the previous. The most notable difference is that of the City of Adel; while it increased in density according to the population per square mile measure (21%), it decreased a similar amount (20%) according to this density measure. As earlier noted, the lower the lane miles per capita, the more dense the city in comparison with others. Obviously, both measures cannot be true in their representation of population density. Second in percent increase (decreased density) for the period is Johnston (18%), which is plausible since it has substantially increased its city area for the duration. It is possible that Adel had a significant area of unimproved land contained within its urban area, and therefore, is now developing it to accommodate a 24% population increase for the decade. However, this does not explain the contradiction in density measures. In similar contrast, most selected cities decreased lane miles per capita for the study duration, the City of Norwalk having the greatest change (-33%). The trend, according to this density measure, is toward increased density. The contradiction in density measures is obvious.

1988-1998 Population Change

The majority of cities gained population during the period; only three cities lost residents, Windsor Heights -3.04% (-156 people), Des Moines -.76% (-1457 people), and Knoxville -.61% (-50 people). Johnston has the greatest percentage gain of new residents, 61% or 2621 people. Clive was the location of the second highest proportional growth, 55% or 3943 people. The percent change is expressed in relation to the 1988 city population; this can be misleading in terms of the actual numerical change, as the previous example indicates. The greatest numerical increase occurred in West Des Moines with 12593 new citizens. Ankeny and Urbandale follow with 7215 and 5533 new community members. Clive, Altoona and Johnston complete the top six in numerical growth. The top seven in percentage growth, in decreasing order of change, is as follows: Johnston, Clive, West Des Moines, Ankeny, Altoona, and Pleasant Hill. Population growth in the study area occurs most rapidly in the Des Moines metropolitan area. West Des Moines had the greatest numerical gain, as opposed to the greatest percentage gain of Johnston.

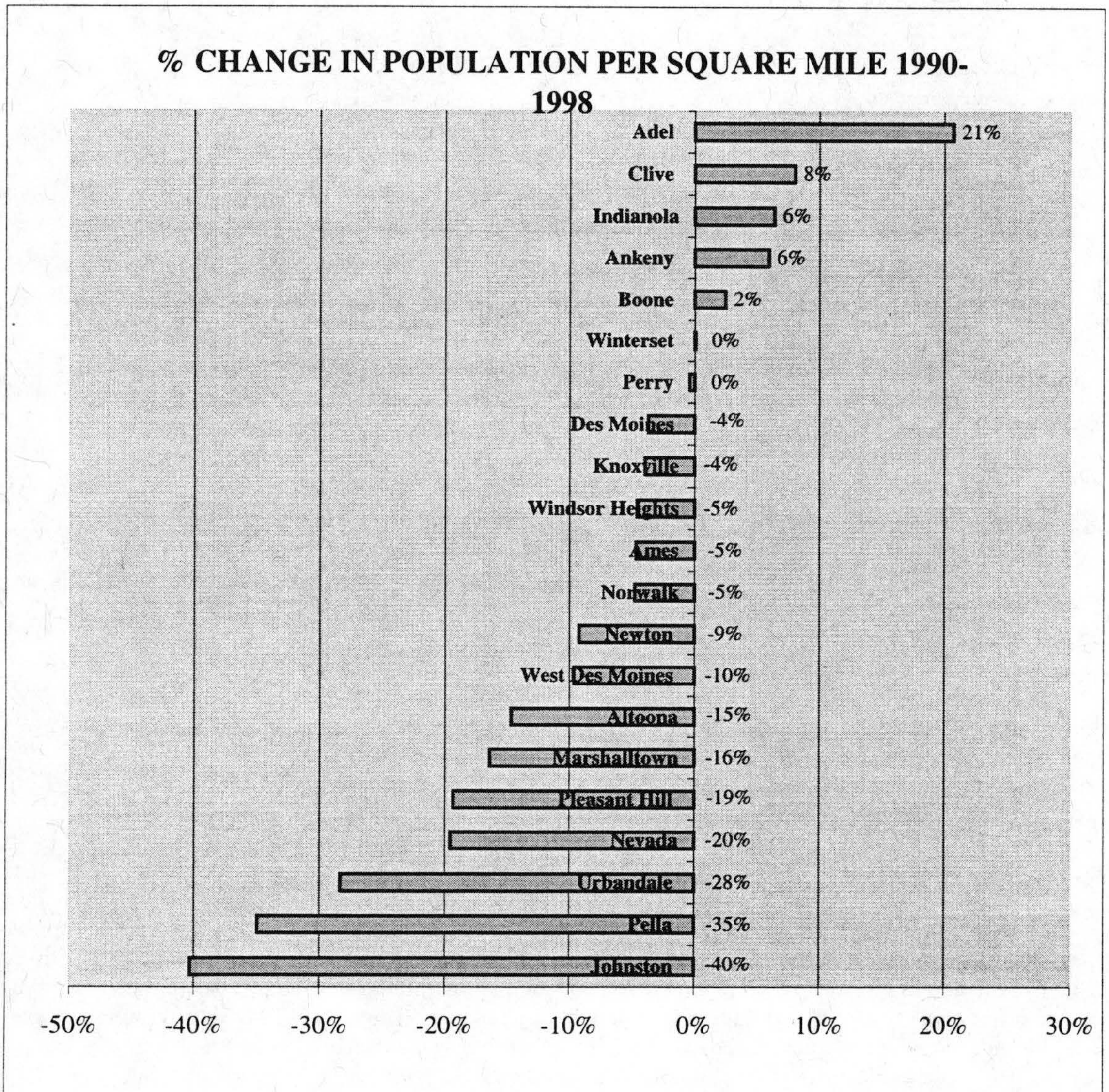

FIGURE 5: 1990-1998 percent change in population per square mile

1990-1998 City Area Growth

The cities with the greatest population increase are also the locations of the most active annexation. Johnston greatly exceeds the remaining selected cities in area growth with over 20 square miles. West Des Moines, Urbandale, Ankeny, and Pleasant Hill follow with 8.55, 7.06, 3.6, and 3.11 square miles respectively. It appears these cities are growing and decreasing in density. The top five cities in growth were again in the Des Moines Metropolitan area. Adel did not grow spatially; Knoxville, Boone, and Windsor Heights had nominal gains. The majority of cities grew; the average growth was 2.9 square miles with Johnston included in the calculation. Without Johnston, the average growth was 2.03 square miles. Most cities are growing at a moderate rate.

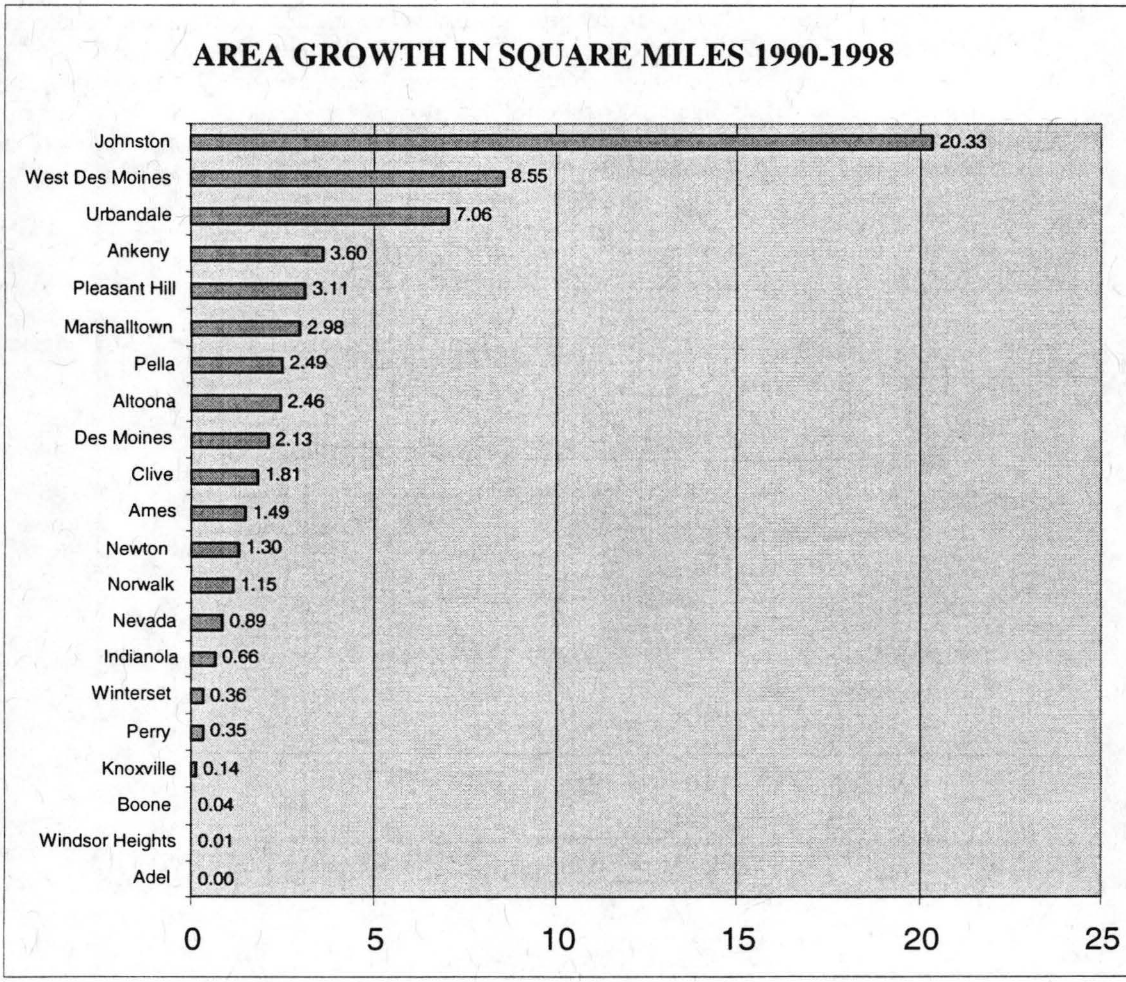

FIGURE 6: 1990-1998 city area growth in square miles

Lane Miles

The changes in total lane miles for the for the study period also provide evidence for the Des Moines Metropolitan area's high growth. The greatest numerical increase in lane miles occurred in West Des Moines (142) with Des Moines (115), Urbandale (102), Ankeny (94), and Clive (72) completing the top five. Expressed as percent change, the order converts to Clive, Norwalk, Pleasant Hill, Altoona and Ankeny. The City of Johnston is anomalous with only a 37% or 25-mile increase, despite its exceptional area growth. It is clear that most of Johnson's annexed area remains undeveloped. The largest growth in lane miles occurred in Des Moines and its suburbs although the City of Des Moines lost 1457 residents for the period. West Des Moines, Urbandale, Ankeny, Clive and Altoona had the greatest gains in population, which corresponds to lane mile increases. Collectively, cities gained lane miles and population simultaneously with the exception of Des Moines, the urban core.

1988-1998 Maintenance Costs

Percent change in aggregate maintenance costs for the period is led by the City of Pleasant Hill (89%) with West Des Moines (81%), Johnston (70%), Norwalk (53%), and Altoona (52%) in the top five. Windsor Heights, Clive, and Pella closely follow with 50%, 49%, and 47%. Adel (-46%), Knoxville (-21%), and Ames (-12%) had the only decreased costs for the period. These three cities also had the greatest decrease in per capita costs expressed as percent change in figure 7. This pattern of the decrease in maintenance costs for three cities also applies to the three of greatest increase. Pleasant Hill, Johnston, and West Des Moines had the greatest increases in terms of total maintenance costs and maintenance costs per capita.

DISCUSSION

It is plausible to connect the greatest gains in lane miles, as infrastructure investment, with area growth, and population increases. In other words, these municipalities have the most rapid growth. Johnston, West Des Moines, Urbandale, Ankeny, Clive, and Pleasant Hill are examples of cities with high growth for the period. The City of Windsor Heights, however, is unique in this respect. Although it had large increases in maintenance costs for the period, its increase in lane miles was disproportionately low and the city actually lost population for the period. Environmental factors may account for some of this anomaly although this increased maintenance cost due to snow and ice removal would likely appear across the board.

There are many other plausible explanations. While winter maintenance and roadway repair standards exist at the state level regarding primary highways; a similar municipal counterpart is lacking. Therefore, it is possible that Windsor Heights has a more aggressive winter maintenance policy; for example, it starts plowing before much snow accumulates as compared to other cities. Windsor Heights also may have a more aggressive level of roadway maintenance; i.e., it repairs the streets more quickly before they become as distressed (as other cities). Another explanation is the age of the streets; older streets obviously require more maintenance. Therefore, older cities or cities with a higher level of deferred maintenance could have higher costs during the study period. This statement implies that fluctuations in city budgets are a factor in the variability of costs. The less money allocated annually for maintenance, the fewer repairs are completed. While collectively the study cities are decreasing population density, this can be partly attributed to consumer preference for large suburban lots.

With this view, increasing income or prosperity coupled with individual choices has an effect on street maintenance costs. The majority of selected cities are increasing in size or experiencing population growth, some just more rapidly. This growth affects maintenance costs by increasing traffic flows on municipal streets. Higher levels of traffic, and more importantly, the traffic composition (i.e. the number of trucks hauling material to support new infrastructure construction) have a large impact on street degradation. Maintenance costs are composed of five categories: roadway maintenance, snow and ice removal, storm sewers, traffic services, and street cleaning. New construction costs should not appear in these categories. Gains in lane miles, as new construction, would not have high road maintenance (the largest expense), storm sewer, or traffic services costs. Accounting practices can also explain some variability in maintenance costs. It is possible that costs are being misallocated to stretch the budget priorities. Street cleaning appears to be done as the funding allows; some cities had no entries in this budget category. Any one or combination of, the above possibilities can explain the variability in maintenance costs for the period.

Density Measures

As the results indicate, the lane miles per capita measure contrasts with the population per square mile criterion. The selected cities cannot be increasing and decreasing aggregate density during the same period. Of the two measures, the results of the population per square mile standard are favored. The lane miles per capita density measure appears flawed. The proposed density measure, buffered area density, has results somewhat similar to the population per square mile measure. In fact, it should be different since it excludes open space in the calculation. However, since density has been found to have little correlation with maintenance costs in Iowa, it is difficult to predict the accuracy of this new measure. In summary, none are a good predictor of maintenance costs in central Iowa cities.

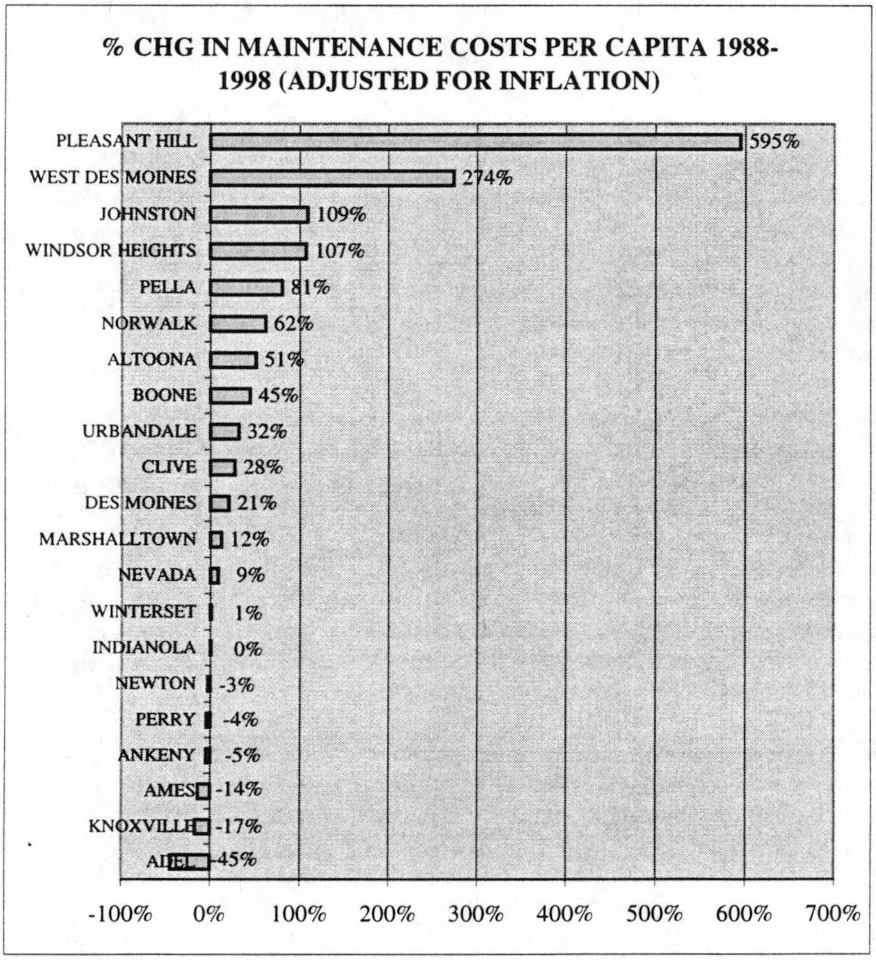

FIGURE 7: 1988-1998 Percent Change In Maintenance Costs Per Capita

Density And Maintenance Costs

The range of development densities in Iowa is not great. When compared with other national and international cities, Des Moines, the Iowa capital, is less dense than all others. A comparison of 1980 populations per square mile between national and international cites (8) follows:

- Boston 3127
- Chicago 4526
- Copenhagen 7881
- Des Moines 2190
- Detroit 3650
- Houston 2300
- Los Angeles 5188
- Melbourne 4269
- New York 5559
- Phoenix 2199
- San Francisco 4011
- Sydney 4547
- Toronto 10257
- Washington 3425

For longitudinal comparison, Des Moines' population density increased from 2190 to 2567 in 1990, and was estimated at 2472 population per square mile in 1998. Furthermore, all cities analyzed in Central Iowa have a rather similar density profile and layout (shape index). Therefore, there is little variation in density to help explain cost fluctuation.

In Iowa, the operating costs of local governments, as expressed in street maintenance costs, are NOT lower when population density is higher. An inverse relationship between the two does not exist. After review, numerous factors other than density and urban form play a major role in the variation of maintenance costs among cities. These factors include local winter and roadway maintenance policies, annual fluctuations in city budgets, individual accounting practices, consumer affluence and preferences, and traffic levels/ composition.

Much of the variation in simple population density among Iowa communities was eliminated with the buffer approach. This indicates that some of the cities in Iowa have large extents of low-density land uses such as parks, golf courses, cemeteries, soccer/ baseball/ football fields and green belts. These land uses are amenities that contribute little to the costs of maintaining a road network; however, they do significantly add to community quality of life.

Policy Implications

Operating costs vary for a variety of reasons, including local preferences and municipal policies. Such variation is not captured in this analysis. In the sort of city configurations common to Central Iowa, density does NOT appear to influence costs in a significant manner. Community leaders, land use planners, and transportation planners in Iowa should not be overly focused on population and development density as levers for influencing transportation costs. Much higher densities of development (as in older US major metro areas or European "walking cities") would be needed to realize the economies of density that authors such as Burchell have previously found. Such high densities would be very difficult to develop in the majority of Iowa cities given their current configurations, and transportation systems, which are heavily auto-oriented.

Recommendations For Future Study

Whereas this study explored the connection between urban density and operational cost efficiency, capital expenditures (construction costs) could reveal a better density link. An approach of this nature, including better cost data, is recommended. If possible, population estimates should be avoided. However, since such few new road miles are being constructed (as compared to total lane miles) this may also reveal a weak relationship.

ACKNOWLEDGEMENTS

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GEOGRAPHIC INFORMATION SYSTEMS-BASED CRASH DATA ANALYSIS AND THE BENEFITS TO TRAFFIC SAFETY

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ABSTRACT

Geographic Information Systems (GIS) have been used in the past to display crash locations and produce maps. Today, the potential uses of GIS include crash data analysis. GIS-based crash data analysis can influence the four E's of traffic safety: engineering, enforcement, education, and emergency response.

Macroscopic applications of GIS provide the ability to analyze a large amount of data quickly. Macroscopic analysis can be used on large regions to identify areas of concern without getting into specifics. The greatest benefit of GIS-based crash data analysis is the microscopic applications that can be done to evaluate crashes in a selected region. Various queries can be performed on isolated groups of data.

By using GIS, the time and effort required to analyze crash data can be reduced. At the same time, an increasing number of scenarios and alternatives previously not possible can be evaluated. A literature review showcases potential applications of GIS-based crash data analysis as a tool to assist engineers, administration, policy makers, law enforcement, and emergency personnel make informed decisions on traffic safety issues.

INTRODUCTION

A geographic information system (GIS) is a computerized database management system that can capture, store, retrieve, assemble, manipulate, and display geographically referenced information. It is comprised of visual and tabular data in a format that allows the user to process large amounts of information quickly. GIS has been successfully applied to many fields outside the transportation realm.

In the field of transportation, GIS has been used to graphically display crash locations and produce maps. With technological advances in personal computers and GIS desktop software, GIS can now be used to perform crash data analysis. By using GIS, the time and effort required to analyze data can be reduced. At the same time, an increasing number of scenarios and alternatives previously not possible can be evaluated. With the use of GIS, crash data analysis can be applied to improving traffic safety. Tom Welch, chair of the Iowa Safety Management System Coordination Committee (SMSCC), summarized the benefits of GIS in traffic safety in three words: "Innovation, creativity, and flexibility *(1)*."

BACKGROUND

The need for highway safety information was recognized and data began to be collected after the Highway Safety Act of 1966 and the Highway Safety Program Standard 10 of 1967 (the Standard) were passed. The Standard mandated that state systems include data for the entire state and that information regarding drivers, vehicles, crashes, and roadways be compatible for analysis *(2)*. An increased emphasis on transportation safety also occurred with the enactment of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA). This act established safety management systems for each state to perform a systematic analysis of highway safety *(3)*. Crash data collection for analysis continued with the implementation of the Transportation Equity Act for the Twenty-first Century (TEA-21).

POTENTIAL BENEFITS TO TRAFFIC SAFETY

GIS that are used for crash data analysis can benefit traffic safety. There are no research results that conclude that lives have been saved because of GIS. Rather, GIS has an indirect impact to traffic safety by assisting decision makers. Miller states it best, exclaiming "... the ultimate goal is not to conduct analysis, but instead, to take actions that will reduce crash frequency or severity *(4)*."

A case when GIS was successfully used to aid decision-making occurred in Miami County, Kansas. Twelve fatalities in 11 months occurred along a 20-mile stretch of two-lane U.S. Highway 169. The county sheriff was aware of the high fatality rate on this roadway section and approached the Land Information Management Office (LIMO) to create a GIS map showing the accident locations and the associated attributes. This graphical and tabular information allowed the Governor and other state officials to quickly analyze the situation and determine that a significant number of crashes were due to drivers failing to maintain control of their vehicles after leaving the travelway. After viewing the map, the Governor decided to widen this stretch of US 169. The Miami County Sheriff Department is now focusing on preventing future fatalities by performing GIS-based crash data analysis on other roadways *(5)*.

Bob Thompson from the Iowa Governor's Traffic Safety Bureau states that "GIS has the potential to revolutionize strategic enforcement applications for police and sheriff's offices around the nation. This potential is just beginning to be explored. EMS and other emergency response services can use the dynamic mapping features of GIS to improve the efficiency of their operations with *potential life saving benefits (6).* "

Macroscopic Applications

As stated previously, one of the largest benefits of GIS is the ability to analyze a large amount of data quickly. Macroscopic analysis can be used on large regions to identify areas of concern without getting into specifics.

High Crash Locations

One of the most common macroscopic applications is the determination of high crash locations (HCLs). HCLs identify the areas that would potentially receive the largest benefit if safety funds were allocated. These locations can be analyzed in many different ways.

One method of HCL identification includes crashes within a specified distance of a major roadway. This method is performed to include ramp and crossroad crashes in the crash frequency locations along a route. Two examples of HCLs in Ames, Iowa illustrate where safety improvements would provide the most benefit in accident reductions. These different approaches to displaying high crash locations are shown in Figures 1 and 2 respectively. Another method determines the crash frequency within a specified distance. An example would be crashes within a certain proximity of intersections. The presence of intersections adds complexity to the driving task, increasing the risk of a crash. This method identifies all crashes that this additional complexity may have contributed to. These crashes are referred to as intersection-related crashes. An example of intersection-related crash frequency is shown in Figure 3.

One of the drawbacks of identifying locations with high crash frequencies is that traffic volume or exposure is not taken into account. This can be accomplished by the crash rate method. The crash rate method for roadway segments divides the total number of crashes by the annual average daily traffic (AADT) and the length of the segment to obtain crashes per vehicle miles traveled (vmt). The crash rate for intersections divides the number of crashes by the number of entering vehicles determined by AADT to determine crashes per entering vehicles. An example of a crash rate analysis is shown in Figure 4.

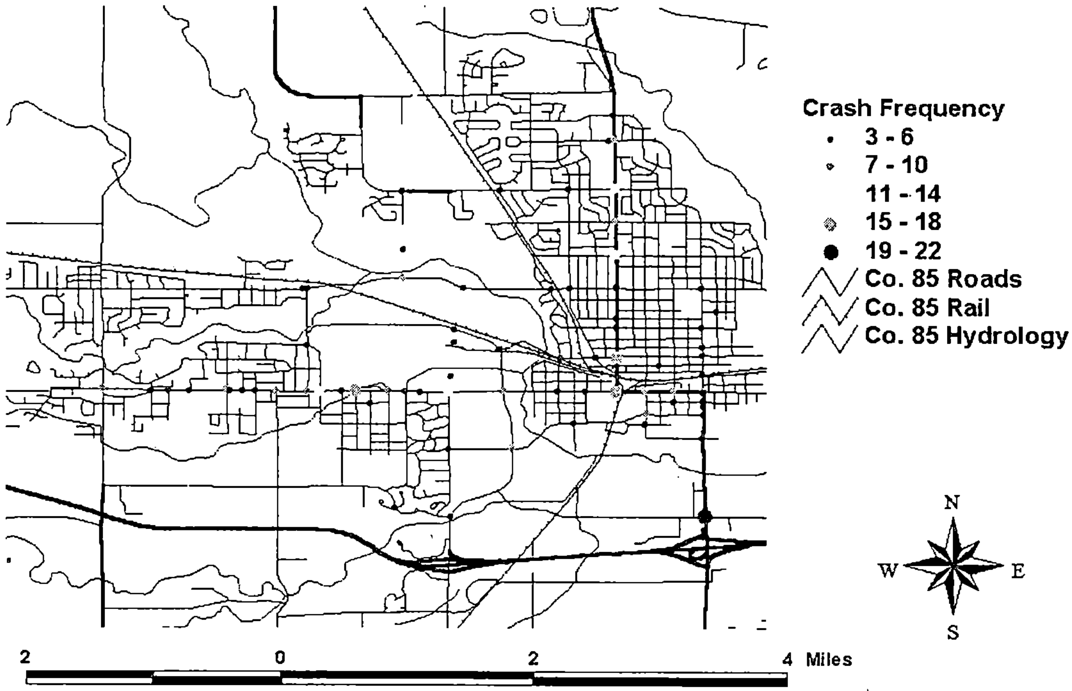

Figure 1 High Crash Locations, Ames, Iowa 1997.

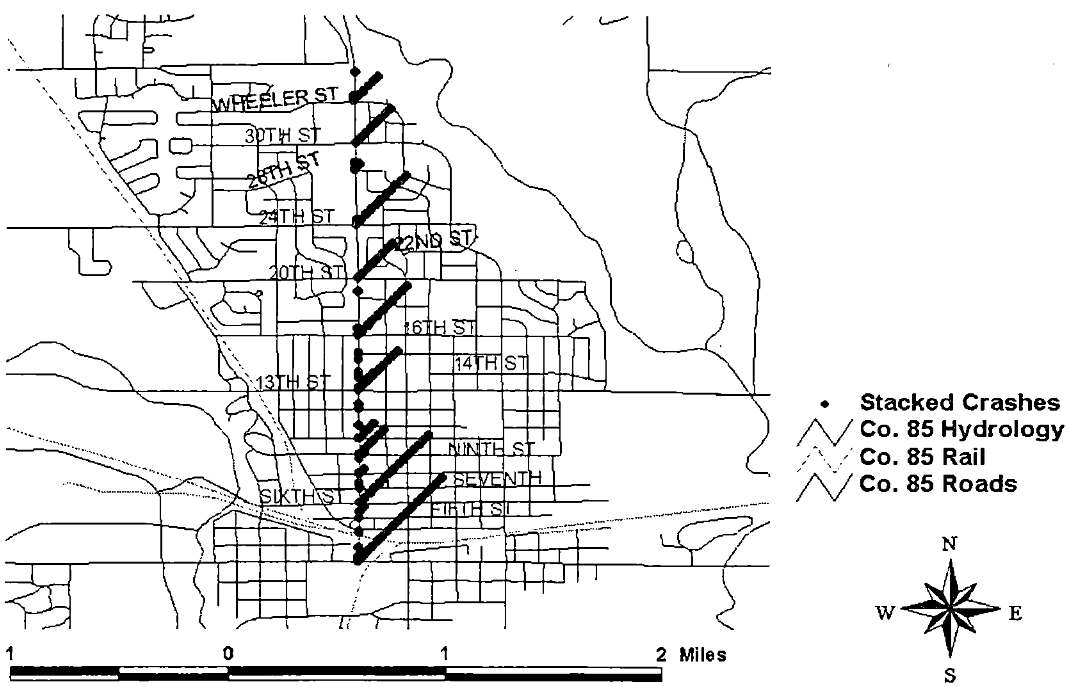

Figure 2 High Crash Locations, Grand Avenue, Ames, Iowa 1997.

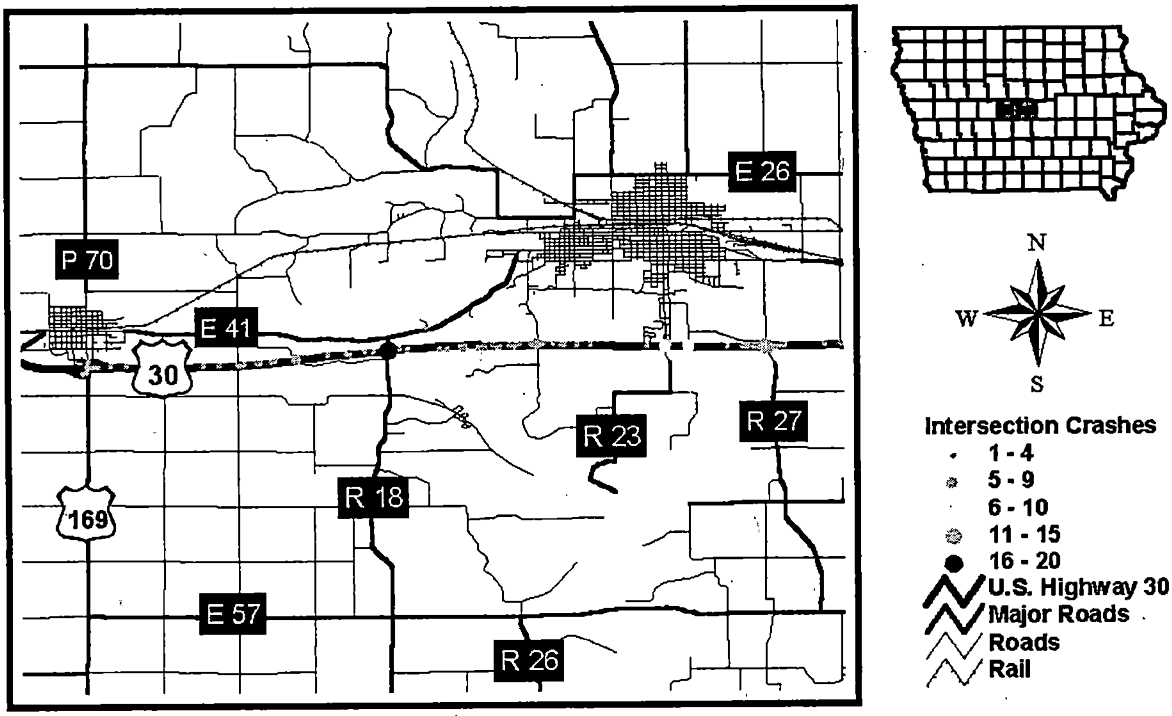

Figure 3 Intersection-Related Crash Frequency, Story County, Iowa 1993-1997.

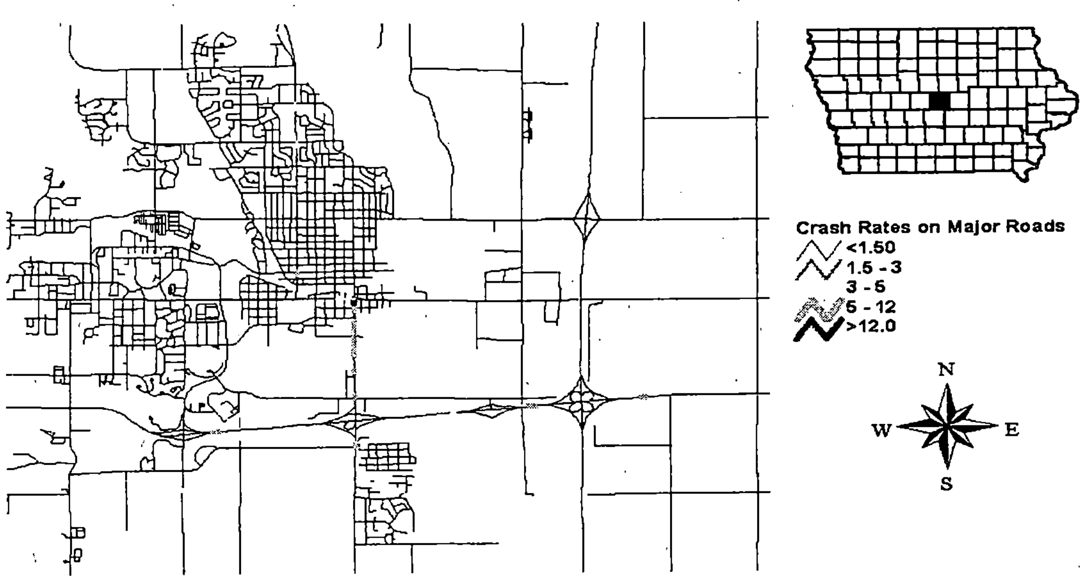

Figure 4 Crash Rate Analysis, Ames, Iowa 1994.

Spatial Queries

Large-scale spatial queries can be performed on an entire roadway network as well. A spatial query involves selecting an entire region and specifying crashes of a particular type. This is done to simplify the data to be studied. Only limited analysis can be done due to the large volume of information available. An example of this would be to evaluate only the location of fatal crashes for an entire county.

All of the macroscopic applications described could be carried out without the use of GIS by tables and spreadsheets. However, the GIS format makes these applications easy to perform and provides a visual display as well.

Microscopic Applications

The greatest benefit of GIS-based crash data analysis is the microscopic applications that can be done to evaluate crashes in a selected region. Various queries can be performed on isolated groups of data. Practical GIS applications are discussed to show how they can benefit each of the 4 Es of traffic safety: engineering, education, enforcement, and emergency response.

Engineering

GIS-based crash data analysis could have the largest impact on engineering improvements. There are only limited funds available for redesign and reconstruction of existing roadways and these funds need to be spent efficiently. Suppose the county engineer in Allamakee County, Iowa has a major roadway with many horizontal curves. There are some safety funds available for improvements. The engineer wants to know how best to spend these funds. With a GIS-based crash data system, the engineer can go to a personal computer and query all crashes that occurred on this roadway for a specified time period. The search can be refined to include only crashes that occurred on horizontal curves. The query can be refined by a number of characteristics. For example, the search could be narrowed down by injury severity, such as fatal and major injury crashes, so as to be concerned with only the most tragic events. The engineer can compare similar curves on different roadways with approximately the same physical features (e.g. number of lanes, lane and shoulder width, traffic volume) to determine if the curve of concern has a higher rate of crashes. Next, the engineer could look at the contributing circumstances of the crashes. If a contributing circumstance such as driver inattentiveness has a high number of occurrences, perhaps greater delineation or warning signs are needed. Perhaps the speed limit should be lowered or the curve redesigned so that drivers can maintain their initial travel speed without losing control. Many different possibilities and alternatives can be analyzed in a relatively short amount of time. All of this information can then be taken into consideration along with good engineering judgment when allocating the safety funds and making improvements.

Another microscopic application that can easily be done with a GIS database is the creation of computer-generated collision diagrams. First, GIS could be used to determine intersection HCLs as discussed previously. Once these locations have been determined, the user can select the desired intersection for a given time period and all of the crashes are shown with symbols depicting vehicle movement on a schematic drawing. Any attribute contained in the crash record databases can be shown on the collision diagram. Using GIS to create collision diagrams saves time and resources that would be spent creating these by hand (on paper or by a drawing program). This GIS-based collision diagram can then be used to study crash patterns and determine their causes. From this, patterns can be found and recommended improvements made to reduce the number of crashes. An example of a GIS-based collision diagram is shown in Figure 5.

One concern that is beginning to attract attention is older drivers. Iowa has the third highest percentage of drivers over the age of 65 and the second highest percentage of drivers over the age of 85 in the United States *(7)*.

Within the next ten years, the "Baby Boomer" generation will be added to this group of drivers over the age of 65. GIS-based crash data analysis can be used to quickly identify locations of elderly driver crashes so their causes may be studied. Engineers could take those locations and causes into account when considering such things as wider pavement markings, larger traffic control devices, and paved shoulders.

Enforcement

There are several benefits GIS-based crash data analysis applications can have in law enforcement. Law enforcement can improve traffic safety by targeting crashes of specific types, particularly those that endanger the lives of others. Most notable would be alcohol-impaired and alcohol-involved crashes. Alcohol-impaired means that the driver was legally intoxicated, while alcohol-involved means that the driver had consumed alcohol, but was under the legal limit. If a police chief wants to know where to place sobriety checkpoints, he could analyze the alcohol-involved and -impaired crash locations. GIS also has the ability to overlay and integrate land use information. This allows such analysis as alcohol-involved crashes and their location with respect to businesses that sell alcohol. An example of this is shown in Figure 6. Alcohol crashes can be further analyzed to determine such things as the day of the week and time of day. This can be used to help make decisions on staffing and shift hours.

Red-light-running (RLR) is also becoming an important topic in traffic safety. With these RLR locations known, patrol cars or mounted cameras could be strategically placed to observe offenders. Figure 7 shows the number of broadside (potential RLR) crashes compared to the total crashes at the designated intersections.

It is important to note that GIS cannot tell the user where to place enforcement solely by crash locations. For example, an area that has a high patrol rate may have no alcohol-involved crashes due to the existing enforcement in place. It does however help the user ask questions to ascertain their enforcement level and performance.

Education

GIS-based crash data analysis can also be used to help identify areas where additional education may be needed. Seatbelt use is one program that continues to get more and more support. In 1998, two-thirds of Iowa fatalities involved unbelted persons *(8)*. With the state seatbelt usage at an all-time high of 78 percent, the focus has now switched to rural areas where seat belt usage is typically below the statewide average *(8)*. GIS can make targeting these areas in need of increased education easy to determine.

Young drivers also remain a primary concern to the education world. New programs such as the graduated driver license have been implemented, targeting younger drivers who are over-represented in traffic crashes. GIS-based crash data analysis can help provide information on crash characteristics allowing more focus on the young driver errors.

A new application that GIS-based crash data analysis can be used for is to establish safe walking routes for school children. A "safe route to school plan" could be developed using GIS-based crash data analysis to generate a map, highlighting each child's safest route to and from school *(9)*. This analysis could also be used to identify locations where to install warning signs, to place crosswalks, to staff crossing guards, and load and unload buses.

Emergency Response

Emergency response is the newest addition to the four E's of traffic safety. Emergency response is concerned with providing transportation to hospitals and providing emergency equipment to the scene of the accident.

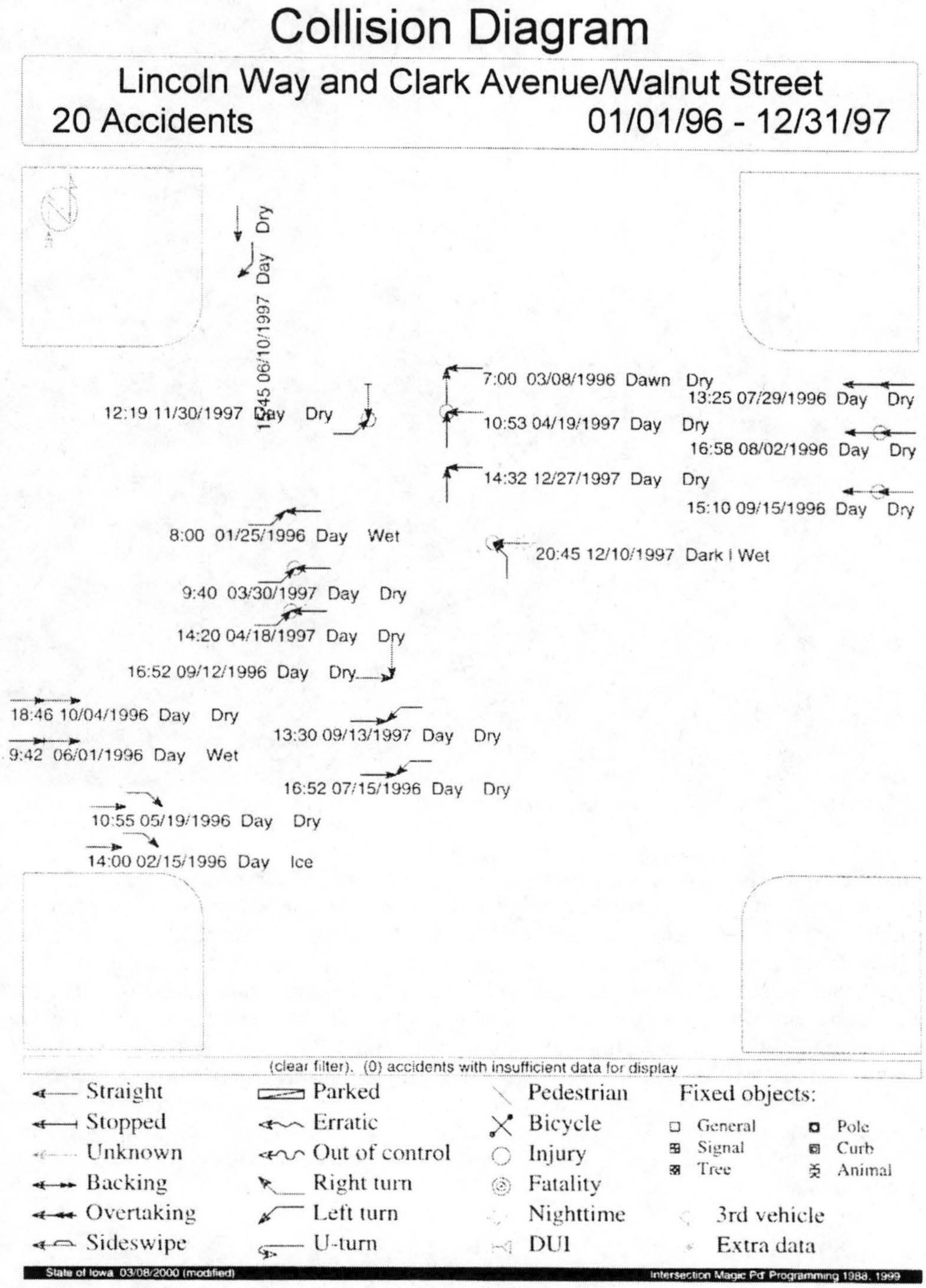

Figure 5 GIS-Based Collision Diagram, Ames, Iowa 1996-1997.

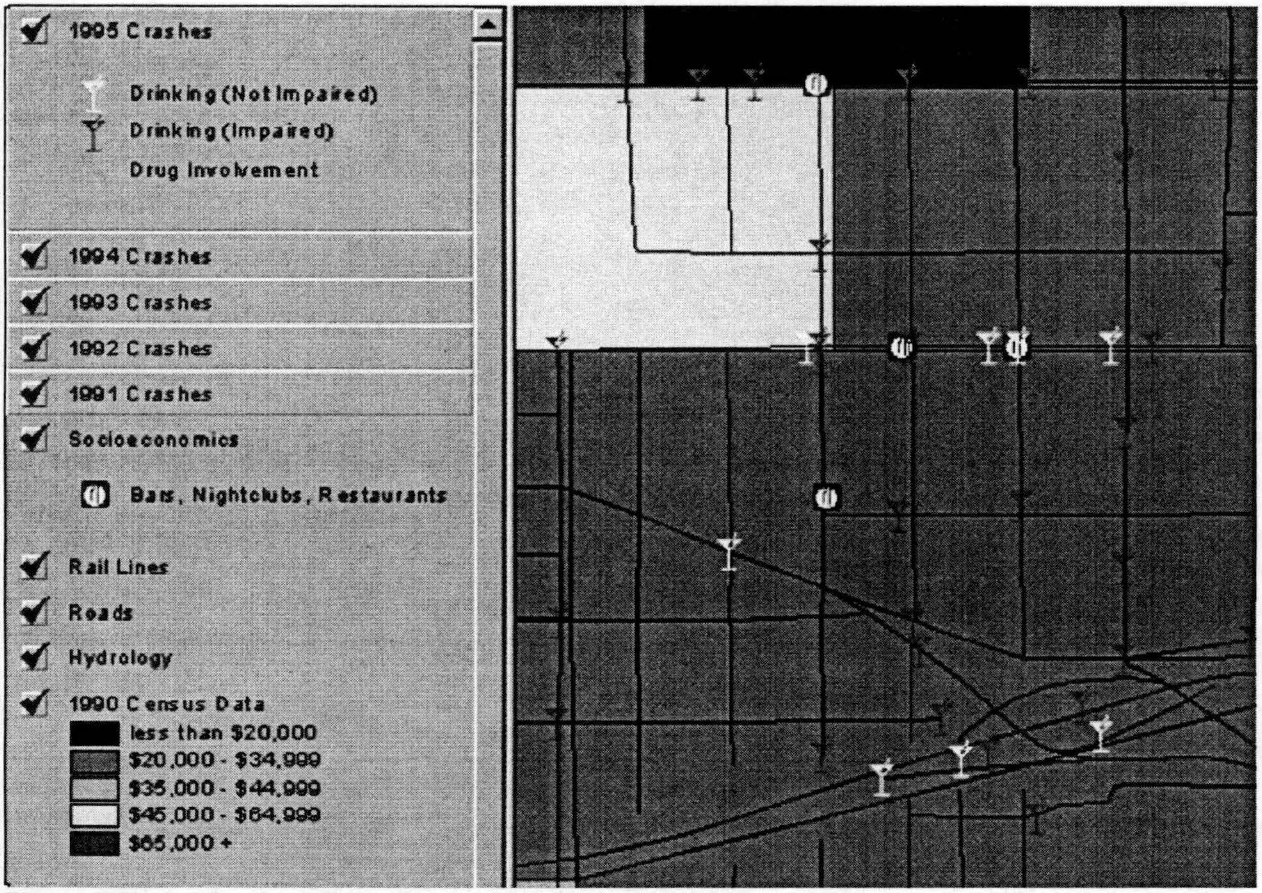

Figure 6 Alcohol-Involved Crashes, Des Moines, Iowa 1991-1995.

GIS-based crash data analysis can also be used to assist emergency medical services. Knowing the location of crashes as well of the frequency and severity can be used to determine satellite locations to place emergency personnel. The relative distance from a crash to the nearest hospital could identify where EMS access time could be critical. A GIS database could contain information such as the roadway system and the location of the nearest hospitals, including attributes, such as trauma level, hospital capacity, and even hospital specialization. In the future, it is expected that an ambulance driver with a global positioning satellite (GPS) receiver and a GIS base map will be able to look at the screen of an on-board laptop computer to determine which roads to take to get to the nearest hospital in the shortest amount of time. This also would allow for the nearest ambulance or fire/rescue unit to respond to a crash. This would be particularly applicable in rural regions where emergency response vehicles sometimes come from farther away than necessary due to established district boundaries. Injury crash location, severity, and frequency information could also be used in an urban setting to strategically place emergency vehicles and personnel in the field in order to shorten response time.

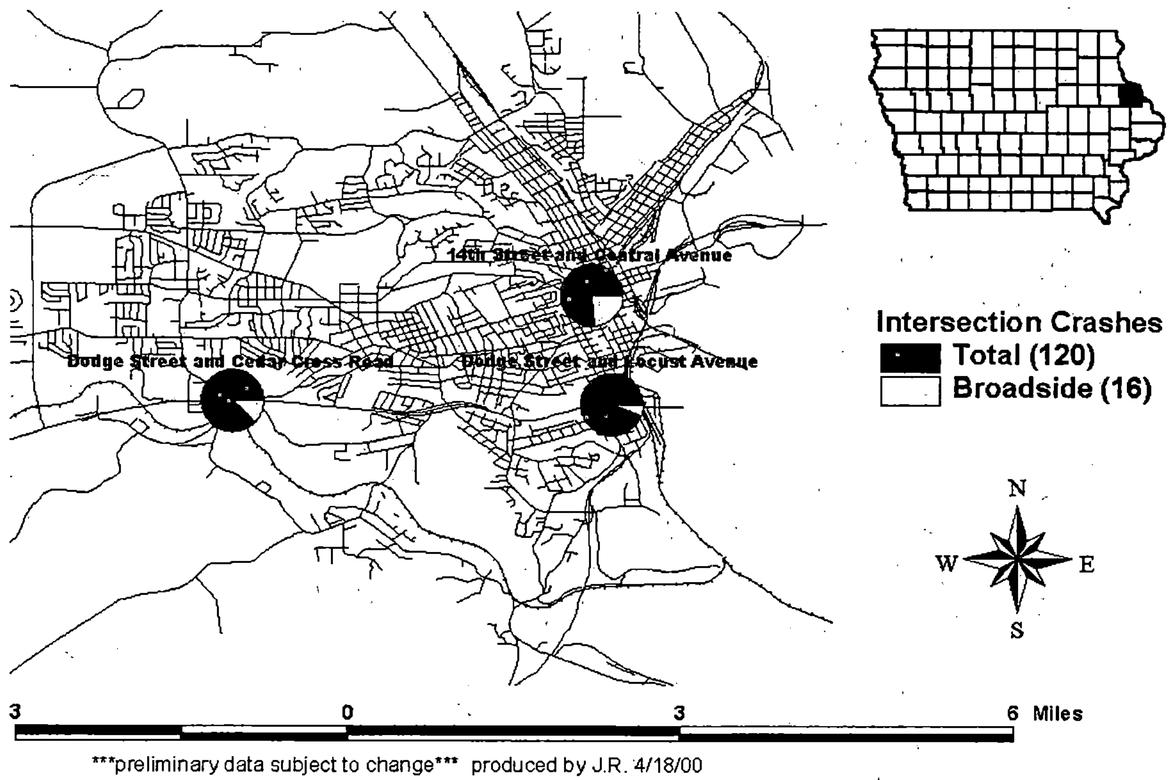

Figure 7 Potential Red-Light-Running Crashes, Dubuque, Iowa 1995-1997.

CONCLUSIONS

With the increasing availability of crash data and popularity of GIS software, GIS-based crash data analysis will be very useful to roadway designers, policy makers, decision makers, law enforcement, and emergency response personnel. Many applications have been presented that show how the time and effort required to display and analyze crash data can be effectively reduced.

At the macroscopic level, applications including high crash location identification, intersection-related crash frequency, crash rate analysis, and spatial queries allow the user to analyze and manipulate data quickly and identify potential problem areas. Some of these applications such as identifying high crash locations could be determined and summarized in a table without even using GIS. However, the GIS-based map connected to the tabular data provides the user with a visual representation and can assist in performing large-scale analysis.

At the microscopic level, crash data analysis can be performed to determine if a pattern or patterns exist. If such a pattern does exist, this may be a red flag that engineering, enforcement, education, or emergency response improvements or modifications are needed at those locations. Scenarios and alternatives that previously could not be analyzed can be considered. Traffic safety agencies can perform detailed queries on small subsets of data to effectively determine the potential causes of crashes and recommend potential countermeasures in their specialized areas.

As stated previously, GIS-based crash data analysis is only a tool. It can help the user make more-informed choices. It allows for a more thorough analysis while at the same time quickening the decision-making process. Used in conjunction with the traffic safety countermeasures already in place, GIS-based crash data analysis can have a significant impact on the traffic community.

SUMMARY OF FINDINGS

Although GIS crash data analysis began over 10 years ago, GIS-based crash data analysis has still not been used to its fullest potential. One main reason for this is the lack of interagency coordination. Collecting the crash data records involves several agencies and requires jurisdictions to work together. Also, some agencies have and use GIS independently of each other, resulting in duplication of data, incompatibility between systems, and inefficient use of resources. Another fundamental reason is that GIS-based crash data analysis requires additional staff, hardware, and software. The necessary funding is also not always available.

These difficulties can be overcome by coordination among agencies and departments by pooled funding and eliminating the duplication of data/effort. By using GIS-based crash data analysis, the time and effort required to analyze data can be reduced. Resources previously allocated to staff can then be applied more efficiently to other traffic safety issues. At the same time, an increasing number of scenarios and alternatives previously not possible can be evaluated, improving traffic safety.

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VISUALIZING SYSTEM-WIDE ECONOMIC IMPACTS OF TRANSPORTATION PROJECTS

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ABSTRACT

The economic evaluation of proposed transportation projects has traditionally been a technical process based on collected data and equations. Future needs must be considered to adequately meet the demands of system users. To ensure project success as political pressure forces transportation projects to be both beneficial and non-intrusive, transportation professionals must begin incorporating the public into every stage of a proposed project, including the economic analysis. By bringing together existing technologies, a streamlined process of producing future needs estimates, performing the economic evaluation of the proposed solution and displaying the costs and benefits of the solution has been created. This process is performed in a geographic information system environment that enables the efficient storage and visualization of data, thereby increasing the efficiency of the economic evaluation as well as providing a venue to display results.

INTRODUCTION

The decision to spend tax dollars for infrastructure improvement requires justifying the capital costs with more than simply a timesavings to the users. Current construction projects will not have the obvious economic benefits enjoyed by the Interstate System projects of the 1950s and 60s, therefore decision makers must be conscious of economic factors from the beginning of any proposed project (*1*). Both the scale and level of detail required for an economic analysis typically varies according to a number of factors including the size and estimated capital cost of the project, makeup of surrounding land-use and existing network infrastructure. Justification of smaller projects can normally be done with very little economic analysis while multi-million dollar projects typically require a detailed economic investigation.

In order to perform an economic evaluation of a proposed transportation project, a large set of data must first be collected. First the population and employment base upon which the costs and benefits will be spread must be established. An attempt at quantifying the costs and benefits must then be undertaken. Costs that should be considered during project planning range from the more quantifiable capital, maintenance and administrative costs to the more abstract environmental and societal costs. Typical benefits are a combination of increased access, mobility, safety or environmental friendliness depending upon the function of the project. Both the costs and benefits must be identified for the entire impacted area as well as the entire useful life of the project. The most critical issue is the effect the project will have on the traffic flow patterns throughout the affected area. This data may originate from spreadsheet calculations or more complicated travel demand models depending upon the size and complexity of the affected area, however either approach results in a link-by-link forecast of traffic volumes for each alternative (*2*). Pozenda suggests integrating a travel demand model with an economic evaluation model to provide better economic predictions as part of the transportation planning process (*3*). This paper uses the commercial travel demand model Tranplan to produce link speeds and volumes for both a base and an alternative scenario.

The Federal Highway Administration (FHWA) has produced several software packages that perform an economic analysis on proposed transportation project scenarios. Programs such as the Spreadsheet Model for Induced Travel Estimation (SMITE) and Sketch Planning Analysis Spreadsheet Model (SPASM) provide estimates based on spreadsheet calculations of traffic flow changes (*4, 5*). To assess impacts at the system level, FHWA has developed the ITS Deployment Analysis System (IDAS) (*4, 6*) and the Surface Transportation Efficiency Analysis Model (STEAM) (*4, 7*) programs. Both programs use network files containing travel speeds and traffic volumes, skim tree data, origin to destination tables, socio-economic data aggregated at the traffic analysis zone and a multitude of other economic variables to calculate traffic impacts due to an alternative scenario. National default values are incorporated into both software packages for all calculated costs, such as emission rate, fuel cost and accident cost. All default values are editable, allowing the programs to be customized for a particular city, county or state.

PROJECT OVERVIEW

To assist in the transportation demand modeling process, including calibration of the model and visualization of output, an interface has been created at the Center for Transportation Research and Education (CTRE) at Iowa State University linking Tranplan with the geographic information system (GIS) ArcView. This interface performs all required functions to build, edit, operate and analyze the Tranplan models for the base and alternative scenarios. The ArcView-Tranplan interface, along with documentation, is available free from the CTRE website at http://www.ctre.iastate.edu/Research/enhance/index.html (*8*). The interface then acts as a platform upon which an economic evaluation between a base and alternative scenario can be performed.

The Benefits Efficiency Analysis Module (BEAM) is the second edition of the STEAM program and was selected for this project over the IDAS software due to BEAM's ability to evaluate non-ITS scenarios. The chief enhancement of the BEAM program is the ability to perform economic analysis on user-defined districts within a network. BEAM conducts a detailed analysis of the costs and benefits between a base and alternative scenario, including an optional risk analysis, allowing for a range of possible values for the economic variables. A benefit to cost ratio for the alternative scenario is produced, along with several other scenario comparison values. This output can then be displayed at the district level in the ArcView-Tranplan interface. The BEAM program may also be available free of charge at the FHWA website (*9*). To produce the BEAM formatted input data and visualize the district creation and analysis output, the ArcView-Tranplan interface is used in conjunction with the BEAM software. A flowchart showing the general steps involved in performing an economic analysis with the ArcView-Tranplan interface and the BEAM program are shown in Figure 1.

To provide examples of the use of the ArcView-Tranplan interface, as well as the required input and output for BEAM, a case study for the city of Davenport, Iowa was conducted. The scenarios consist of the Tranplan network as provided by the Bi-State Regional Commission. The base scenario represents the 15 links that make up Kimberly Road as a four-lane facility as it exists today. The alternative scenario configures these same 15 links as six-lane facilities with higher capacities and free-flow speeds. Figure 2 provides the Tranplan network in the Kimberly Road vicinity.

ARCVIEW-TRANPLAN INTERFACE

The ArcView-based interface allows the user to either recreate an existing Tranplan network or create a new Tranplan network with the use of aerial photography within ArcView. Once the Tranplan network of links and nodes has been created, the interface creates control files to be executed by Tranplan in order to perform the traffic forecast. Tranplan is executed by the interface and the output information is returned to the interface to be added to the original Tranplan network. Analysis and calibration tools have also been added to allow for model validation with the ArcView-Tranplan interface (*10*). Figure 3 shows the interface with the Tranplan information added for the base and alternative scenarios of the Kimberly Road case study. The original network (links and nodes shapefiles) was created from the formatted text files used by Tranplan. The ld_links and connect shapefiles contain the original link attributes from the links shapefile, along with the Tranplan output of forecasted link speeds and volumes.

The interface has incorporated the BEAM program to provide users with economic analysis functionality when comparing transportation project alternatives. Similar transportation GIS packages are also available commercially such as Viper (*11*) and TransCAD (*12*). The Federal Highway Administration's STEAM website provides conversion programs to convert several travel demand models including TransCAD, TP+, Emme/2 and Tranplan into STEAM input files (*13*).

To begin an economic analysis using the ArcView-Tranplan interface, a completed Tranplan model loaded into ArcView must exist for both the base and alternative scenarios. Note: Both projects must have the same map projection to allow for overlaying the network files. The first step in performing the economic analysis is to load the alternative scenario shapefiles into the base scenario's ArcView project. This feature has been automated through the use of the ArcView-Tranplan interface.

FORMATTING REQUIRED INPUT DATA FOR BEAM

The BEAM program requires input files for the both the base and alternative scenarios. By using a GIS package to both visualize and reformat the data, the user can easily verify the validity of the BEAM input files. For each scenario, a formatted network file containing information about each link in the network is required. An origin/destination file and skim tree data file are also required for both scenarios. Since the BEAM program requires that the number of traffic analysis zones remains constant from the base scenario to the alternative scenario, only one centroid file is required as input. A district file containing the name of each district is also required. Table 1 shows the required input files for a BEAM analysis.

To distribute user costs and benefits to each district, the population and employment of each zone may be used by BEAM. To transfer this zonal information to BEAM, the data must first be entered into the centroid attribute table. Although these data are not required to perform a BEAM analysis, the population and employment data will result in a more appropriate analysis of user benefits. Due to its ease of data handling, the use of ArcView to populate both the population and employment fields in the centroid attribute table before beginning the BEAM analysis is encouraged.

The BEAM economic analysis program allows the user to specify districts upon which an economic evaluation will be conducted. These districts are smaller regions within the network, such as the central business district or a suburb. The interface allows the user to create the district polygons on top of the existing Tranplan network (Figure 3), however, the district file used as a BEAM input file is created in a later step in the process. The district polygons are also used later by ArcView to thematically display the BEAM output at the district level.

BEAM requires that a network file containing data for every link in the network is created for both the base and alternative scenarios. These link records must be in one-way format, ordered by both the a-node and b-node of the link and must contain the distance, free flow speed, capacity, volume and functional class of the link. These files are created by the interface through the use of the Tranplan Data Formatting input screen (Figure 4). The interface accesses the ArcView database files then formats and prints the data in BEAM format. The BEAM Users Manual (*9*) discusses the format requirements for the network links in more detail.

After the link files for the base and alternative scenarios have been reformatted, the shortest impedance route data from each zone to every other zone (skim trees) for both scenarios are extracted from Tranplan through the aid of the interface. With link speeds, volumes, skim trees and turn prohibitor information, a Tranplan control file is created that will output the shortest path between all zones for both the base and alternative scenario networks. After the new control file is written, Tranplan is executed and a text file containing the shortest path information for both the base and alternative scenarios is output. These files contain each origin zone, destination zone and the corresponding time to travel between the zonal pair based on the last traffic assignment iteration. Another vital data source is the origin /destination table for both the base and alternative scenarios.

A final reformatting of the Tranplan output into the required BEAM format is required and is performed by the BEAM Input Builder screen. The output of this function is the BEAM formatted skim data files and origin/destination files for both base and alternative scenarios, as well as the corresponding centroid and district files. Intrazonal trips are not included in the BEAM analysis.

RUNNING BEAM

Once all required input files have been created by the ArcView-Tranplan interface, the BEAM program is started. If creating a new BEAM analysis, the user must first define the analysis and enter the district file name which will be used later to bring the BEAM output back into ArcView.

The next step is to define the market sectors to be analyzed. The user also specifies the auto occupancy, expansion factor, value of time and speed relationship along with several other parameters. The expansion factor allows BEAM to convert a daily travel demand model into a yearly economic analysis. A value of 250 is suggested by FHWA to convert a weekday model into a yearly analysis. After defining the market sector, the formatted input files are entered into BEAM. Both the network and trip table files are entered into BEAM for the base and alternative scenarios in addition to the common centroid while the skim files are input into BEAM as 'In-vehicle Changes'.

Users may elect to modify the default model parameters used during a BEAM analysis. The default values are based on national averages, however values appropriate to the study area should be obtained if possible. For the Davenport case study, information regarding crash costs and frequencies were gathered from the Center for Transportation Research and Education's traffic safety data center and the Iowa Department of Transportation. Other variables considered during the case study included the cost and tax rate of gasoline and the value of time to drivers in their vehicles.

The last required inputs are the capital costs and operations and maintenance costs. These values are input into BEAM, along with the useful life of the improvement, the completion date and the salvage value of the improvement. The Kimberly Road improvement is estimated to cost $18 million dollars and increase the annual maintenance cost of the corridor by $7650 per year.

The final step is running BEAM. The user may choose to perform a risk analysis that will vary the value of each economic variable in order to produce an output showing the outcome's sensitivity to that variable. Upon completion, the results may be viewed in BEAM along with histograms showing the range of possible outcomes as calculated during the risk analysis. To learn more about the BEAM program, users may consult the BEAM Users Manual or contact user support at Cambridge-Systematics (*9*).

BRINGING BEAM OUTPUT INTO ARCVIEW

After the BEAM economic analysis has been completed, the user should verify the analysis results. Verification of results may be done by comparing the travel demand model's traffic assignment outputs of system-wide VHT and VMT with BEAM's system-wide VHT and VMT. When satisfactory results have been achieved, the output may be brought back into ArcView. This is done to allow the user to graphically view the analysis results at the district level and compare the proposed project's effects on one district versus another district.

ArcView can quickly and easily join data sets on a one to one basis as long as both sets have a common attribute such as district number. The final step in the BEAM analysis process is the joining of the BEAM district output to the district shapefile originally used to create the BEAM district input file. This information can then be displayed graphically in ArcView showing the effect the proposed project would have in one of various economic categories. By providing aerial photography, street networks or land use maps in conjunction with the BEAM output, information can be more easily conveyed to both decision makers and the public. Tying the economic values to geographic features provides an easily understood basis upon which to build arguments concerning the proposed project.

CASE STUDY

The Kimberly Road case study provides a good basis upon which to evaluate the performance of the ArcView-Tranplan interface's BEAM module as well as the feasibility of using the BEAM program on smaller urban projects. Perhaps due to the use of only one mode of transportation for this case study, the true strength of the BEAM program was not fully utilized. Another factor to consider when evaluating the use of BEAM is the amount of congestion experienced within the network. BEAM has been developed to explicitly evaluate the more complex congestion associated with queuing and incidents rather than volume to capacity ratios alone as well as redistribute the productions and attractions within the network according to accessibility to the network. BEAM also allows for multiple capital cost projects to be included over a period of time, while the Kimberly Road case study used only one capital improvement. However, BEAM did appear to produce reasonable results when the Tranplan daily output for VMT was compared to the BEAM yearly VMT. BEAM's prediction of VHT was greater than the Tranplan estimate again due to BEAM's accounting for complex congestion. Figure 5 shows the risk analysis histogram of the costs of the Kimberly Road case study. The bottom line of the Kimberly Road case study is the benefit-cost ratio of the proposed project. The system-wide ratio was calculated by BEAM to be approximately 5.3 to 1. This indicates a very high benefit return for the capital and user costs associated with the project. Figure 6 shows the total user benefits to the Quad Cities residents that are attributed to the proposed widening of Kimberly Road. By overlaying the BEAM output with the major streets within the Quad Cities, the proposed projects effects have been tied to geographic locations that are quantifiable by nature. The ArcView-Tranplan interface proved to be a useful aid in the Kimberly Road analysis by storing, displaying, and formatting input data as well as displaying the BEAM output at the district level.

The Kimberly Road case study has brought about several items to consider when contemplating the use of BEAM for the economic analysis of transportation projects. The first item is the quality of the travel demand model output used in BEAM. Users should be sure that the traffic assignment is revealing assignment differences due only to the scenario change. The second item is the functional classification used in the model network. BEAM relies heavily on the functional class assigned to each link when assessing benefits, especially pollution and accident costs. Finally, users should consider collecting as much local information related to the user costs as possible. Contacting the state department of transportation may be a great starting point for collecting the accident, emissions and other costs that make up an economic evaluation.

CONCLUSIONS

The goal of any alternatives analysis is to gather information which will aid in the decision making process. In the past, technical professionals involved in alternative analysis relied upon the comparison of benefits and costs to select the course of action. However, the interaction of the transportation network, the transportation users and the surrounding land use make quantifying costs and benefits over any period of time a difficult assignment. By utilizing FHWA software advancements, a more comprehensive assessment of project impacts can be made. As transportation related decisions become more complex and embedded in public opinion, the need to convey technical information in a simple and graphical manner must be addressed.

The ArcView-Tranplan interface is similar to other available tools that allow transportation professionals to create and edit Tranplan networks, perform the Tranplan analysis, use standard modeling tools on the output and visualize the entire process with the aid of aerial photography or other underlying cartographic layers. BEAM is comparable to other economic evaluation programs in its ability to account for a variety of economic factors that affect the benefits of transportation users in a given system. By combining these two programs, a visual economic analysis can be performed on projects that are only in the early planning stages such as the Kimberly Road project. This package of programs could be quite useful for not only the technical portions of an economic feasibility study, but in educating the public on the effects of proposed projects as well.

ACKNOWLEDGEMENTS

The author would like to thank the Iowa Department of Transportation Offices of Systems Planning and Traffic Safety for their assistance and guidance during this project as well as the Bi-State Regional Commission and the Center for Transportation Research and Education. Cambridge Systematics was also instrumental by providing technical assistance throughout the project.

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TABLE 1. Required Input Files to a BEAM Analysis

Input File Type	Base Scenario	Alternative Scenario	Both Scenarios
District Description File			X
Centroid File			X
Network Link File	X	X	
Origin/Destination File	X	X	
Skim Tree File	X	X	

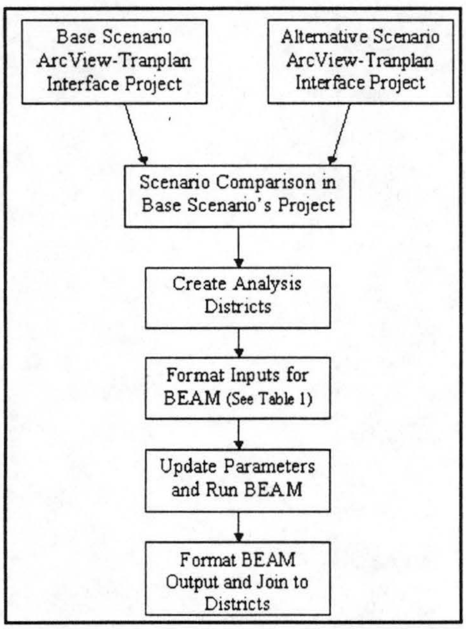


FIGURE 1. Flowchart of Economic Analysis Using ArcView and BEAM

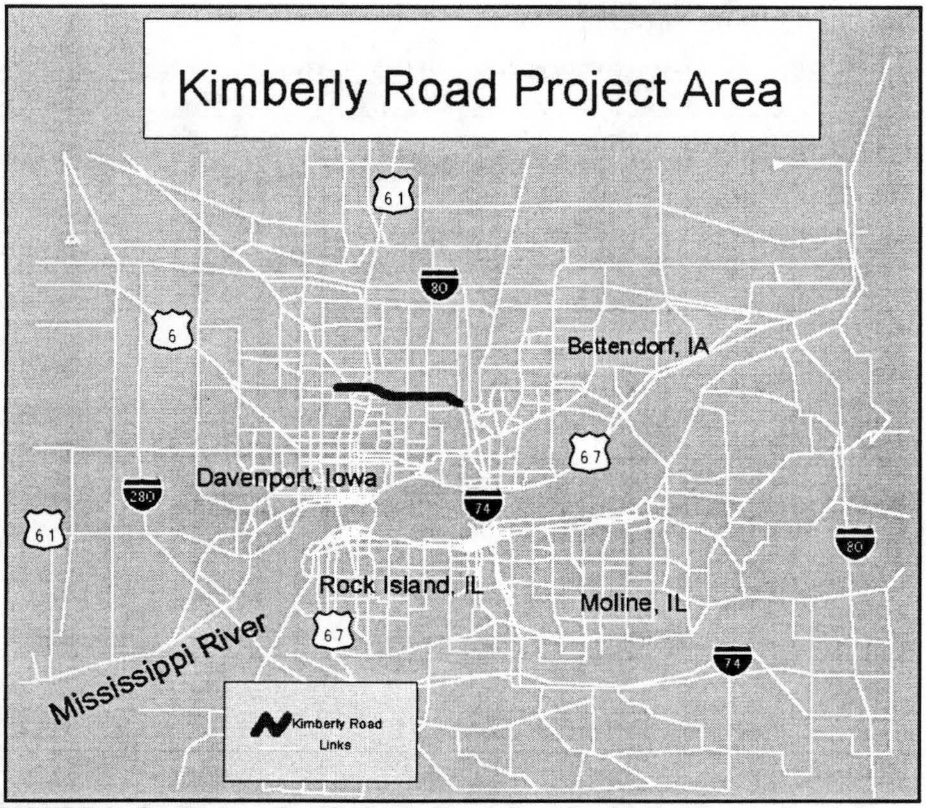


FIGURE 2. Study Area for Kimberly Road Case Study, Davenport, Iowa

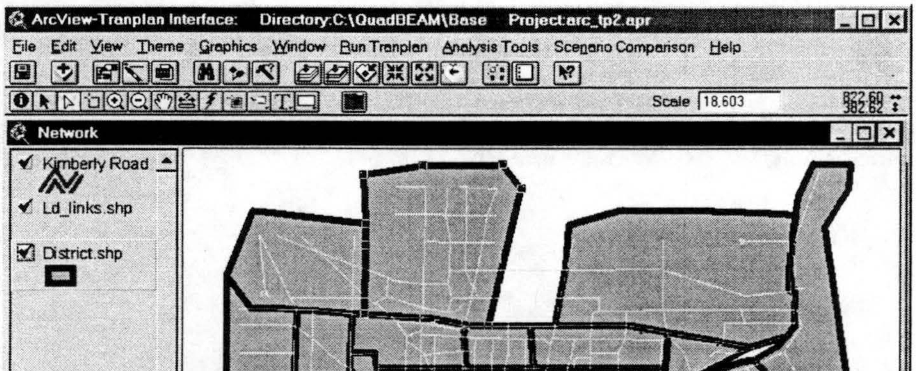


FIGURE 3. Creating Analysis Districts in ArcView

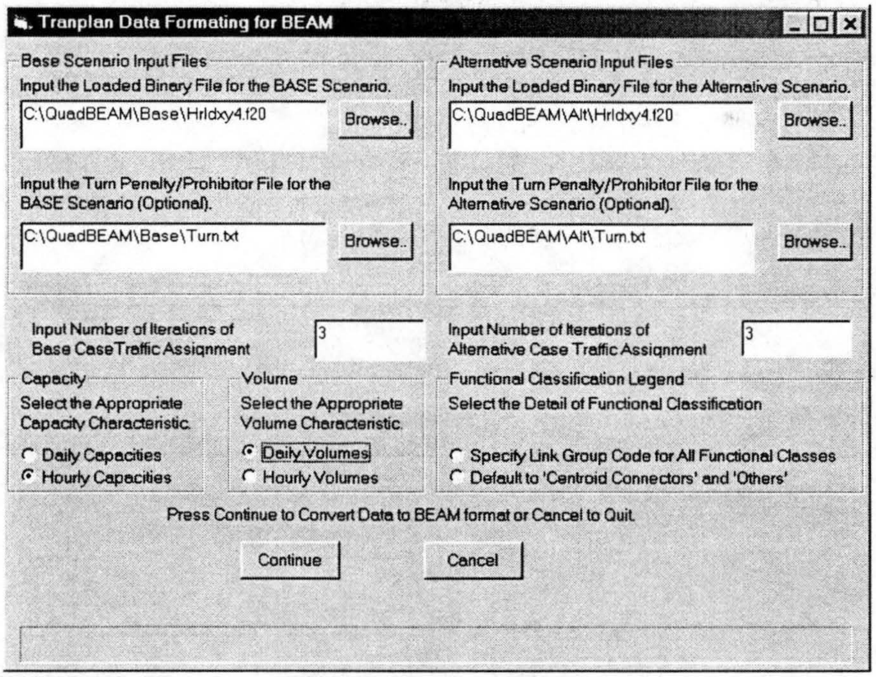
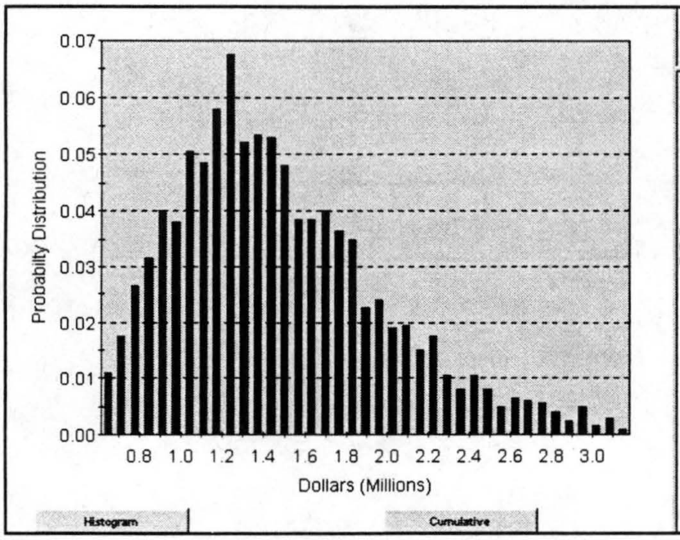


FIGURE 4. Tranplan to BEAM Data Formatting Screen

(Creates Network Links Files and Tranplan Trip Table Control File)



Value	Prob. of Exceed.
3.28	1%
2.59	5%
2.24	10%
1.91	20%
1.72	30%
1.56	40%
1.43	50%
1.30	60%
1.20	70%
1.07	80%
0.92	90%
0.81	95%
0.65	99%

FIGURE 5. Risk Analysis Histogram for Costs in Kimberly Road Case Study

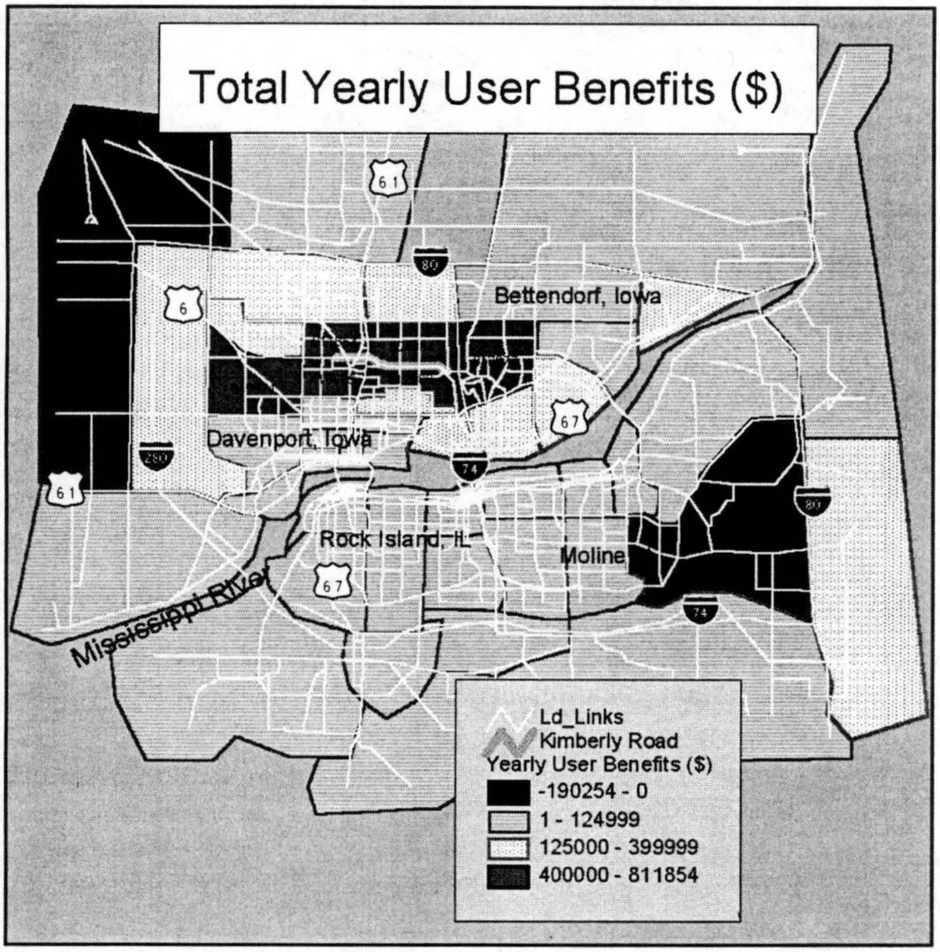


FIGURE 6. BEAM District Level Output in ArcView

HUMAN FACTORS AND TRAFFIC CRASHES

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ABSTRACT

A major focus of transportation engineers and all engineers in general is safety. Currently, an estimated 41,000 people die annually in automobile crashes *(1)*. There are many factors that contribute to crashes with the main instigator of crashes being driver error. Driver error makes designing safe roadways difficult and challenging for engineers since every driver is different. The list of possible human errors is long. However, this paper focuses on three distinct factors that may cause a driver to commit an error while driving.

The first factor is the effect of alcohol on the driver and his or her performance. Studies dealing with alcohol impaired driving have indicated that the number of alcohol related fatalities may be underestimated. Studies also show that some drivers become impaired well before the blood alcohol content (BAC) of 0.10 is reached. Several methods of deterrence of drunk driving have proven to be effective. One of the more effective measures is the use of sobriety checkpoints.

The second factor discussed is driver fatigue. Research has shown that fatigue can be caused from such activities as business of social activities, holiday events, and family gatherings. Research has also shown that the majority of the fatigue related crashes occur between 8 P.M. and 6A.M. Legislation and enforcement of fatigue related driving is difficult since fatigue is not an easy concept to define or place limits on. Several solutions have been proposed to reduce fatigue-related accidents including additional rest areas and stricter limits on the trucking industry.

The third factor discussed is driver distraction by the use of cellular phones. Research has shown that the risk of an accident increases when cellular phones are combined with driving. It has been shown that driver reaction time and mental workload both increase with cellular phone usage. Several countries around the world have already banned the use of cellular phones while driving while other areas consider enacting legislation.

INTRODUCTION

Crashes have occurred since the invention of the automobile. In February of 1966, Lyndon B. Johnson said that the death and injury occurring on highways was "...the greatest problem before the nation next to war in Vietnam *(2)*." Currently, an estimated 41,000 people die annually in traffic accidents *(1)*. This number has decreased slightly over the past few years but still accounts for a large number of deaths in the United States. Some reasons for the slight decrease in automobile crash fatalities may include legislation requiring seatbelt usage, the use of airbags, better vehicle and roadway designs, advances in emergency and medical response, and better road maintenance in poor weather conditions. The reason these remedies have a minimal impact is simple; they deal with only safety features regarding the roadway and the vehicle while error on part of the drivers accounts for 90 percent of all traffic accidents *(3)*. Safer cars and roadways can be produced through design, but making drivers safer is much more difficult. Driver error can include a number of topics including misinterpretation of traffic control devices, road rage, driver expectancy, driver age, and mental workloads. This paper focuses on three factors that contribute to driver error and crashes. Two of the topics covered are problems that have contributed to accidents since cars were first produced. The first is drivers operating under the influence of drugs and/or alcohol and the second is driver fatigue (i.e. drivers falling asleep behind the wheel). The third topic, which will be discussed briefly, is a problem that is relatively new to the transportation field but has become more of an issue with time. This topic is driver distraction due to the use of cellular phones and how they made contribute to a crash.

IMPAIRMENT DUE TO ALCOHOL/DRUGS

Background

It is estimated that two out of every five Americans will be involved in a crash in which alcohol is involved. In 1995, alcohol fatalities accounted for 40 percent of all fatal crashes (17,274 deaths) nationally as well as over 300,000 injuries. This death rate is equivalent to an alcohol fatality rate of one every half-hour and is up 4 percent from 1994 totals *(4)*. In Iowa, 27 percent of all fatal accidents in 1997 was alcohol related.

A significant number of these accidents involve younger inexperienced drivers. Motor vehicle deaths are the number one cause of death for people 15 to 20 years of age *(5)*. Figure 1 shows the number of motor vehicle fatalities for this same age group and the number of fatalities in which alcohol was present. As can be seen, nearly half of the fatalities on weekends and almost one-third on weekdays are alcohol related *(5)*. Between 1992 and 1997, 30 percent of all alcohol related accidents involved intoxicated drivers 16 to 25 years of age *(4)*. In 1995, 55 percent of the fatal accidents involved intoxicated drivers 21 to 34 years of age. If past trends continue, it is estimated that in the next ten years nearly 400 people under the age of 25 will be killed in alcohol related accidents *(4)*.

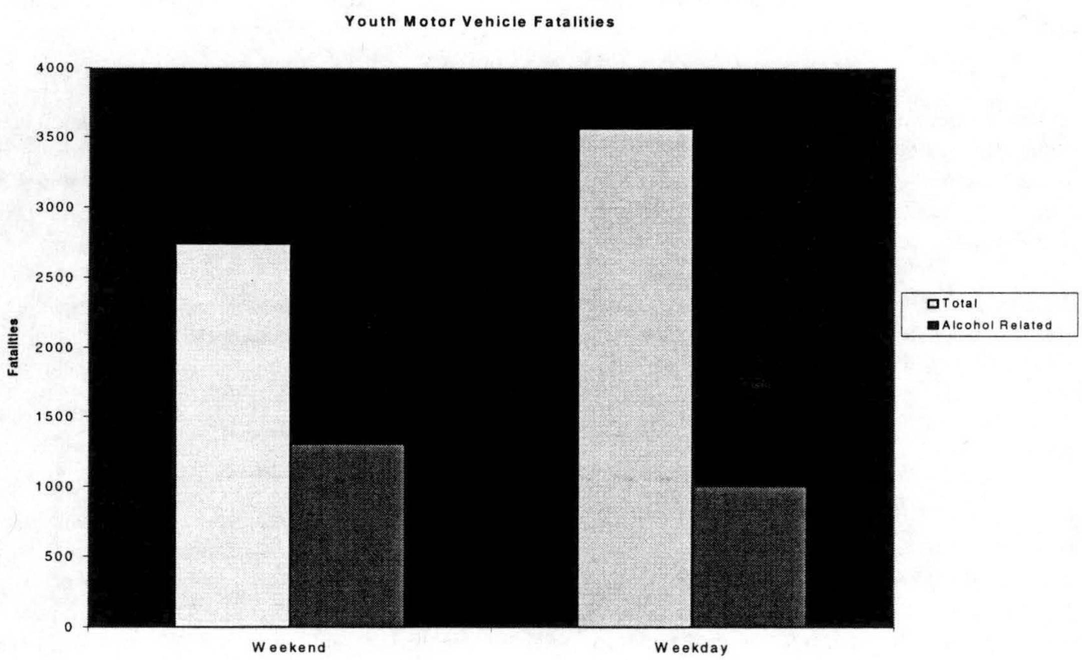

Figure 1-Youth Motor Vehicle Fatalities *(5)*.

A Study on Driver Fatalities and Alcohol Involvement

The number of actual alcohol related fatalities might be underestimated. A group of researchers, Olga J. Pendlton, Nancy J. Hatfield, and Ron Bremer, of the Texas Transportation Institute and Texas A & M University conducted a study in which fatal crashes and driver intoxication levels were compared *(6)*. The purpose of the study was to determine how accurately fatal crashes are investigated and traffic accident reports prepared by police with respect to the involvement of alcohol.

The methodology used by the researchers included the comparison of toxicology and autopsy reports with traffic accident reports. The goal was to determine the degree of accuracy of traffic accident reports when compared to reports prepared by medical examiners. The blood alcohol content (BAC) of the fatally injured driver was used to make the comparison between the two reports. BAC is measured as a percentage in terms of the amount of alcohol in a volume of blood. For example, a BAC of 0.10 (the legal limit in Iowa) means that there is one-tenth of a percent of alcohol in the blood by volume. Pendlton, Hatfield, and Bremer collected BAC content data from medical examiners in nine Texas counties that perform postmortem BAC tests on fatally injured drivers. Though only nine counties in Texas were studied, over half of the population of Texas was included in the study area. The data collected came from crashes that occurred between January 1, 1984 and December 31, 1984. In total, 1,260 driver fatalities were studied.

Of the 1,260 accidents reviewed, Pendleton, Hatfield, and Bremer determined that 51 percent of the drivers had a BAC of 0.10 or greater according to the toxicology reports. This means that over half of the 1,260 drivers killed were legally intoxicated at the time of the accident. When the accident reports were reviewed, only 20 percent of the crashes were listed as alcohol related. In other words, the investigating officer misinterpreted about 30 percent of the crashes. Therefore, the number of alcohol related fatal crashes that occurred that year were greatly underestimated. Pendleton, Hatfield, and Bremer also investigated the discrepancies between toxicology reports and accident reports with the law enforcement agency responsible for the investigation of the fatal crashes. It was determined that investigating officers with the Texas Department of public Safety failed to list alcohol as a contributing factor for 30 percent of the accidents they investigated while the local police agencies misinterpreted 76 percent of the accidents that they investigated with respect to alcohol involvement. This discrepancy between law enforcement agencies may indicate a difference in, or lack of, training and education in the investigation of crashes and in determining what factors contributed to the crash. It is also worth noting that the study indicated that the investigating officers for the Texas Department of Public Safety over reported alcohol as a contributing factor in crashes by 15 percent while local police agencies over reported alcohol related accidents by 6 percent.

An accurate estimate on the number of fatal accidents in which alcohol was a contributing factor can be very difficult to determine from merely traffic accident reports. These reports are both subjective and non-quantitative in nature. A highly reliable estimate can be obtained from determining the BAC of all drivers of all vehicles involved in an accident. The BAC value is non-subjective and quantitative *(6)*. However, this data is often unavailable. This may be due to several reasons. First, the lack of legislation requiring postmortem BAC tests on all fatally injured drivers. As of 1986 only 35 states required postmortem BAC tests by law *(6)*. Second, the lack of facilities and staff needed to perform BAC tests on all fatally injured drivers *(6)*. And the third reason is there may be an excessive period of time that elapses between the time of the accident and the time of the autopsy, which can lead to erroneous test results *(6)*.

Effects of Low BAC's on Drivers

There have been numerous studies performed over the years that deal with alcohol and it's impact on drivers. An obstacle encountered in all of these research studies is the fact that each individual person will respond differently at a given BAC. Two main factors that cause these variations are a person's metabolism and the rate at which the alcohol is absorbed into the bloodstream.

The Committee on Benefits and Costs of Alternative Federal Blood Alcohol Concentration Standards for Commercial Vehicle Operators, a committee of the National Transportation Research Board, published a special report titled *Zero Alcohol and Other Options,* which, contained research data from various studies dealing with the impact on driving functions and BAC levels. One study performed by G. Wilson and R. Mitchell showed that drivers were unable to separate fine visual details as accurately or judge distances between themselves and other vehicles with BACs as low as 0.04 to 0.06 *(7)*. Another study listed by the National Transportation Research Board Committee was one performed by R.G. Mortimer. Mortimer studied how BAC effected a drivers tracking ability. The study revealed that with BAC as low as 0.01, tracking was greatly impaired when combined with low illumination (i.e. nighttime driving) and a glare (i.e. oncoming headlights). As BAC increased, the tracking ability of the driver decreased for any illumination situation *(7)*. These studies suggest that many drivers become impaired at a low BAC.

Deterrence of Drunk Driving and Alcohol Related Fatalities

There are two distinct levels of deterrence, general and specific. General deterrence is "the effect of threatened arrest and punishment upon the total driving population *(8)*." Specific deterrence is defined as "efforts to prevent single offenders from drunk driving again *(8)*." The following sections provide some examples of general and specific deterrence that can be implemented to reduce the number of alcohol related accidents.

Adjustment of the Legal BAC

Minnesota was the first state in the United States to create "illegal per se" legislation. In 1976 the state of Minnesota made it "illegal per se" to drive a vehicle while under the influence of alcohol defined as having a BAC of 0.10 or higher. Iowa followed in the footsteps of Minnesota and in 1982 made it illegal to drive with a BAC of 0.10 or higher. This "illegal per se" idea has spread throughout the United States and the world.

Some agencies and legislatures would like this legal BAC limit lowered to 0.08. There are many states that currently have a legal BAC limit of 0.08. These states have shown a "significant statistical correlation in lowered alcohol-related fatalities" since the BAC limit was lowered to 0.08 *(4)*. As was discussed previously, studies have shown that some drivers do become impaired before a BAC of 0.10 is achieved. This is a strong argument for lowering the BAC from 0.10 to 0.08.

Licensing and Revocation

As was mentioned earlier, the leading cause of death in people of 15-20 years of age is automobile accidents. Currently, many schools and organizations attempt to educate students on the effects of alcohol and drugs. Though this has been shown to decrease fatalities, 30 percent of all the fatal alcohol related accidents from 1992 to 1997 involved 16-25 year old drivers *(4)*. One way to decrease the number of young people killed in alcohol related accidents are to keep the young people off of the road. This can be accomplished by a graduated drivers license system. A graduated drivers license system would require new drivers to progress through a gradual program that limits the time of day, destination, and number of passengers in the car. A driver can graduate from one level to next with a clean driving record over a fixed period of time until the point where full driving privileges are reached. Studies have shown that alcohol plays a large role in the number of fatal traffic accidents that occur on the weekends. An application of a graduated license system could be to limit when young people are allowed to drive over the weekend (i.e. during daylight hours only). This may help reduce the fatal alcohol related accidents involving young people.

Another way to reduce alcohol related accidents among young people would be to keep them alcohol free. To discourage minors from purchasing alcohol, the minor's driver's license could be revoked and anyone under the age of 21 who purchases or attempts to purchase alcohol with a valid, invalid, altered or fake identification could have there driving privileges revoked for a specified amount of time. A fine or citation may also accompany this revocation. Illinois currently has similar legislation that revokes the driving privileges for up to one-year *(4)*.

Revocation may also include the seizure of the vehicle being driven at the time of an alcohol related offense. Immediate seizure of the vehicle may be the best way to keep an operating while intoxicated (OWI) offender from driving while their license is suspended. However, seizure of vehicles has proven to be somewhat ineffective. The main reason for the ineffectiveness is there is only "sporadic" enforcement since vehicle seizure is included in the sentencing of a guilty party and not all judges order the seizure *(4)*.

Drinking and Driving Citation

Another solution may be the issuance of a drinking and driving citation. This citation would be issued to drivers who have a BAC below 0.10 but higher than 0.04. This citation would be a simple misdemeanor with a fine. If a citation were issued to drivers who have BAC of 0.04 or greater, the use of designated drivers would be encouraged. For citations to be effective, criminal operating while intoxicated (OWI) charges cannot be lowered to a simple citation. If this were to happen, an opposite effect would take place and drivers might be encouraged to drive wile intoxicated.

Sobriety Checkpoints

Another method proven to effective in decreasing the number of alcohol related crashes is the use of sobriety checkpoints. This idea was first implemented in Scandinavia. With the use of sobriety checkpoints, every driver, sober or intoxicated, has the potential to be stopped by law enforcement officers. The location and times of the checkpoints are determined by the law enforcement agency based on past OWI arrests, alcohol related crashes, and locations of moderate to heavy traffic to contact as many drivers as possible. Either all traffic passing through the checkpoint or vehicles on a pre- established basis (i.e. every tenth vehicle) will be stopped by police officers. After being stopped, the officers will make contact with the driver checking for license, registration, and proof of insurance. If the officer has any suspicions of alcohol impairment, the driver will be directed to another area where roadside OWI tests can be performed (i.e. a Breathalyzer test). The checkpoints become very effective when the results of the checkpoints are made public and the public is made aware of the possibility of random checkpoints in the future.

In 1978, the City of Melbourne, Australia performed such a sobriety checkpoint campaign. Researchers reported that because of the sobriety checkpoints, there were "59 percent fewer nighttime fatalities, 39 percent fewer serious injury crashes, and 30 percent fewer crash-involved drivers with BAC's greater than 0.05 percent *(8)*." In Montgomery County, Maryland, sobriety checkpoints were established between July 1982 and July 1983. During that year, only 7 alcohol related fatal crashes occurred compared to 28 crashes the previous year, a decline of 75 percent *(8)*.

DRIVER FATIGUE AND TRAFFIC ACCIDENTS

Background

Fatigue has been defined as "a progressive decrement in performance which if not arrested, will end in sleep and is related to the level of arousal of the driver *(9)*". A fatigued and tired driver can be as dangerous on the road as a driver operating under the influence of alcohol. National studies have shown that male drivers between the ages of 16 to 25 are most likely to be involved in these types of crashes most of which occur around 2 A.M. *(10)*. Many researchers feel that the accident rates for which fatigue was a contributing factor is underestimated due to the fact that it is hard to define a level of fatigue. A study in New South Wales stated that police officers investigating an accident only reported it as a fatigue related crash if the driver himself or passengers in the car admitted to being tired or falling asleep *(11)*. It is currently unknown how many crashes occur each year because of fatigued drivers.

The Causes of Fatigue

The causes of fatigue vary from person to person making fatigue a difficult concept to define. Dallas Fell conducted a research study in New South Wales which consisted of a questionnaire that was sent out to 1,000 automobile drivers. Quotas were set for sex and age groups to assure that an adequate comparison could be made. Of the 1,000 drivers questioned, 280, or 28 percent, of the drivers admitted to having an accident or a near accident that they felt was due to their own driver fatigue *(11)*.

When the results of the questionnaire were studied, several interesting trends were reported. First, about 25 percent of the 280 drivers were going to or from holiday gatherings, family gatherings, or were on business or social trips, all of which have the potential for being long trips *(11)*. The results indicated some activities that might lead to driver fatigue. About 33 percent of the accident or near accident trips began between 8 P.M. and 6 A.M. compared to 19 percent of the non-accident trips *(11)*. The peak time for fatigue related traffic accidents occur during the night when the environment is conducive to sleep and fatigue related impairment. People traveling during nighttime hours are subjected to dark conditions with little if any driver stimulation from the surrounding environment. The density of traffic at these times also decreases which allows a driver to become less alert. The questionnaire study conducted by Fell indicated that about 20 percent of the 280 crash or near crash drivers had been driving 5 or more hours at the time of the incident. However, the majority of the 280 drivers were involved in an incident within the first two hours *(11)*. However, this last statistic may be misleading. Those crashes or near crashes that occurred within the first two hours of the trip, should be compared to the time when the trip began. If a number of these trips began late at night (i.e. between 11 P.M. and 4 A.M.), the number of crash or near crash incidents that occurred may be more related to the lack of driver stimulation rather than the number of hours driven. Another interesting trend determined from the questionnaire study was when asked if the driver had gotten enough sleep prior to the trip, the answer for more than 50 percent of the subjects was no *(11)*.

A Study on Fatigue Related Accidents and Rest Areas

Many fatigues related studies focus on the trucking industry due to the fact that truck drivers often drive long hours and perform irregular schedules. In 1992, the Federal Highway Administration Office of Motor Carriers supported a study on the adequacy of locations for truckers to stop and rest. The study determined that there was a problem and that the current system was unable to accommodate some 28,400 truckers' nationwide *(12)*. When these 28,400 truckers became tired and needed to stop, there were no parking spaces for them at nearby rest areas. Either these drivers continued to drive even though they were "impaired" or they pulled off the side of the road. Trucks parked on the shoulder of Interstate highways or ramps are a completely different safety issue. These parked trucks add another hazard along the roadside that other drivers could strike should they leave the roadway.

A study performed by William C. Taylor and Nakmoon Sung of Michigan State University compared the number of rest areas, their location with respect to one another, and how the rate of fatigue related accidents was effected by the rest area location. The objective of the study was to relate how single vehicle nighttime crashes involving a heavy truck (truck tractor combination used for transporting property) to current rest area spacing *(12)*. The single vehicle accidents were limited to those that occurred between 10 P.M. and 6 A.M. and involved only one truck. Routes that were studied only consisted of rural freeways since those were likely to be used by trucks traveling long distances. Taylor and Sung used a hazard function to measure the probability of a crash occurring in a predetermined distance interval (i.e. every 10 miles) from the previous rest area. The hazard function was set up to define "an estimate of the potential for a crash in the designated interval, given that a crash will occur and that it has not occurred in a prior interval *(12)*". To do this, average daily truck traffic (ADTT) compiled by the Michigan Department of Transportation Bureau of Transportation Planning was combined with truck crash data from 1994 to 1995. The study included 333 single vehicle accidents that occurred on a total of 1,080 miles of Interstate highway and included all of the major rural Interstates in Michigan *(12)*. All of the rest areas studied were spaced at least 50 miles apart. The majority of the single vehicle truck accidents used in the study occurred between midnight and 8 AM. It was assumed that all of the accidents studied were fatigue related.

By combining the crash data with the rest area locations, Taylor and Sung determined that there was no significant difference in crashes per mile over the first 30 miles between rest areas or the last 20 miles. This indicated that the spacing of the rest areas was a leading factor in the location of the single vehicle accidents and not any roadside or geometric factors *(12)*. After reviewing the research data, it was determined, that the probability of a single vehicle accident occurring on a rural freeway increases when the distance from the preceding rest area exceeds 30 miles and continues to increase to at least the 50 mile mark *(12)*. The results from this research study favor adding more rest areas to the rural Interstate system. A plot of the data found in the study can be seen in Figure 2.

Methods to Reduce Fatigue Related Accidents

Additional Rest Areas

The study by Taylor and Sung along with a study performed by the Federal Highway Administration Office of Motor Carriers supports the addition of more rest areas to the nations highway systems in an effort to combat fatigue related crashes. Building additional rest areas along the nations Interstates would be an expensive solution. Many state Department of Transportation agencies may not have the needed money in their budget for this solution. However, if the government agencies were to provide some aide to the private trucking companies, the trucking industry may be willing to help pay for the construction of these rest areas.

Driver Awareness and Enforcement

Additional driver education in the areas of the causes of fatigue and the effects of driving while may reduce the number of fatigue related accidents. The additional information passed on to the public should stress the following *(11)*:

- Share the driving
- Take longer and more frequent breaks.
- Get a good nights sleep prior to the trip.
- If possible, avoid driving at night, at least during normal sleeping hours.

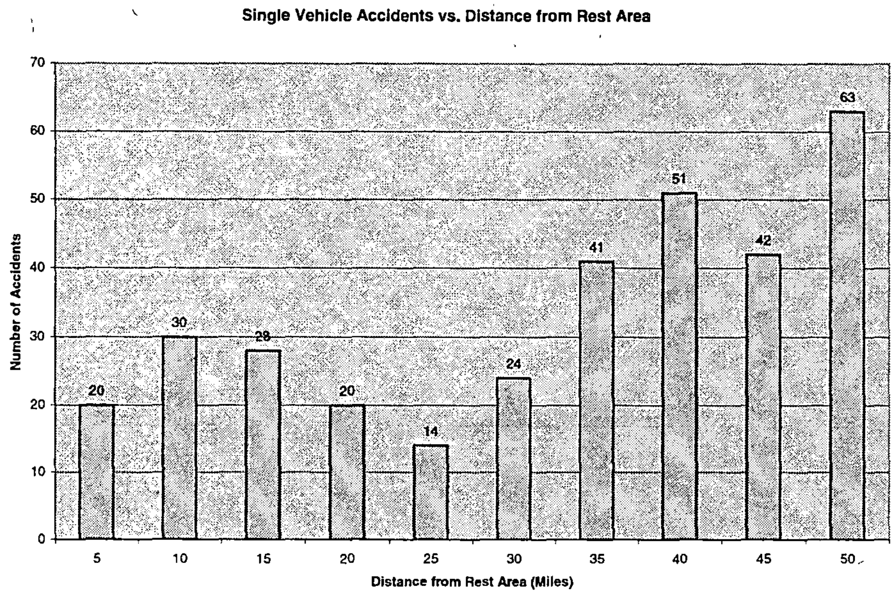

Figure 2-All single vehicle crashes as a function of distance from the previous rest area (12).

Legislation and enforcement dealing with fatigue on day to day travelers will be difficult since there is no way to define whether or not a driver is fatigued. However, enforcement of the trucking industry is possible through logbooks kept by the driver. Laws limiting the amount of time a truck driver is allowed to drive should also be strictly enforced. Fines and any citations should not effect only the driver of the truck but also the trucking company that employs that driver. This will force the trucking industry to take an active role in preventing fatigued driving.

DRIVER DISTRACTIONS DUE TO CELL PHONE USAGE

Background

The use of cellular telephones has increased dramatically over the last several years. In 1995, the rate of new cellular phone subscribers exceeded the birth rate in the United States *(3)*. A number of these people who now have cell phones use them while they are driving. For many people, talking on their cell phone while driving has become part of their job while others do it simply for convenience. Several countries around the world have banned the use of cellular phones in vehicles because it is felt that this added driver distraction can lead to an increased number of crashes. Some of these countries include Australia, Brazil, and Israel. A Canadian Study has shown that people who talk on cell phones while driving are nearly as impaired as drunk drivers *(15)*. In the United States, some states already have adopted regulations regarding the use of cellular phones while driving. In fact, in Iowa a state lawmaker has recently proposed a law that would allow the driver to make calls of up to one minute but calls of over one minute would be considered a criminal offense punishable by a fine of $100 and up to 30 days in jail.

Studies on Cellular Phone Usage and Traffic Accidents

Several studies have been conducted in the area of cellular phone usage while operating a motor vehicle. Donald A. Redelmeirer M.D. and Robert J. Tibshirani P.H.D. conducted one such study. The purpose of their study was to "evaluate potential associations between the use of a cellular telephone and the risk of a motor vehicle collision in real-world circumstances *(3)*". In other words, this case-crossover study attempted to determine whether or not the use of a cellular phone prior to an accident increased the likelihood of the accident occurring as compared to the risk expected by chance only. The study was conducted in Toronto, Canada where there were no current laws restricting cellular telephone usage. The study focused on 742 people who were involved in crashes that resulted in substantial property damage as specified by police. The 742 test subjects were given a short questionnaire concerning personal characteristics and characteristics of the accident. The subjects' cellular telephone bills were obtained, as were the accident reports prepared by the investigating police officers. Using the questionnaires, accident reports, and calls for emergency personal listed on the phone bills, the time of the crash was estimated. A hazard time, a specified period of time prior the traffic accident, was determined for the analysis. Hazard times of 1, 5 10, and 15 minutes were used.

From the analysis, Redelmeirer and Tibshirani determined that 170 of the subjects, or about 24 percent, had used cellular phones in a 10-minute interval prior to the estimated time of the crash. For this hazard interval (10 minutes), a relative risk factor of 4.3 was determined. In other words, the chance of an accident occurring quadrupled when cellular phones were used while driving *(3)*. The risk of an accident also appeared to be inversely proportional to the length of the calls made. Risk factors of 4.8 and 1.3 were determined for calls placed within five minutes before the crash and for calls placed more than 15 minutes before the accident respectively *(3)*. There appeared to be no significant difference in relative risk rates for persons using hand held phones versus hands free phones nor were there any differences found between the "...subjects' age, education, socioeconomic status, or other demographic characteristics *(3)*". The fact that there was no significant difference between the hands free and hands on phones indicated that a leading factor in the crashes was the driver's inattention and not the driver's hands.

The above study attempted to determine how the risk or chance of a crash occurring is effected by the use of cellular phones but does not investigate what actually causes the driver to become involved in a crash. Roberto A. Tokunaga, Toru Hagiwara, Seiichi Kagaya, and Yuki Onodera of Hokkaido University in Japan performed a study on how a driver's reaction time and mental workload are effected by using cellular phones while driving. The study was comprised of 31 subjects ranging in age of 22 to 65 years of age. All of the subjects had at least 3 years of driving experience and were familiar with the use of cellular phones. The tests were conducted on the Central Expressway in Hokkaido Japan. The experimenters drove a leading vehicle at a controlled speed of 90 kilometers per hour (km/h). A test subject, who was instructed to maintain a distance of 50 meters behind the leading car, drove a trailing vehicle. Each vehicle was equipped with devices to measure reaction times and a hands free cellular phone was mounted on the dashboard of the test subject's vehicle. The driver's reaction time was measured using a multiple data recording system, which included a camera mounted in the back window of the subject's vehicle. The driver of the leading car would randomly turn on his warning lights for a period of five seconds. The test subject was instructed to push a bottom on the steering wheel when the warning lights of the leading vehicle were turned on. The camera mounted in the trailing car was placed so that the warning lights of the leading car as well as the test subject could be seen.

The test subjects were called twice during the test via a cellular phone used by the experimenters in the leading vehicle. The test subjects were required to reach to the dashboard mounted cellular phone and hit the "On" button to receive incoming calls, but otherwise their hands were free to perform the driving task. Each of the two calls lasted approximately two minutes in length. These two phone calls were used to measure the mental workload of the driver while talking on the cellular phone. One of the calls was used to evaluate a simple conversation task. This consisted of a simple conversation about the driving conditions and the test section. The second call was more complex. The test subject was asked a series of simple mathematical problems (i.e. "How many is 7+1-1+1+1") *(16)*.

The results of the driver's reaction time analysis showed that driver reaction time increased when the cellular phones were in use. When the cellular phones were not in use, the driver's had an average reaction time of 0.76 seconds *(16)*. When the simple phone conversation took place, the reaction time increased and continued to increase when the complex phone conversation took place *(16)*. This indicated that the use of cellular phones while driving could have a negative impact on the driver's reaction time.

Methods to Reduce Cellular Phone Related Accidents

More research needs to be conducted before a conclusion can be made on the effects of driving while using a cellular telephone. The two studies mentioned both indicate that driver impairment does occur. A large factor in any legislation that may be passed limiting the use of cellular phones, short of banning their use, would be the difficulty of enforcement. For example, the legislation that was proposed in Iowa to limit cell phone calls while driving to a minute or less would be extremely difficult to enforce. In addition, the study by Redelmeirer and Tibshirani indicated that the risk of an accident is greater for calls of shorter duration. Legislation that would require the driver to pull off of the road and be completely stopped before using a cellular phone would be easier to enforce. However, this would increase the number of obstacles along the roadside. This problem was mentioned earlier in the discussion of fatigue and the trucking industry.

CONCLUSIONS

The first topic discussed dealt drivers operating under the influence of alcohol. A study by Olga J. Pendlton, Nancy J. Hatfield, and Ron Bremer, of the Texas Transportation Institute and Texas A & M University suggested that the number of alcohol related fatalities is underestimated. The underestimation is due to the lack of required postmortem BAC tests and the inability of police departments to correctly identify alcohol as a contributing factor to a crash. Studies by R.G. Mortimer, and G. Wilson and R Mitchell both indicated that some drivers could be impaired at BAC's as low as 0.04. This paper also discussed several methods to decrease the number of alcohol related crashes. These methods included more education, graduated drivers license system, license revocation, drinking and driving citations, and sobriety checkpoints.

The second topic discussed was fatigue and fatigue related crashes. A study by Dallas Fells indicated possible sources of fatigue. These sources include business of social trips, family gatherings, and holidays. It also showed that the majority of fatigue related crashes occur between the hours of 8 P.M. and 6A.M. A study performed by William Taylor and Nakmoon Sung suggested that additional rest areas should be built to accommodate the number of truck drivers traveling the nations highways. The study showed that the likelihood of a truck being involved in a fatigue-related crash, increase as the distance from the previous rest area increases. It has also been stated that prevention and enforcement measures dealing with fatigue related driving is difficult since fatigue is a difficult concept to define.

The final topic discussed in this paper is a relatively new safety issue to transportation engineers. The use of cellular phones has become popular in the United States. Studies have shown that using cellular phones while driving may increase the risk of being involved in a crash. A study by Donald A. Redelmeirer M.D. and Robert J. Tibshirani P.H.D. has shown that using a cellular phone while driving a car can increase the chances of a driver being involved in an accident by a factor of 4. This increased risk is especially true for calls of 10 minutes or less in duration. Another study performed by Roberto A. Tokunaga, Toru Hagiwara, Seiichi Kagaya, and Yuki Onodera has shown that using a cellular phone while driving increases the driver's mental workload and reaction time. Some countries currently have laws banning the use of cellular phones while driving while other areas are considering legislation that would put restrictions on cellular phone usage. Further research is needed on the use of cellular phones while driving.

The topics discussed in this paper are only a few of the many factors that lead to driver error. By decreasing the number of errors made by drivers, the crash rate would certainly decrease resulting in numerous lives being saved each year. Research needs to be continued on not only the factors discussed in this paper, but also on the many other factors that lead to driver error and ultimately automobile crashes.

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PAVEMENT MARKINGS AND INCIDENT REDUCTION

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INTRODUCTION

Pavement markings come in many different forms, but the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) defines the two general categories of pavement markings as longitudinal and transverse markings (*1*). A classic example of longitudinal markings would be the center and edgeline markings along a roadway. Traditional transverse markings include crosswalk lines, stop lines, and symbol markings. Pavement markings may also have as many different meanings. Regardless of their meaning, however, pavement markings are used to inform and warn drivers, pedestrians, and bicyclists of local and federal regulations and potentially hazardous locations (*1*). Possibly the greatest advantage to using pavement markings is that they allow drivers to focus their attention where the hazard is most likely to be located; on the roadway (*1*).

There are nearly an unlimited number of situations in which a crash may occur. Pavement markings may best be suited as a means for incident reduction in only a select few. The objective of this report is identify the areas where pavement markings may be most beneficial for crash reduction and to investigate how pavement markings have been used to reduce the number of accidents in these areas. A literature review showed that pavement markings can be effective in the areas of horizontal curvature, turning movements, and pedestrian crosswalks.

The remainder of this report is a review of the limitations of pavement markings and a discussion about their use in each of the following areas: horizontal curvature, turning movements, and pedestrian crosswalks. The discussion in each area also includes general recommendations for use of pavement markings as determined from the results of the reviewed studies. The report concludes with a summary of the limitations and use of pavement markings to increase the motorist and pedestrian safety.

PAVEMENT MARKING LIMITATIONS

Despite versatility and easy recognition of pavement markings by drivers, pavement markings as a means for crash reduction must be carefully considered before the final decision to use is made. First, one must determine if pavement markings are even appropriate in the particular situation. Second, other alternatives should be considered to find the best solution for the problem.

Use of Pavement Markings

Pavement markings are often placed in anticipation of improvement of the roadway safety at known hazardous location. Often these hazardous situations are due to a poor geometric design that violates driver expectations. Pavement markings may be successfully utilized to temporarily increase driver awareness at such locations, but they may not be the best solution. Often, the most effective action would be redesign and reconstruction of the roadway geometry to meet the driver expectations (2). Roadway redesign may not be utilized due to funding restrictions or a roadway redesign may be for the long-term. If reconstruction is not planned for the near future, pavement markings could be part of a temporary solution (along with signing, signing in conjunction with pavement markings, traffic calming, and others). If properly used, pavement markings can be a good short-term solution because of their low installation and maintenance costs. For example, the installation of transverse markings at a sharp horizontal curve was estimated to have a benefit-cost ratio of 45.9 due to a large reduction in crashes that resulted (*3*).

Determining Accident Reduction Factors

When considering improvement alternatives, trying to determine the safety benefit (i.e., predicted reduction in the number of crashes) of pavement markings may be the most difficult part. Al-Masaeid, et al. investigated the safety effectiveness of pavement markings on rural undivided highways (*4*). The stated purpose of the pavement markings were "...to delineate the travel path and improve visibility (*4*)". In their analysis, accident reduction factors were calculated for 100 improved roads in Indiana (*4*). The accident reduction factor was defined as the "...proportion of change in the accident rates from the before to the after period" (*4*). The analysis resulted in accident reduction factors ranging from -0.762 to 0.592 (a negative accident reduction factor indicates an increase in the number of crashes).

One may not expect the inclusion or improvement of pavement markings to increase the accident rates, but this result was evident in several other studies reviewed by Al-Masaeid (*4*). Al-Masaeid, et al. identified that the variability in calculated crash reduction factors may be due to three factors (*4*). First is before-and-after studies may overestimate the crash reduction potential. Second, crash reduction factors are often treated as an absolute value, but actually may have a range of values. Third, previous studies had ignored crash severity and the extent to which improvements were made.

Further analysis by Al-Masaeid, et al. study revealed that sites with accident rates above the mean (defined as hazardous sites) had an average crash reduction of 13.5 percent (*4*). This is a significant accident reduction benefit when compared to the average crash reduction factor for the entire study (-3.4 percent). The increase in the crash reduction factors at non-hazardous sites may be due to the random nature of accidents. At a site with a low crash rate, one more crash in the after period than in the before period could result in a high negative crash reduction factor. The additional crash may just be a random occurrence, rather than the negative impact from the inclusion of pavement markings.

HORIZONTAL CURVATURE

A sharp horizontal curve is a perfect example of where redesign and reconstruction of the roadway would be the preferred alternative, but pavement markings can produce beneficial results if it is not feasible. Retting, et al. reported that excessive speeds are a significant factor in the number of crashes on curves and the severity of these crashes (*2*). Therefore, Retting, et al. proposed that pavement markings should be used to reduce the vehicle speed on the tangent section prior to the points of horizontal curvature (*2*).

To effectively reduce vehicle speeds on the tangents, safety engineers must understand the proper use traffic control devices and then consistently use traffic control devices to meet driver expectations. For example, the use of curve warning signs before moderately sharp horizontal curves will allow drivers to assume that the next curve warning sign will be followed be by a moderately sharp horizontal curve. If instead, the curve warning sign precedes a significantly sharp horizontal curve, the driver may not decelerate sufficiently to safely navigate the curve. Special warning devices, such as pavement markings, could be used to inform the driver of the unusually sharp curve.

Two studies that used innovative pavement marking design to alert drivers of an unusually sharp horizontal curve are discussed in the following paragraphs. The first study report used left curve arrow and text to warn drivers of the approach curve (*2*). The second report used transverse pavement markings to give the driver the appearance of acceleration on the curve approach (*3*).

Left Curve Arrow and Text

In Virginia, Retting, et al identified a sharp left curve after a long tangent section (*2*). The traveled way width was reported as approximately twenty feet, with one lane of traffic in each direction. The posted speed limit of the roadway was 35 miles per hour (mph) and the posted speed limit for the curve was 15 mph.

To warn the drivers of the sharp curve, an experimental pavement marking was placed 220 feet prior to the curve. The experimental pavement marking included the word "SLOW" in eight-foot white letters and an eight-foot left curve arrow (See Figure 1). Speed measurements were taken at two locations, 90 feet and 650 feet prior to the curve. Speed measurements at both sites were done before the placement of the pavement markings and two weeks after the pavement marking was installed.

The speed results for were reported for three different times of day: daytime (10:30 a.m. to 5:00 p.m.), evening (9:00 p.m. to midnight), and late night (midnight to 3:00 a.m.). The results for all three time periods are summarized in Table 1. As can be seen from Table 1, the mean speed at the location just prior to the curve dropped for all three time periods. For the location just prior to the curve, Table 1 also shows a reduction in the percentage of vehicles traveling faster than 35 mph, 40 mph, and 45 mph; regardless of whether the upstream site mean speed increased or decreased. Retting, et al. concluded that the experimental pavement marking resulted in a net speed decrease for all time periods (*2*). A seven percent decrease in the mean speed was reported for the daytime period (three percent decrease near the marking plus a four percent increase upstream). The evening time period had a two percent decrease in the mean speed (five percent decrease near the marking minus a three percent decrease upstream). Finally, a seven percent decrease in the mean speed was reported for the late night period (ten percent decrease near the marking minus a three percent decrease upstream).

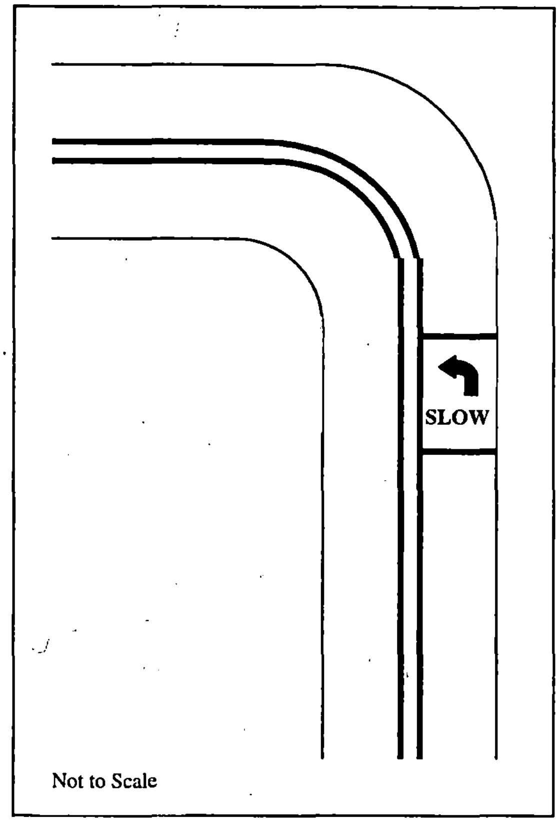

Figure 1. Arrow and Text Placement (*2*)

Table 1. Speed Measurement Results (*2*)

Time of Day	Distance Prior to Curve	Before or After	Mean Speed (mph)	Percentage > 35 mph	Percentage > 40 mph	Percentage > 45 mph
Daytime	90 feet	Before	34.3	41	9	0.8
		After	33.2	34	4	0.2
	650 feet	Before	40.2	85	54	18
		After	41.7	91	66	26
Evening	90 feet	Before	33.5	36	9	2.0
		After	31.9	25	3	1.2
	650 feet	Before	40.1	84	49	14
		After	39.1	78	39	14
Late Night	90 feet	Before	35.1	49	19	2.0
		After	31.7	22	2	1.6
	650 feet	Before	40.3	87	63	20
		After	39.1	98	74	28

There were two shortcomings to this research study. First, this experiment was performed at one site. Secondly, a follow-up study was not performed to determine whether the marking had any long-term impact on mean speeds.

Transverse Pavement Markings

In a study similar to the one described previously, Agent also used experimental pavement markings to decrease vehicle speeds before entering a sharp horizontal curve (*3*). A high-accident location site was located on US-60 in

Meade County, Kentucky. This curve had a total of 48 accidents in the six years pervious to the installation of the pavement markings, 46 of which had been an eastbound vehicle that left the roadway or crossed the centerline and collided with a westbound vehicle. Thirty-six of the accidents listed excessive speed as a contributing factor. Failure of conventional signing and marking led authorities to try a new approach until reconstruction could be performed. The posted speed limit for the roadway was 55 mph and the horizontal curve had a rated speed limit of 35 mph.

To reduce vehicle speeds, experimental transverse markings were placed on the eastbound approach lane. The transverse markings were spaced such that a driver had the appearance of acceleration towards the curve. This was accomplished by placing the transverse markings at an ever-closer spacing. Drivers that slowed to acceptable speeds would see the transverse markings at a constant rate, and therefore would not experience the sensation of acceleration.

Due to resurfacing of the roadway, researchers were only able to gather crash data for one year after placement of the pavement markings. In addition to the crash data, speed data was gathered prior to, one week following, and six months after the striping of the roadway. Speed measurements were taken at the beginning and end of the transverse markings. Table 2 summarizes these speed measurements.

As can be seen from Table 2, the transverse markings lowered the mean speeds in both the daytime and nighttime conditions. However, the six-month follow-up did show that speeds increased slightly when compared to the measurements taken immediately after striping, but these speeds were still lower than the prior condition. In addition to the decrease in the mean speed, the speed reduction through the pavement markings and the percentage of vehicles exceeding the posted speed limit show similar results.

In the year after the placement of the transverse markings there were only three reported accidents. This is a significant decrease over the average eight accidents per year in the six years prior to striping (*3*). Of the three accidents that occurred after striping, two reported alcohol as a contributing factor and speed was only mentioned as a factor for one accident The major shortcoming to this experiment was that is was only performed at one location.

Table 2: Effect of Transverse Markings on Speeds (*3*)

Time of Day	Before or After	Mean Speed	Average Speed Reduction through Markings	Percentage > 35 mph
Daytime	Prior to	41.3 mph	8.5 mph	90
	One Week After	33.9 mph	15.3 mph	40
	Six Months After	34.9 mph	12.3 mph	Not Reported
Nighttime	Prior to	40.5 mph	2.4 mph	81
	One Week After	35.1 mph	9.3 mph	43
	Six Months After	39.1 mph	6.8 mph	68

TURNING MOVEMENTS

Various studies have shown that rear-end collisions account for 18 to 23 percent of all crashes, and up to 20 percent of all injury crashes (*5*). Ten percent of rear-end collisions involved a vehicle making or preparing to make a turning movement (*5*). With rear-end collisions such a large percentage of all crashes, it is easy to see why researchers are trying to reduce this type of crash.

Much like the sharp horizontal curves, a violation of driver expectancy can be a reason for a large number of accidents involving turning movements. It is not the roadway geometry that is violating driver expectations, but the action of other drivers. When drivers slow or come to a stop to make a turning maneuver, the following driver may be unprepared for the vehicle in their path and the result can be a rear-end collision. This is especially true were sight distances are limited. In such circumstances, pavement markings can possibly be used to alert drivers of locations with a large number of turning vehicles.

The following studies review the results of pavement markings about the safety of turning movements. The first report used pavement markings to alert drivers of vehicles turning right into a commercial driveway (*5*). The second study discusses pavement markings at freeway lane drop exits (*6*).

Commercial Driveway Entrances

Retting, et al. identified several key driver factors that led to rear-end collisions at commercial driveway entrances (*5*). These factors included inattentive drivers, short following distances, and a low use of the right turn signal. To offset these factors, the authors identified two methods of remediation. The preferred method was the construction of right turn lanes. In some instances, this method is not feasible due to right-of-way restrictions and high construction costs. The second method (which was the focus of the study) is the use of pavement markings to alert drivers at locations with a high incident rate.

The experiment included the addition of the standard through arrow accompanied by a right-turn arrow at four commercial driveways (*5*). The driveways were to a bank, a post office, a shopping center, and a fast food restaurant; all of which are located in Virginia.

To record the data, a concealed camera was placed near each business entrance. When the videotapes were later reviewed, a potential conflict was recorded each time a vehicle making a right turn was followed by another vehicle by less than four car lengths on low speed roadways. For high speed roadways, the trailing vehicle had to be less than five car lengths to be included in the study sample. Through observation of the trailing vehicle taillights, each potential conflict was classified into one of three categories. The category called *abrupt braking* was considered to be a rear-end conflict. In order for the trailing vehicle to be classified in the *abrupt braking* category, a rapid rate of deceleration and one of the following criteria had to be met: squealing of the tires, visible rise and fall of the vehicle's rear end, or pulling of the vehicle to the left or right.

The research showed that the pavement markings were effective in increasing the average car following distance. At the post office, bank, and fast food site, there was a small decrease in the number of potential conflicts and a large decrease in the number of rear-end conflicts. Unfortunately, the number of potential conflicts and rear-end conflicts actually increased at the shopping center. The results of the Retting, et al. study may have been more conclusive if enough time was available to collect crash data versus conflict data (*5*). The results are based upon conflict data, and previous studies do show that a reduction in crashes does correlate to conflict reduction (*5*).

Freeway Lane Drop Exits

Freeway lane drop exits are defined as a location where one or more freeway lanes are eliminated at an exit (*6*). Such locations can cause confusion when the driver does not expect the lane to exit; leading to an increased number of lane changes and erratic maneuvers (*6*). In a study sponsored by the Texas Department of Transportation, Fitzpatrick et al. (*6*) studied the effects of lane drop markings at three locations. Lane drop markings are larger-width lane striping that starts approximately 0.5 miles before the gore (the area between the through lanes and an exit ramp (*7*)) along with a solid, eight inch wide white line for 300 feet before the gore. At the three chosen locations, the exiting lane existed for at least a mile before the exit and there was minimal influence from either nearby entrance ramps or poor geometric designs. In addition to the lane drop markings, white left turn arrows were placed in the dropped lane.

Through the use of video cameras, 1200 to 1900 feet of roadway was observed. At each experimental site, the location and number of lane changes and erratic maneuvers were recorded for several days. An erratic maneuver was defined where a motorist changed lanes through the gore; changed two or more lanes; swerved between lanes and the shoulder; or drove over the solid white line. Before-and-after traffic counts showed no discrepancies in the before-and-after traffic volumes at the locations. The Texas Department of Transportation provided information to verify that there was no nearby construction or nonrecurring congestion that could skew the data.

Overall, the results of the study showed that there was a significant decrease (from 6 to 31 percent decrease) in the number of lane changes at all locations. In addition, two of the locations had a larger percentage of the motorists in the after condition exiting the dropped lane further upstream than in the before condition. The decrease in erratic maneuvers was even more statistically significant, with a decrease in the number of erratic maneuvers ranging from 18 to 40 percent. In the 300 feet closest to the gore, the results were even more conclusive. The percentage decrease in the number of lane changes was from 42 to 64 percent, and the drop in erratic maneuvers was 29 to 50 percent decrease. These decreases can be critical to driver safety because the motorist is no longer having to perform complex maneuvers (i.e., speed and/or lane changes) while the demand on the mental workload is high. If used consistently, lane drop markings and left-turn arrows may be able to perform even better once a driver expectancy is established.

PEDESTRIAN CROSSWALKS

Retting, et al. reported that nearly 100,000 pedestrians are injured every year in motor vehicle crashes in the United States (*8*). Pedestrians are encouraged to cross streets at intersections, which are often equipped with crosswalk markings and/or traffic signals (*8*). Despite this, intersections account for about 39 percent of nonfatal pedestrian injuries and 18 percent of fatalities (*8*). The best method to increase intersection safety is to physically separate the pedestrian and motorist (i.e., pedestrian tunnels or pedestrian bridges), but these alternatives are expensive to construct and may not be used by the pedestrians (*8*).

Retting et al. (*8*) reported that the use of messages painted in the crosswalks ("EXTEND HAND TO CROSS") increased the percentage of pedestrians signaling to motorists that they intend to cross the street. Because of this, Retting et al. (*8*) experimented with the use of special signs and pavement markings to alert pedestrians to turning vehicles. The experimental signs had the message "LOOK FOR TURNING VEHICLES" and the message painted in the crosswalk was "WATCH TURNING VEHICLES".

The experiment was conducted at three intersections that had a large number of pedestrians and a high daily traffic volume. At the first site, the signs were installed first and the pavement markings were added later. The second site had the pavement markings placed first with the signs added at a later time. The third site had both the signs and pavement markings installed at the same time.

As evident in Table 3, all three methods decreased the average number of conflicts for every 100 pedestrians. At the site where the messages were installed first followed by the signs, there was a slight increase in the average number of conflicts after the signs were installed. Yet, this was still a 61 percent reduction in the number of conflicts than in the before condition. Similarly, the number of pedestrians looking for turning vehicles increased for all three experiments (See Table 4). In the follow-up studies, both the average number of conflicts and the percentage of pedestrians looking for turning vehicles were as good as or had slightly decreased when compared to the results immediately following installation. In either case, the follow-up study showed a significant safety benefit over the before condition.

Table 3: Conflicts per 100 Pedestrians (*8*)

Description	Before Study	Sign Only	Paint Only	Sign and Paint	Follow-Up Study
Signs followed by Painted Messages	2.7	0.6	---- NA ----	0	0
Painted Messages followed by Signs	3.1	---- NA ----	0.7	1.2	0
Both Added at the Same Time	2.5	---- NA ----	---- NA ----	0	0

Retting, et al. were unable to observe actual crash data to determine the effectiveness of the pavement markings and signs installed, but one can determine that the reduced number of conflicts and increased pedestrian awareness should result in reduced number of accidents over time (*8*). As discussed in the section PAVEMENT MARKING LIMITATIONS, this method to increase the pedestrian safety may prove to be especially effective when used at locations that are known to a high accident rate

Table 4: Percentage of Pedestrians Looking for Turning Vehicles (*8*)

Description	Before Study	Sign Only	Paint Only	Sign and Paint	Follow-Up Study
Signs followed by Painted Messages	82	92	---- NA ----	97	97
Painted Messages followed by Signs	85	---- NA ----	95	97	95
Both Added at the Same Time	85	---- NA ----	---- NA ----	96	93

SUMMARY

Pavement markings may not be applicable at all locations that are known to be hazardous, but their use may help increase the safety of locations. At sharp horizontal curves, experimental pavement marking patterns have been shown to decrease the entrance speed of vehicles and reduce the number of accidents at these locations. At freeway lane drops, pavement markings have been shown to decrease the number of erratic maneuvers and lane changes while at the same time reducing the mental workload for the motorists. Pavement markings were also shown to increase the awareness of motorists to turning vehicles and to increase the number of pedestrians looking for turning vehicles while crossing the street. Both of these experiments resulted in fewer conflicts, which could mean fewer crashes in the future.

When evaluating the use of pavement markings, one should remember that crashes are random occurrences and thereby may result in mixed results. The key to cost effective use of pavement markings is to first identify high crash locations and then determine if pavement markings may prove beneficial. This methodology should produce better results than systematically using pavement markings in all hazardous locations.

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REPORT ON THE TECHNICAL ISSUES SURROUNDING THE DEMONSTRATION OF THE SIGN MANAGEMENT AND RETROREFLECTIVITY TRACKING SYSTEM (SMARTS) TECHNOLOGY -- SUMMER OF 2000

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INTRODUCTION

The Midwestern Resource Center of the Federal Highway Administration (FHWA) entered into a contract with the Transportation Infrastructure Center (TIC) at the University of Missouri-Columbia (UM-C) to assist them in demonstrating a new technology for measuring the retroreflectivity of highway signs while traveling at regular highway speeds. The goal was to schedule demonstrations in ten upper Midwest states and complete the weeklong demonstrations over the summer of 2000. FHWA wanted to demonstrate this technology as quickly as possible. As the director of the TIC, I set up the schedule for the visits with the ten State Departments of Transportation and the Division Offices of the FHWA. Mr. Joseph Turner, a senior Civil Engineering student at UM-C, drove this van throughout the ten states and was an active participant in the demonstrations. He drove the research van, operated the sophisticated equipment and related computers, trouble shot vehicle, equipment, computer and software problems when they occurred in the field, and became so well versed in the technology that he was able to present the technology by himself at several sites including: FHWA's ITS engineers in Kansas City, engineers in Kenosha County, Wisconsin and The National Rural ITS meeting in Branson, Missouri. What follows is Joe's summary report.

Charles J. Nemmers, P.E.

SMARTS REPORT

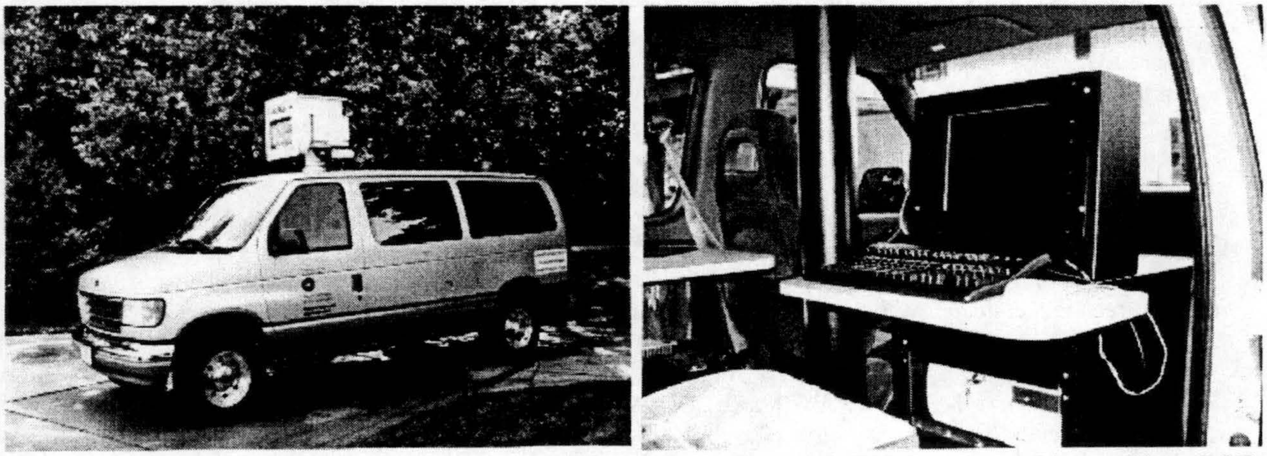
The nighttime visibility of highway signs is essential for the safety, comfort and guidance of the motoring public. Recent highway crash statistics show that while 25% of the travel is at night nearly 55% of the fatalities occur at night. Better highway sign visibility can help to reduce these kinds of statistics. In 1954 the MUTCD required that signs needed to be reflectorized (or illuminated). An ASTM specification has been developed for the reflectivity of new signs and now the U.S. Congress in 1993 established a requirement for in service retro-reflectivity of signs.

Since the retro-reflectivity (nighttime visibility) of signs deteriorates over time due to the suns rays and other environmental factors, a method for measurement of the in place retro-reflectivity needed to be developed. The Federal Highway Administration in cooperation with 34 state DOT's and the US Naval Research Labs have developed a Sign Management and Retro-reflectivity Tracking System (SMARTS). This system has been designed to fit into a van and be driven at highway speeds while making the necessary measurements. This results in accurate digitized readings collected without interference to traffic and equally important does it very quickly and safely for the worker.

During the summer of 2000, the FHWA's Sign Management and Retroreflectivity Tracking System (SMARTS) vans were presented for demonstration to traffic officials from various agencies across the country. In the East, Southeast and West the FHWA used their own staff to demonstrate the SMARTS van, however the Midwestern Region entered into a contract with the University of Missouri-Columbia and I was selected as the student whose summer job would be to cover the Midwest helping to demonstrate this leading edge technology. Most of the participants at the demonstrations were State Department of Transportation officials and related technical people. This report will highlight the SMARTS van and its equipment's operation, its behavior under varying conditions; as well as addressing the observations, questions, and concerns about the system as raised by those who were introduced to the SMARTS system this summer. *Note that throughout this report, the word "camera" will be used to refer to the data collecting equipment on the roof of the vehicle which includes the turret, laser range finder, strobe light, two cameras and related computer interfacing hardware.*

PROJECT OVERVIEW:

The dates and locations the SMARTS van visited during the summer of 2000 are as follows:
April 15: Missouri Traffic Conference – Columbia, MO
May 15-19: Illinois Department of Transportation (District 6 and Headquarters) – Springfield, IL
May 22: FHWA Midwestern Resource Center – Olympia Fields, IL
June 5-6: Iowa Department of Transportation (District 6) – Cedar Rapids, IA
June 8-9: Iowa Department of Transportation (District 1 and Headquarters) – Ames, IA
June 12-13: Wisconsin County Engineers Association Conference – LaCrosse, WI
June 14: Wisconsin Department of Transportation (District 6) – Eau Claire, WI
June 15: Wisconsin Department of Transportation (District 3) – Green Bay, WI
June 16: Wisconsin Department of Transportation (District 1 and Headquarters) – Madison, WI
June 19-21: FHWA ATSSA Conference – Kansas City, MO
June 26-27: Michigan Department of Transportation (Headquarters) – Lansing, MI
June 28: Michigan Department of Transportation (Detroit Metro District) – Southfield, MI
July 7: Kenosha County Highway Department – Kenosha, WI
July 10: 3M – Cottage Grove, MN
July 11: Minnesota Department of Transportation (District 4) – Morris, MN
July 12-14: Minnesota Department of Transportation (Metro Office) – Minneapolis/St. Paul, MN
July 17-18: Indiana Department of Transportation (Greenfield District) – Indianapolis, IN
July 19: Indiana Department of Transportation (Seymour District) – Bloomington, IN
July 20: Indiana Department of Transportation (Fort Wayne District) – Fort Wayne, IN
July 24: Nebraska Department of Roads (District 1) – Lincoln, NE
July 25: Nebraska Department of Roads (District 4) – Grand Island, NE
July 26: Nebraska Department of Roads (District 6) – North Platte, NE
July 31-Aug.1: Ohio Department of Transportation (District 6 & Headquarters) – Columbus, OH
August 2: Ohio Department of Transportation (District 4) – Boston Heights, OH
August 3: Ohio Department of Transportation (District 1) – Bowling Green, OH
August 7-8: Kansas Department of Transportation (District 1) – Topeka, KS
August 9: Kansas Department of Transportation (District 5) – Wichita, KS
August 14-16: National ITS Rural "RATTS " Conference – Branson, MO



This is the vehicle and equipment that was used for these presentations and demonstrations: (License Plate: DOT 40911)

During this project, the van traveled a total of 14,103 miles. Approximately 470 hours were devoted during the course of the summer. 280 people were introduced to the technology, and a significant percentage of those actually tried operating the equipment themselves. An estimated 5800 signs were photographed and analyzed during this project (rough estimate assuming 200 signs each demonstration day for 29 demonstration days).

At each visit, a presentation was conducted on the need for visible highway signs, aspects of this relating to the MUTCD, federal legislation, the history of sign visibility technology, and detail on the science of retroreflectivity and its measurement. These presentations ended with a discussion of the SMARTS van, its operation and most importantly, showing how the data to be collected would apply to state or local jurisdictions' Sign Management Programs. Following this classroom presentation, the SMARTS van itself was showcased: the van and its computers, cameras, lasers etc. Following this introduction the remainder of the week was spent taking people out in the van to actually collect data using the SMARTS van. At the end of each run (usually an hour or so) we would download the data from the onboard computer to a CD-ROM and give it to state (or local) engineers who requested a copy. In nearly all states we spent time measuring signs on many different types of highways within the state (rural, urban, arterial, highway, Interstate, etc.). As we collected data and demonstrated the equipment, I noted problems encountered, solutions developed, opinions and concerns expressed and, in general, kept track of what was said. What follows is a detailed listing of these observations. They are presented so as to contribute to improving the implementation of this state-of-the-art technology. In general the van, equipment, cameras and computers all worked very well. Making the computers more robust to withstand the tough operating environment (rough roads and extreme temperatures) is strongly suggested.

OPERATOR CONTROL:

During the field demonstrations, participants were given the opportunity to ride in the second seat of the van situated behind a computer screen and operate a computer mouse to track the highway signs that were displayed on the computer screen. The sign was being "videoed" by the camera system on the roof of the van as the van moved down the highway. The most common difficulty encountered by those operating the equipment for the first time was the difficulty in effectively controlling the computer's mouse so as to control the motion of the camera. This was especially true when the road being traveled upon was rough. A common complaint was the sensitivity setting of the computer's mouse was too high. However, the mouse speed was set to the slowest possible setting in the Windows© control panel. Also, it was noted by many that the motion of the camera was also too sensitive to the motion of the mouse. A slight movement of the mouse caused a radical motion of the camera.

Many people also had difficulty understanding how the motion of the camera was governed by the position of the pointer on the screen (the "crosshair") relative to the center green square in the camera's field of view. During operation, many participants would overcompensate in trying to redirect the camera's motion when the camera would drift from its front position. Many participants had various suggestions for a better type of pointer controlling device instead of a standard computer mouse ranging from a joystick to a track-and-ball system, to even a "steering wheel" type of control. Some investigation into a more user-friendly control system may be warranted.

WEATHER CONDITIONS AND EQUIPMENT SETTINGS

As expected, the effective operation of the equipment is extremely weather and lighting condition dependent. The setting of the color shutter is highly important for obtaining quality data. During periods of partly cloudy skies, and even mostly cloudy skies, the amount of sunlight present can change dramatically and frequently. It was observed that the color shutter setting must be continually monitored during these situations, and adjusted as necessary. It was discovered that the best way to check this setting of the **color camera**, during collection of data, is to observe the clarity and brightness of the **black-and-white** picture that appears for a brief moment on the monitor as the strobe flashes and the pictures and data are being recorded. A clear and legible black-and-white photograph is a reliable indicator of the best setting for the color shutter. Of course, this is assuming that the sign photographed for this observation has good retroreflectivity. Therefore, for best results, it is suggested that there be a particular sign that should accompany the van solely for the purpose of setting the color shutter prior to actual road sign data collection. This "setting" of the color shutter should be made by setting the sign at the prescribed 200 foot distance from the camera, manually taking the picture, observing the clarity of the black-and-white picture along with the calculated percent dynamic range, and adjusting the color shutter setting accordingly. For best results, a sign with areas of both good and poor retroreflectivity would be desired. Further, since the percent dynamic range and the black-and-white photograph are indicators of the quality of the data, it would be desired that both be displayed somewhere on the data collection screen, so that both could be continuously monitored during data collection. This would enable the color shutter to be continuously set at the appropriate level for the lighting conditions present during the collection of data. The ability to do this would result in fewer elements of incomplete or poor quality data. Thus, it would also result in less time spent relocating signs that were not photographed and analyzed to the best quality. *Note that* *"quality" here refers to the reliability of the data obtained, based on image quality produced by the equipment and certain related numerical output, not the retroreflective properties of signs that are analyzed.*

Since the setting of the color shutter is highly important to the reliability of the data, it would be desired that there be finer increments of change between one setting and an adjacent setting. It was observed that in many instances, a particular color shutter setting was too low, but the next highest setting was too high, making it impossible to set the color shutter at an optimum level. This mainly occurred when cloud cover was not constant, and lighting conditions were changing continuously.

In addition to the lighting dependent conditions noted above, is should be noted that failure to adjust the color shutter setting for the outdoor lighting conditions can result in undesired behaviors by the system. When not properly set, the automatic tracking system will not function properly. Manual control of the camera is required when this occurs. Also, pictures that are collected will tend to be incomplete, meaning that the picture will not be centered on the sign. In many cases, only half or three-quarters of a sign may be photographed. This is most common with the black-and-white photograph. Another occurrence related to improper setting of the color shutter is that the system may fail to record both photographs. One picture, but not the other, will be stored in memory.

There are a few other factors that occasionally interfere with the operation of the SMARTS system. For road signs next to traffic signals, such as street name signs located between signals, collecting data from those signs can be difficult because of the light emitted from the signal itself. The automatic tracking of the system tends to lock onto the illuminated traffic signal. Also, in some instances there may be other objects on the roadway that are brightly colored that "attract" the SMARTS system. Some of these objects include trailers on trucks, advertising billboards, towers, water towers, guardrails, and even the retroreflective pavement markings on the road. Also, in areas where patches of trees are along the road, the system will also sometimes direct itself toward the edges of treelines.

The SMARTS system was also operated, during one demonstration, during a heavy rainfall. The system's ability to track a road sign and take a picture is minimally affected. The reliability of the data and the quality of the pictures taken during these conditions are uncertain however. The primary problem encountered when using the equipment during such weather conditions is water on the front of the camera mount, which obstructs the camera's view. In general we tried to avoid collecting data in the rain, because even if the readings were accurate (of which we are not sure) the clarity of the picture would be severely compromised.

COMPUTER

The computer that controls the SMARTS system appears to be developing a graphics-processing problem. When the computer is operating for long periods of time, the Windows© graphics become distorted, especially the icons on the SMARTS programs. One area that is a particular problem is the area of the sign evaluation program that displays the histogram. When these graphics distortions occur, it is usually the histogram that distorts first. This, and much more severe problems, seems to occur more readily when the temperature in the cabin of the van rises above standard room temperature, and/or outside moisture is present in the van. This is especially a problem during the summer months, when the temperature inside the van can reach extremely high levels, and humidity levels are high. To counteract this, it is important to make sure the cabin temperature is as close to normal room temperature as possible, and the air inside the vehicle is dry, **before** starting the computer. It is just as important to maintain this controlled climate inside the van **during operation** of the SMARTS system. The more severe computer problems related to temperature and humidity are lock-ups, crashes, loss of data, operating errors, booting difficulties, etc.

Because the computer which operates the SMARTS system is part of a moving vehicle, and since any object in a moving vehicle is subject to erratic, sometimes violent motion (especially on a bumpy road), the computer is subject to periodic disruptions in its operation which can compromise data collection and processing. Because of this, during actual collection of data on the road, it is highly important to **save the data often at regular time intervals.** This is to prevent a significant loss of retroreflectivity data should a computer problem occur. In several instances throughout this demonstration project, varying amounts of data were lost when a malfunction would occur, such as a lock-up or a frequent volatile memory storage problem that this particular computer was prone to. On occasion during operation, the computer would display a message indicating that there was an inability to store data to certain locations of the random access memory. To correct the problem, whatever data not saved to the hard disk that was in the RAM had to be purged. This could present a significant problem if large amounts of sign data were not written to the hard disk, but rather existed only in the RAM.

Many times throughout the project, those introduced to the technology requested a personal copy of the data collected during the field demonstrations. A copy, along with the sign evaluation software, was provided to all who requested it without problem. However, a couple of things were noticed. First, while the software and data are compatible with all versions of Windows© encountered during this project ('98, NT), **computers that are connected to certain networks** (such as those for some state departments of transportation) **may reject the installation of the software.** It is believed that this is due to network-wide safeguards, which prevent such software installation. While this does not directly relate to the SMARTS program itself, this fact should be made clear to anyone who would wish to use the software on such a computer. Second, while the sign evaluation software may install properly on a computer, the data itself may not be able to be read by the program when accessed directly from the compact disc. This problem is evident when the program window will appear on the screen, but no pictures or retroreflectivity data appear or can be accessed. If this occurs, the user should **copy** all sign management databases, along with all accompanying image files, directly **to** a directory on the **computer's hard disk** from the CD-ROM, and access the data directly from the hard drive. In all cases where this problem occurred, this corrected the problem, and the software operated correctly.

PARTICIPANT INPUT

The following items are the most frequently noted observations/comments/suggestions noted by those who participated in the summer 2000 demonstrations of the SMARTS technology:

- A device, similar to the SMARTS van, that could be developed to determine the dimensions (size) of a road sign in the same manner as the SMARTS van determines the retroreflectivity of road signs would be useful.

- The measured values for the coefficient of reflectivity were too high to be useful for any significant application without appropriate correlation factors. Those who viewed the equipment could not make sense of the obtained values. Measured values will have to correspond to the reflective guidelines that are being established to be useful.

- A type of algorithm that could determine the exact Global Positioning System (GPS) coordinates of the road sign itself would be more useful in sign management than the GPS coordinates of the rear of the vehicle.

- The van and its equipment were impressive to almost all of the participants, but the cost of the equipment would make it prohibitive for most agencies. If only a sample or a small percentage of signs would be needed for an actual application, the hand-held retroreflectometers may be adequate enough, and less expensive than the mobile unit.

- It would be ideal if the data obtained using the SMARTS's software could automatically download its recorded data to one of the more widely used sign management programs (i.e. Cartegraph, SignView), so that the SMARTS's software could be used to update existing sign management databases.

- When the system records a picture, the black-and-white picture that is recorded is displayed in the viewing area for about a second. This was a disturbance to one operator, especially when attempting to take pictures in an area with a large density of signs. Most however, considered this as simply an indication that the picture was taken. If this momentary still picture were not displayed, some other indicator that the picture was taken would be necessary, since most who operate the system do not look for the flash of light on the sign when the picture is taken.

- The tracking system does not move the mount fast enough for side images of signs located on the right side of the roadway.

- The GPS location (given in degrees latitude and longitude) is okay, especially with its high level of accuracy. However, the datum in which these coordinates are recorded needs to be known so that the collection of data can be useful for other applications.

- It would be desired that the system take a picture even if the automatic tracking were lost. This could be accomplished by having the system calculate the last tracking angle, distance, and speed of the vehicle, to determine where and when to automatically take the photograph.

- It would be most beneficial if the van could be developed to also incorporate a system for measuring the reflectivity of pavement markings by recording a reading every 1/10 of a mile so that it could be determined if remarking were necessary. It was suggested that there be a type of arm that could extend from the side of the van that would take these readings as the vehicle travels down the highway.

- One concern that was voiced was that the SMARTS system is recording data at an angle that is different from what motorists will see on the road because of the fact that the fact that the recording unit is higher than the motorists' eye level.

- For the van itself, a type of sensor on the vehicle that would warn the driver when he/she is getting too close to an object. It can sometimes be difficult to see when backing up, simply because of the size and length of the vehicle. This would be most useful when parking in a parking lot where vehicles are parked close together, and other places where space is limited. It can sometimes be difficult to pull into and out of a parking space, especially when there is only one space available between two cars, and cars are located behind.

- The green square that represents the camera's center of view on the monitor is sometimes too lightweight or too light in color to see for some of the participants.

- Would desire that the evaluation software might be able to sort signs by level of R_a, so that actual useful query of a database could be performed. This would be so that a person reviewing the data would not have to scroll through thousands of signs to locate those that are unacceptable. This would require that the standards be programmed into the software, and the program may be able to sort and list only those signs that measure unacceptable.

- It is desired that there be some way of making the sign evaluation software compatible with existing Sign Inventory Management Systems (SIMS). It was stated that GPS coordinates do not provide a meaningful location of a sign to people. Also, most agencies do not use GPS in their SIMS. It would be desired that the programming might be able to use the GPS data, along with existing sign location data, do identify a given sign using GPS and the highway that the van is collecting data on.

- A type of camera stabilizing system for the turret may be beneficial. This may help provide for better quality pictures. Oftentimes, on bumpy roads, the black-and-white picture may be incomplete; only part of a sign may be photographed. This seems to be due to the vehicle hitting a bump in the road as the black-and-white picture is being taken.

- Since some people, during demonstrations (and possibly during actual use) may want to know the reflectivity of the sign immediately, an output area displaying this on the data collection screen may be useful.

- A type of rigid-body housing for the turret would be highly beneficial in protecting the turret during transport, when the system is not in use but the vehicle is in transit. This housing could be something as simple as a custom-fit durable "box" that could be pinned to the roof of the vehicle when the SMARTS equipment is not in operation. This would help protect the turret from potential damage caused by bugs, birds, road debris (rocks, flying asphalt), hail, and rain.

- It should also be noted that one person who operated the equipment during a demonstration experienced mild motion sickness from operating the system. This was most likely due to the constant, erratic movement of the picture on the monitor, and the motion of the vehicle. As when operating any computer, it may be wise to recommend that persons should not operate the system for more than about an hour continuously.

COLLISION AVOIDANCE SYSTEMS

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ABSTRACT

The topics discussed in the paper include the possible cause and type of crashes and the available technology to avert or reduce crashes. An attempt is made to explain the relevance and importance of the collision avoidance systems with respect to the existing conditions. Possible advantages and disadvantages of the collision avoidance systems are also discussed.

INTRODUCTION

Transportation is one of the most important economic activities of any country. Among the various forms of transport, road transport is one of the most popular means of transportation. Transportation has an element of danger attached to it in the form of vehicle crashes. Road crashes not only cause death and injury, but they also bring along an immeasurable amount of agony to the people involved. Efforts to improve traffic safety to date have concentrated on the occupant protection, which had improved the vehicle crash worthiness. The other important area where research is currently being done is collision avoidance.

Technological innovations have given the traffic engineer an option of improving traffic safety by utilizing the available communication tools and sophisticated instruments. Using sensors and digital maps for increasing traffic safety is in its infancy. Systems are being developed to utilize the available state of the art facilities to reduce or possibly prevent the occurrence of crashes. Total prevention of crashes might not be possible for now, but the reduction of crashes could easily be achieved by using the collision avoidance systems

BACKGROUND

The 1998 United States national crash statistics indicate 6,335,000 crashes. These crashes resulted in about 41,471 fatalities, 3,192,000 million nonfatal injuries and 4,269,000 crashes involving property damage only. On an average 114 persons died each day in a motor vehicle crash - one every 13 minutes. According to an estimate for reported and unreported crashes economic cost of traffic crashes in United States for the year 1994 was around $150.5 billion (*1*).

Figure 1 below indicates that about 45 percent of the crashes that occur are caused by human errors. Human errors that cause crashes include failure to keep in proper lane, failure to yield right of way, inattentive, failure to obey traffic control devices, operating vehicle in negligent manner, drowsy driving, over correcting, driving wrong way and making improper turns. Some of these crashes may have been possibly avoided if the driver was provided with the real time information.

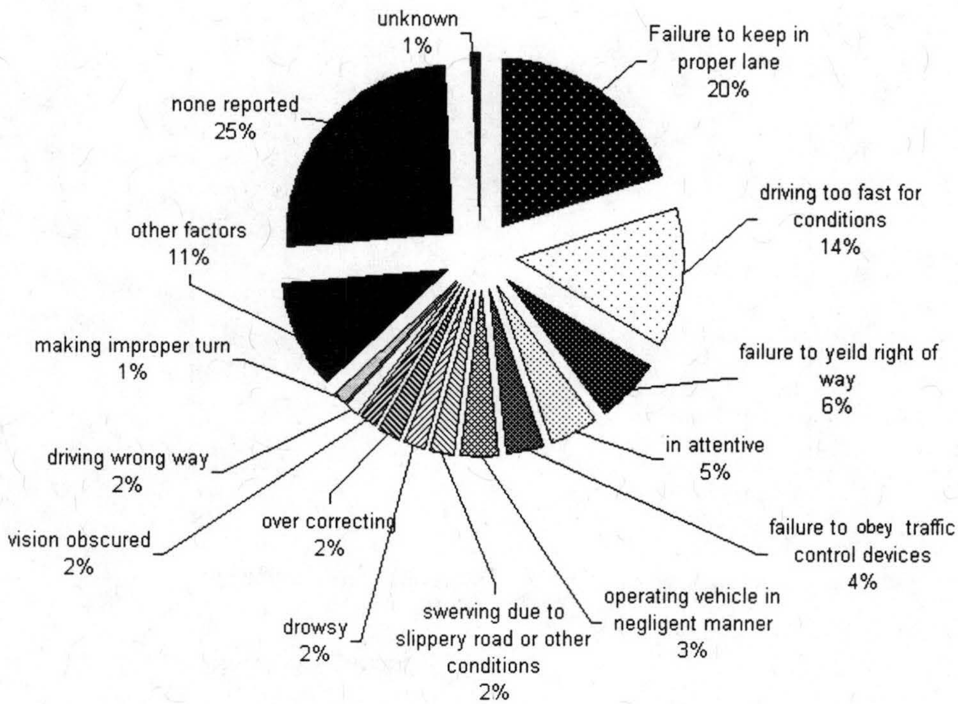

FIGURE 1 Factors Related to Fatal Crashes (*1*)

WHAT CAUSES A COLLISION?

The Indiana Tri-Level study found that driver errors are a cause for about 93 percent of the crashes (2). Human errors that contribute to crashes include:

- Recognition errors

These errors include situations where a conscious driver does not properly perceive, comprehend or react to a situation. This type of errors may occur due to inattention, distraction and improper look out. These errors were found to be a definite/probable factor in 56 percent of the in-depth Tri-Level crashes.

- Decision errors

These errors include situations where a driver selects an improper course of action to avoid a crash. This might be due to misjudgment, false assumption, excessive speed, tailgating, and inadequate use of lighting and signaling. These errors were found to be a definite/probable factor in 52 percent of the Tri-Level crashes.

- Performance errors

These errors include situations where a driver selects an appropriate course of action, but commits a mistake when executing the action. These errors occur due to overcompensation, panic, and inadequate directional control.

Figure 2 shows some of the most common factors that cause a crash (2)

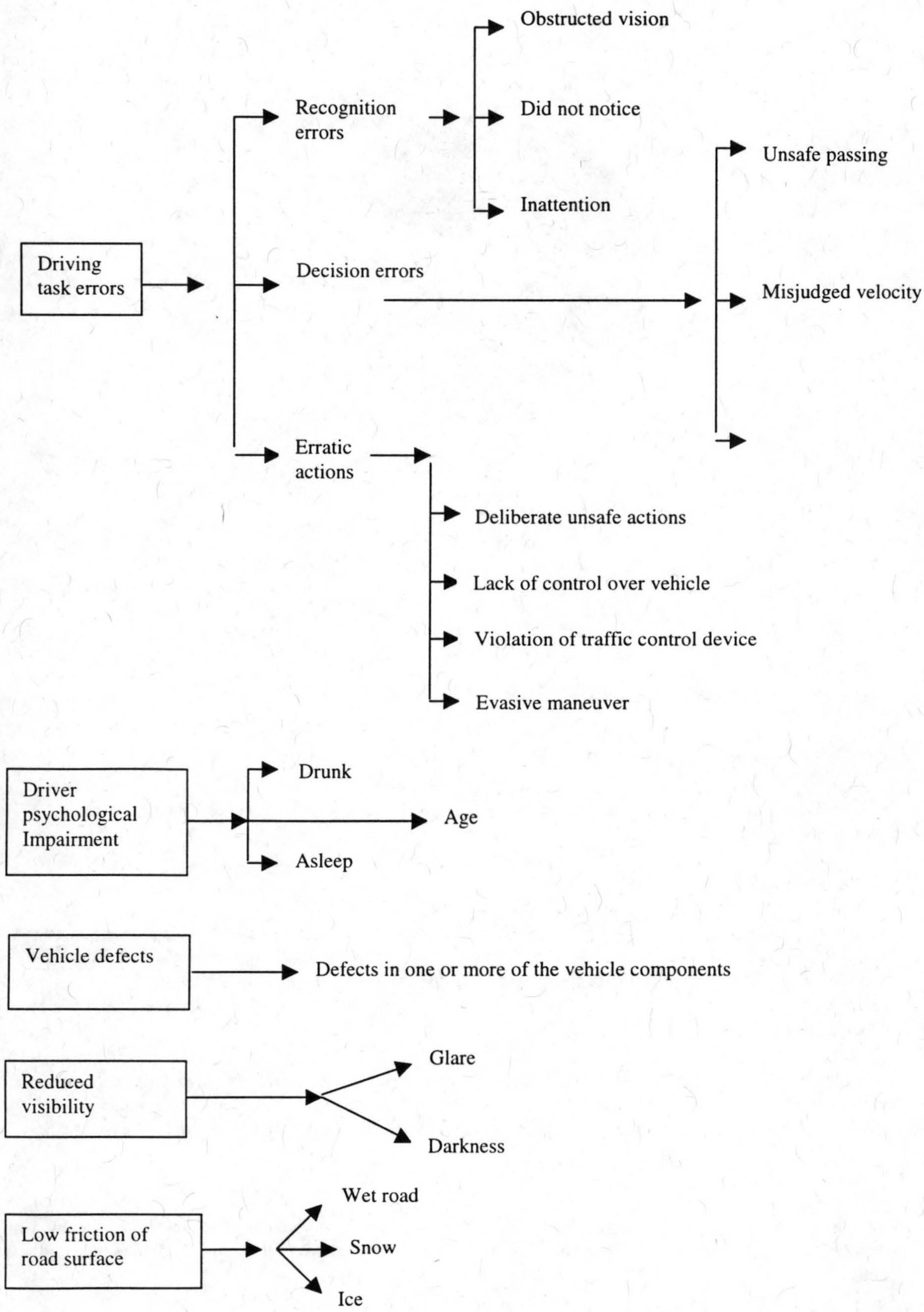

FIGURE 2 The Classification of Factors that Cause Crashes (*2*)

COLLISION AVOIDANCE SYSTEMS

The human errors related to crashes may be avoided if information is provided to the driver about the possible occurrence of a collision. Collision avoidance systems are technological tools that interact with the driver to reduce or prevent the occurrence of a crash by informing or warning the driver or by taking the control of the vehicle. The collision avoidance systems include driving enhancement tools like vision improvement, antilock brakes, obstacle detection and collision warning systems. Collision avoidance systems not only reduce the crash frequency, but also reduce crash severity (2).

Collision avoidance systems prevent a particular type of crash under a set of circumstances. Identification of major crash types, factors related to crashes and vehicle types are necessary to understand the possibility of crash reduction by the usage of collision avoidance systems. Figure 3 shows how a collision avoidance system works.

The collision avoidance systems may achieve traffic safety by preventing accidents altogether. According to one study, 60 percent of crashes at intersections and about 30 percent of head-on collisions could be avoided if drivers had an additional half-second to react (3). Collision avoidance systems will alert the driver of a possible collision in advance, giving him sufficient time to react. If the driver does not react to the information the collision avoidance system warns him of the impending collision. If still the driver does not react to the situation, the collision avoidance system takes control of the vehicle to avoid the collision. It is estimated that inattentive drivers are the cause of 75 percent of vehicular crashes (3).

Single vehicle roadway departure crashes and crashes into stationery parked vehicles account for about 20 percent of all crashes, but about 50 percent of these crashes are fatal (2).

From figure 4 it can be observed that the single vehicle roll over and angle crashes constitute for about 48 percent of the total occupant fatality, while the head on and rear end collisions contribute to 25% of the occupant fatalities. This indicates that if real time information is provided to the driver a major part of the accidents may be avoided.

Figure 5 indicates that the majority of occupant fatalities involve passenger cars. This shows that passenger vehicles equipped with collision avoidance systems would be more beneficial. The expected number of truck involvement's in crashes over the vehicle lifetime is more than twice that of other vehicle types, this is mainly due to their high vehicle miles traveled (2). It is estimated that a truck tractor on average has 60,000 vehicle miles traveled annually versus 10,000 vehicle miles traveled by a passenger vehicle (2). Therefore trucks are likely to need collision avoidance measures many times during their operational lives, which makes the installation of the collision avoidance systems in trucks more effective.

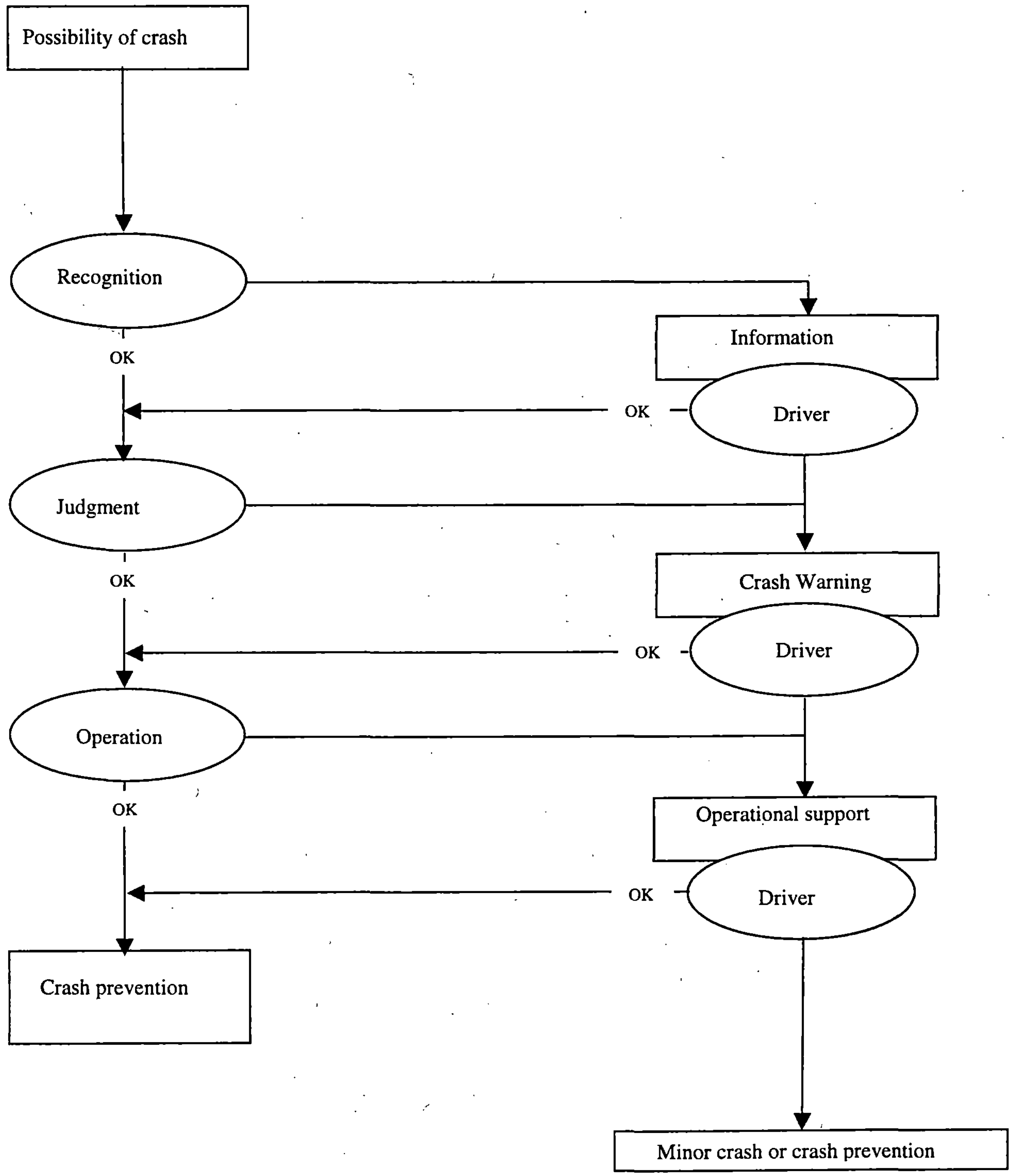

FIGURE 3 Crash Occurrence and Avoidance Process (4)

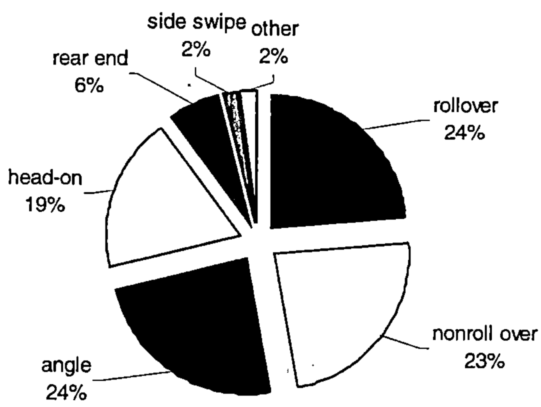

FIGURE 4 Occupant Fatality Rates Based on Collision Type *(3)*

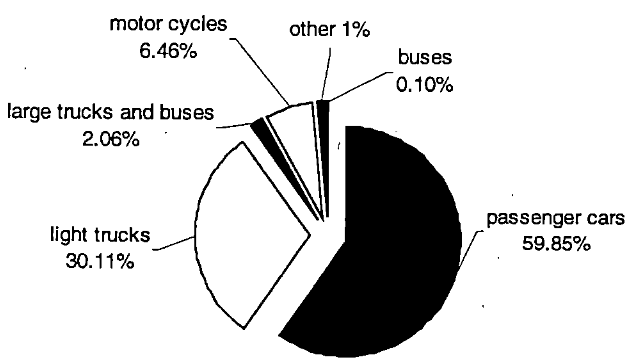

FIGURE 5 Occupant Fatality Rates Based on Vehicle Type *(3)*

CLASSIFICATION OF COLLISION AVOIDANCE SYSTEMS

Functional Classification

Collision avoidance systems can be classified based on their function. They include the following types *(2)*:

1. Advisory collision avoidance systems
2. Collision warning systems
3. Automated crash avoidance

Advisory Collision Avoidance Systems

These systems constitute information tools that tell the driver of the proximity of vehicles and obstacles within a specified distance from his vehicle. The interface could be either visual or auditory. These intelligent tools also inform the driver of the possible course of action to take to negotiate a steep curve, or a slippery surface or any other situation that requires his attention. These systems are designed to give the driver sufficient time to react to a situation that could possible occur. These systems also serve as sensory enhancement tools in reduced visibility conditions (2). Variable message signs come under this category.

Collision Warning Systems

These systems warn the driver of an imminent collision when the driver selects an improper course of action or takes no action to avoid a crash. The warnings are in the form of alarms. These systems require a correct and immediate response to avoid collisions (2).

Automated Crash Avoidance Systems

These systems are the most advanced systems that could prevent collisions from occurring. These systems include tools that take total control of the vehicle when the driver does not react in a necessary manner to prevent the collision (2).

Collision Avoidance Technologies based on Crash Type

Single Vehicle Road Departure

Single vehicle road departure occurs when the driver goes at an excessive speed in relation to the road environment, or due to reckless driving, or due to vehicle failure. Technologies to prevent these types of crashes include:

Driver Vigilance Monitoring This technology measures a driver's alertness by using steering wheel movements or by detecting the driving trace (2,4). The same technology is also used for lane departure warning. The driving trace is detected by using the signals from a vibration gyro sensor and a velocity sensor in the car navigation system. These signals are used to calculate a base line for normal driving. Horizontal deviation of the vehicle is estimated and if the estimated horizontal deviation exceeds a particular limit an audio warning or alert is issued (4). Figure 6 gives a graphical depiction of the driving trace technology.

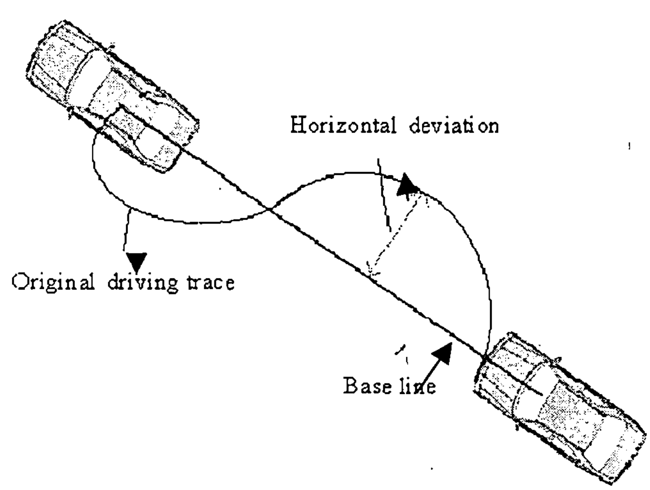

FIGURE 6 Maroon Zigzag Driving Detection Function (*5*).

Ice Warning In-vehicle warning devices or variable message signs can be used to warn the driver of a slippery condition on the road. Technologies to detect the presence of ice include in-vehicle tire and road friction monitor and roadside devices that measure air and ground temperature and humidity. These parameters are used to calculate dew point to determine the possibility of ice formation (2). If the calculations indicate ice formation, Variable message signs can be used to display the warnings automatically.

Vision Enhancement These technologies use infrared radiation sensors to detect moving objects and roadside features in low visibility conditions to provide the driver with an enhanced view of the road ahead (*2*).

Curve Approach Warning

Vehicle-to-roadside communications devices can provide in-vehicle warning by integrating vehicle speed and significant vehicle dynamics information with knowledge of road geometry either from a map database in the in-vehicle navigation system or beacon input provided on the curves. These curve-warning devices are used to complement existing road signs (*4*).

Rear End Collisions

Six percent of occupant fatalities in crashes that occurred in United States during the 1998 year were due to rear end collisions. Most of these collisions result from lack of attention and/or following a vehicle too closely, technologies to prevent the collision from occurring would include (*2*):

Headway Detection Systems Sensors to detect the presence and speed of vehicles in front of the vehicle would be utilized to provide advice regarding the safe minimum headway to be maintained. These sensors would also detect the presence of obstacles in the vehicle path and would provide the driver with warnings to minimize the risk of collision.

Adaptive Cruise Control This system is an application of headway detection system, which utilizes automatic braking to avoid collisions with vehicles in the lane of travel. Route guidance systems with enhanced map databases, and cooperative communication with the highway infrastructure can be utilized to set adaptive cruise control systems at safe speeds (*4*).

Collisions at Intersections

Driver inattention, deliberate violation and intoxication are the three types of human errors involved with the crashes at intersections. Advisory collision avoidance systems can prevent the occurrence of crashes that occur due to driver inattention.

Other Tools for Collision Avoidance

Anti Lock Brakes The objective of antilock brakes is to automatically modulate braking pressure to prevent the vehicle's wheels from locking during braking. Anti locks pump brakes automatically, many times a second, to prevent lockup and enable a driver to maintain steering control. Anti lock brakes prevent Locking of vehicle wheels, which are the cause of longer stopping distances, skidding and loss of control on wet and slippery roads (*6*).

ADVANTAGES

Collision avoidance systems would reduce the number of accidents to a great extent. Accident reduction would also help reduce vehicle-hours of delay. According to the study conducted by Sullivan (*9*), around 40 percent of accidents occur in travel lanes, 10 percent on the median shoulder and the rest on the right shoulder. During congested periods an average accident can induce 500 to 1000 vehicle hours of delay (*6*). According to National Highway Traffic Safety Administration some of the collision-avoidance systems could prevent 1.1 million accidents in the United States each year and would save 17,500 lives and $26 billion in accident-related costs (*3*).

"Greyhound Lines have installed systems on its bus fleet, which give collision warnings for the front of the vehicle and lane change warnings for obstructions in the driver's blind spot. As a result, Greyhound's accident rate fell 21 percent from 1992 to 1993". (*8*)

According to a study the static and dynamic vision of people with age greater than 60 years is less compared to drivers below the age of 60 years (*5*). The percentage variations are found to be around 17.17 percent and 31.14 percent for static and dynamic vision respectively (*5*). The time required for the drivers with age greater than 60 years to react and apply the brakes is found to be 1.35 times that of the drivers with age less than 60 years (*5*). Vision enhancement devices and other simple collision avoidance devices could assist the aging drivers by providing them with more amount of time to react for a given situation. This could possibly help reduce collisions.

Simple collision avoidance systems that prevent backing collisions are available at low costs. These systems could help reduce the number of collisions that are very common in the parking lots.

DISADVANTAGES

These systems might produce false alarms, which might make the driver to discard the warnings. The frequency of the audio alarms has to be carefully designed so that it is not irritating to the driver. Increased speeds will reduce the headway and would increase the probability of a severe accident in the event of malfunctioning of a collision avoidance device. Some of these systems might make driver lose control over the vehicle, this would make the drivers dislike the deployment of collision avoidance systems. More extensive research in this area would help the engineers to design a foolproof system that is more reliable, economical and safe.

CONCLUSIONS

Collision avoidance systems if carefully designed would increase the situation awareness of drivers by eliminating or decreasing the human errors. These systems would bring about a major change in solving traffic safety related problems. Consideration of human factors in the design of the collision avoidance systems plays a very important role. Human centered design would make them more acceptable and useful to mankind.

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SAFETY AND DESIGN IMPROVEMENTS AT RURAL EXPRESSWAY MEDIAN CROSSOVERS

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ABSTRACT

Current progress of study on safety and operational issues of a particular type of wide rural median crossovers in the state of Missouri was described. In so far as the progress of the study, two potential analysis tools were compared. Found to be able to provide needed information, the CORSIM simulation software was selected as the main analysis tool in further study instead of the other potential tool, Highway Capacity Software. General CORSIM diagram of studied median crossovers was presented. Three sites were selected for data collection upon particular criteria. Traffic data in those three sites were recorded for further study use. Observations of safety and operational problems occurred in those sites were presented. Potential improvement techniques were recommended.

INTRODUCTION

According to the standard plans on the web page of the Missouri Department of Transportation (MoDOT) *(1)*, a typical type II median crossover has a wide median (at least 18m) which has a separate deceleration lane for the left-turn vehicles in each direction on the major road. Normally, the side roads are controlled by two-way STOP signs, and the median area is controlled by YIELD signs. A simple sketch of this kind of crossover is shown in FIGURE 1-1. A detailed sketch is available on the web page *(1)*.

While a wide median crossover can serve the normal function of providing storage space for crossing traffic in addition to left-turn vehicles and U-turn vehicles, there are some problems that affect the safety and operation of the intersection as a whole associated with wide median crossovers. The specific problem MoDOT faces is that multiple vehicles can be stored in the median area when the volumes increase through a crossover area. Those stored vehicles can actually block each other and impede visibility to oncoming vehicles. Some long vehicles (e.g., a WB-20 truck 22.5 meters long) may enter a wide median crossover without enough area to store it when trying to make a maneuver through the crossover area, and protrude into the through lane. This produces a dangerous situation to the vehicles going through the major expressway in high speed.

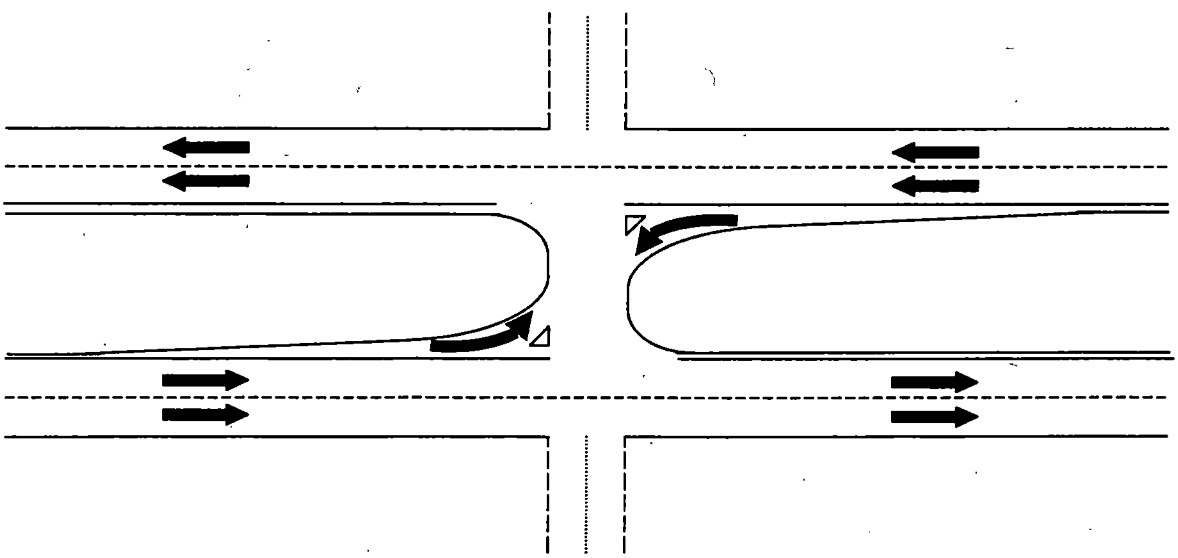

FIGURE 1-1: Typical type II median crossover

The object of the research is to try to provide a means for MoDOT engineers to determine whether particular high-speed rural expressway crossovers are performing satisfactorily and, if not, to assess alternatives for crossover design. The research begins in January 2000 and is expected to be completed by July 2001. This paper describes the progress so far.

Previous studies in this area were reviewed. A few studies were found that address safety and operational issues on rural expressways with wide median crossovers. Among those, three documents need to be addressed. In NCHRP Report 375: Median Intersection Design (2), some safety and operational problems associated with wide median intersection were studied. Relative strategies of the geometrics and traffic control measures like special left-turn treatments and U-turn treatments were presented. Bonneson, James A. et al. (3) described current state of practice of potential measurements that some state highway departments used to improve traffic safety at intersections on rural expressways. In NCHRP Synthesis of Highway Practice 281: Operational Impacts of Median Width on Larger Vehicles (4), some safety and operational problems found in median crossover were classified into several groups and described in detail. Corresponding practices used by the states to improve each situation were summarized as alternative improvement techniques. The reader is referred to the above references for more details.

ANALYSIS METHODOLOGY

Analysis Tools

Two analysis tools were for potential use in the research. One is the *Highway Capacity Manual* (HCM) *(5)* by using the *Highway Capacity Software* (HCS) *(6)*; the other is the simulation software *Traffic Software Integrated System (7)* through using the package of CORSIM.

The HCM, based on a deterministic model, provides a basic analysis methodology for two-way stop-controlled intersections, such as the type II median crossover. Unfortunately, the HCS cannot serve the objective of analyzing the performance of the crossover location. The HCM/HCS procedures do not provide information about the median storage spaces needed by vehicles. The potential for applying the HCM/HCS to a crossover as two separate intersections (where the major highway is modeled as two separate one way roadways) was examined, but abandoned because of the storage location of major road left-turns in the deceleration lane.

Computer simulation is a useful analytical tool for traffic engineering. The package of CORSIM of the Traffic Software Integrated System *(7)*, like all other simulation softwares, makes it possible to predict the effect of expected strategies on the system's operational performance prior to field demonstration. It provides a whole set of measures of effectiveness (MOEs), including average vehicle speed, vehicle stops, delays, fuel consumption, and pollutant emissions which can be used to compare the effects of the applied strategy on the traffic stream, and hence provides the basis for selecting the most effective alternative. Furthermore, by defining the segment of the intersection as a studied area, CORSIM can analyze the MOEs of the segment as well as the median crossover as a whole, which cannot be accomplished by using the HCM/HCS. For this reason, CORSIM was chosen as the main analysis tool in this research.

Data Collection

Data input required by CORSIM generally includes geometrics of the intersection, lane usage, traffic volumes, traffic composition and turning movements, and control devices.

Traffic volumes are being collected by video camera. Traffic composition and turning movements and some other input of CORSIM, such as critical gap, will be derived from the data. Geometrics of the intersections and information of control devices are provided by MoDOT and by field verification.

The output of CORSIM will be compared with the recorded data. To quantify the impacts of the alternatives on each part of the system and identify trade-offs, output of alternatives will be compared with (8):

- Individual links (e.g., the median area)

- Intersections (e.g., the intersection as a whole)
- System components (e.g., major road through traffic vs. cross-minor road)

For the objective of this research, particular criteria were used to select field data collection sites:

- Traffic volume, especially the volume of traffic passed through the crossover, should be high.
- The experience of operational and/or safely problem in the study area is more frequent (e.g. number of accidents is higher for the same time period).
- Types of accidents related to median crossover are severe.
- A range of intersection geometrics or other characteristics of the crossover is preferred.
- The data collection site is convenience for video.

Three sites were selected based on these criteria. Two are typical type II median crossovers. The median crossover area of one intersection (US Highway 63 & Route H) is controlled by STOP sign, while the other's (US Highway 50 & Cityview Dr.) is controlled by YIELD sign. The third intersection (US Highway 54 & Business 54/Route W) is an intersection of typical type II median crossovers plus acceleration lanes for the left-turn vehicles turning onto the main road.

Traffic data are being collected for two hours including the peak hour. The author found that the time periods with the higher traffic volumes passing through the median crossover area corresponded to the peak flow time period for each intersection.

A feasible set up of the video camera to record the field data is shown in FIGURE 2-1 *(2)*.

The videotape data will be used to:

- Count traffic volumes. Traffic volumes will be counted separately for each individual turn movement (through, or left-turn) in each approach and in each direction of median area.
- Classify the vehicle mix.
- Calculate critical gap.
- Identify operational problem.
- Identify and count erratic maneuvers.
- Identify problems encountered by specific vehicle type in negotiating the intersections.

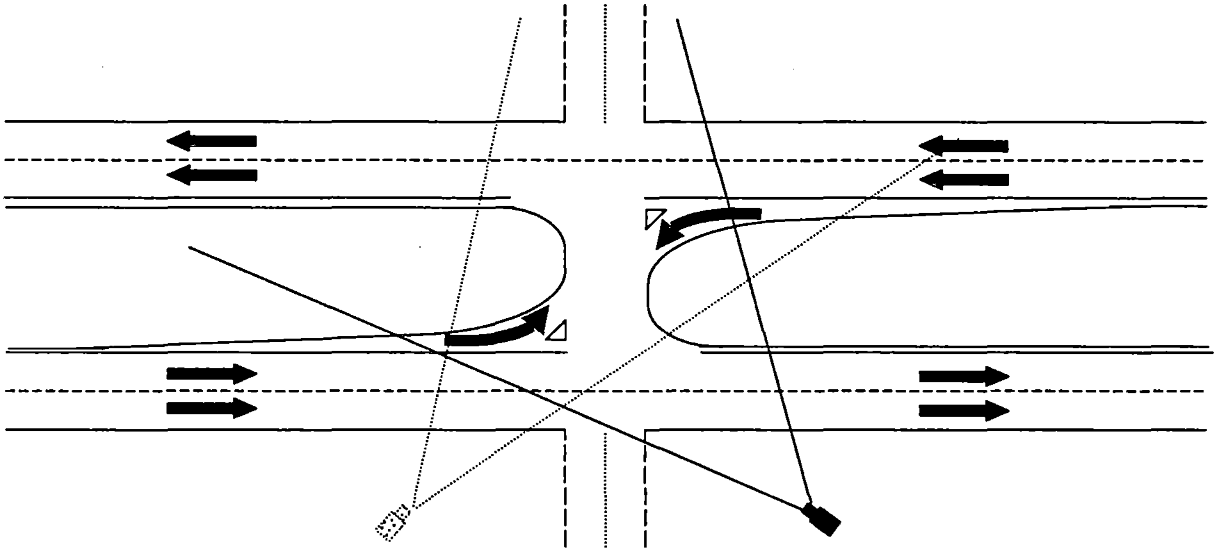

FIGURE 2-1: Typical setup for video camera

GENERAL SIMULATION PROCEDURE

Converting the intersection layout to a so-called link-node diagram is the first step to build the final model. In CORSIM, links are one directional segments of the studied facility, and nodes are usually the intersections of two or more links. According to the typical layout of the rural type II median crossover, the link-node diagram for it generally is like the diagram in FIGURE 2-2.

In the diagram, links between node 2 and 5 represent the median in the rural type II median crossover. The link from node 8001 to 8002 represents one direction of the major road, and link from node 8003 to 8004 represents the other direction. The various links between node 8005 and node 8006, except those between node 2 and 5, represent the two directions of the minor road that cross through the major road. Link from node 7 to 2 and link from node 8 to 5 are controlled by stop sign. Link from node 2 to 5 and link from node 5 to 2 are mostly controlled by yield sign.

There are two limitations in the current versions of CORSIM should be noted:

- In version 4.32, control delay is not calculated. The simulation can output total delay, queue delay, and stop delay.
- The software cannot show curved links, such as the right turn channelization. Some of the curved links can be coded by dividing the link into several short segment to get a smooth curve line, but usually short-distance links may cause a great deal of problems.

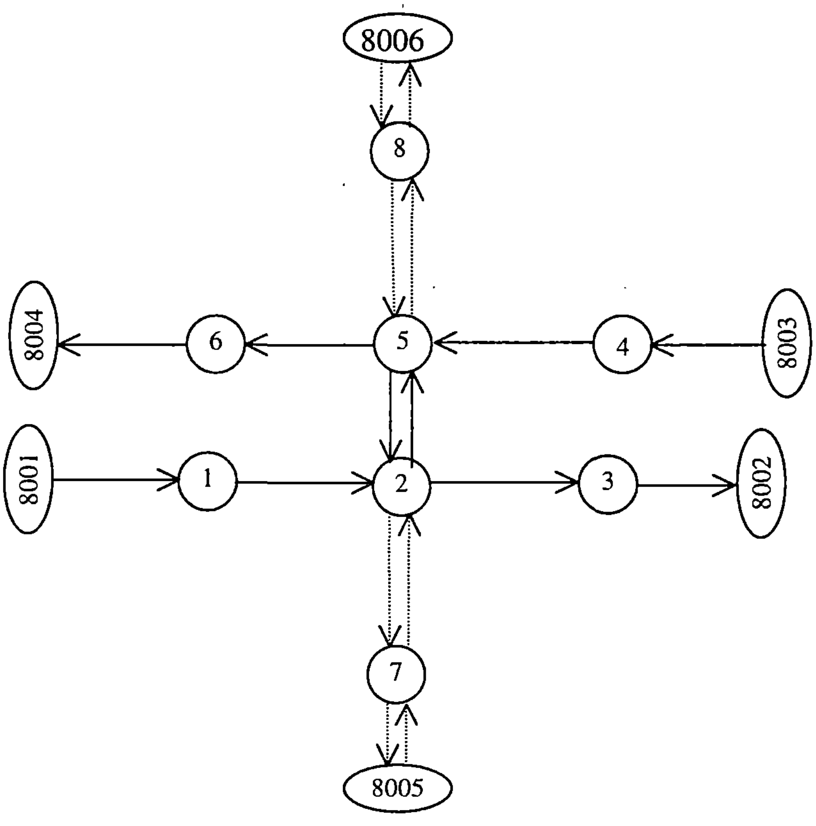

FIGURE 2-2: General representation of O-D diagram of typical type II median crossover

ANTICIPATED RESULTS

From the observation of the field data collected so far, there are some problems associated with the median crossover at the three selected sites. Some safety and operational problems are listed below:

1. Though all of the three intersections serve the function for vehicles traversing well, they did have some extreme situations. The median area of Hwy. 63 & Route H had as many as 6 vehicles stored in one direction, which made the queue backup into the deceleration lane. The median area of Hwy. 50 & Cityview Dr. had 5 vehicles stored in one direction. Most vehicles in the queue backtracked to the deceleration lane because the median area of this intersection can only store at most two passenger cars. For the Hwy. 54 & Bus. 54/ Route W, 7 vehicles were stored once in the minor road. Because the median area of this intersection is relative narrower, it can store only one vehicle for each direction. These are some potential sources for the safety concern.

2. Vehicles stored in the median area can block the vehicles wanting to traverse the median area from the other direction. One such situation occurred in the intersection of Hwy. 63 & Route H when there are 3 vehicles stored in the median area. The passway for the left vehicles on the other direction was blocked by the queue of the three vehicles. Only when a vehicle in the queue discharged was the blocked vehicle able to enter into the median area from the deceleration lane.

3. When the queue backed up to the deceleration lane, there seemed to be confusion for the vehicles wanting to traverse the median area from the minor road. Also, there were conflicts between stored vehicles and vehicles entering the median from other directions.

4. For some drivers who had to traverse the median area from the minor road to the major road, a longer waiting time for the first stage of gap acceptance may make the driver want to reduce his/her normal gap acceptance time for the second stage. There is one observation in the intersection of Hwy. 50 & Cityview Dr. where a car from the minor road passed through the nearside of the major road after a long time waiting. The driver seemed to want to leave the median area soon. After a few seconds of waiting in the median area, the driver tried to pull onto the major road, but had to back up into the median area again because of the gaps between the vehicles on the major road were not permissive.

5. There was confusion of priority between drivers in conflict with each other. Though most of the drivers assumed that the vehicles on the major road had the priority, some did not comply with that assumption.

6. There was confusion of the right of way in the median area. When the intersection was not at a right angle and was not marked, drivers used the median area in various ways. Some drivers turning left from the major road occupied the right side of the median area, while some others used the left side of the median to cross the major road traffic.

7. Drivers tend to ignore the storage function of the median area if the median area is narrower. This situation was found in the intersection of Hwy. 54 & Bus. 54/Route W. The median area can store only a car stopping at any significant angle. Though a few drivers used this narrow median as a refuge, most of the drivers tend to complete the two-stage gap acceptance at once without stopping in the median area. The median area seemed to be too narrower for drivers to make them feel comfortable to use it as a storage area. Complicating the situation, most of the vehicles here were recreation vehicles with longer length than a passenger car.

In general, according to D. Harwood and W. Glauz *(4)*, those problems that may occur in the type II median crossovers can be classified as:

1. Undesirable driving behavior, which includes:
    - Encroachment on through lanes by vehicles in the median opening.
    - Side-by-side queuing in the median opening.
    - Angle stopping in the median opening.
2. Collisions between left-turning vehicles and vehicles stopped in the median opening area.
3. Collisions between vehicles turning left from the divided highway and other same-direction vehicles.
4. Collisions between vehicles turning left from the median area and opposing through vehicles.
5. Collisions between vehicles making U-turns and opposing through vehicles.

POTENTIAL RECOMMENDATION TECHNIQUES

There are some safety and operational problems related to the Type II median crossovers. To improve the situation, the geometrics of the intersection, characteristics of traffic flow in each direction of the intersection, and traffic control devices are all involved in the consideration of mitigation techniques. Based on the field observation and related literature like *(2, 4)*, some mitigation techniques are recommended below. Further study will be conducted by using the CORSIM simulation software to analyze the current performance of the selected intersections, and to compare it with some alternative mitigation techniques later on.

- <u>Make the geometrics of the intersection uniform.</u>

- Widen some narrow medians: The width of median should be able to store at least one of the largest vehicles using the intersection most often safely. The width of the median should not cause the line stored in the intersection which combined by any kind of vehicles like passenger cars or school buses encroach the through lanes.

- Provide median acceleration lanes: It was indicated by NCHRP Report 375 *(2)* that, on the basis of the guidelines used by state highway agencies, acceleration lanes for left-turning vehicles from a crossroad onto the divided highway should be considered at locations where adequate median width is available, and:

  (a) limited gaps are available in the major-road traffic stream;

  (b) low-speed turning traffic merges with high-speed through traffic;

  (c) there is a significant history of rear-end or sideswipe accidents;

  (d) intersection sight distance is inadequate;

  (e) high volumes of trucks entering the divided highway.

- Extend edgelines to better define median opening area.

- Mark double yellow centerline on roadway in the media opening to discourage angle stopping.

- Remove STOP signs in median. The actual practice of most highway agencies was to use no control in the median opening area for median widths up to 9m(30ft), to use YIELD control for median widths from 9 to 25m(30 to 82ft), and to use STOP control for median widths of 25m(82ft) or more *(2)*.

- Install traffic signals. Traffic signals are less desirable in rural area because of high approach speeds. Installation of traffic signals at median openings should be considered only at locations where the *Manual on Uniform Traffic Control Devices (10)* signal warrants are met *(4)*.

- Install better delineation.

- Install advance intersection signing.

- Install bigger signs.

- Prohibit left-turn maneuvers.

- Increase the deceleration and storage length of existing left-turn lanes. Extending the length of a left-turn lane may only be feasible where the highway median is sufficiently wide to accommodate the extended median *(4)*. AASHTO policy provides guidance on the appropriate deceleration and storage lengths for left-turn lanes *(9)*.

- Provide median crossover or indirect routes for U-turns.

- Prohibit left-turn maneuvers or U-turn maneuvers.
- Close median opening.
- Reconfigure median to prohibit crossing maneuvers while still permitting left-turns. Consideration must be given to the alternate routes that will be used by the diverted traffic and the traffic operational and safety impacts on other locations for the above three techniques.

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