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Transportation Research Record 1172

Contents

Foreword

In this Record, various aspects of accident data bases and analysis and of the visibility of different parts of the highway system are discussed.

Mak et al. assess the utility of present highway data bases for providing the information necessary to aid safety analysis and decision making. Turner and Colson then describe the need and techniques for and benefits of using accident data in risk management programs.

The next three papers give specific methods developed to improve the usefulness of accident data. A loglinear approach, emphasizing a generalized loglinear interactive model for calibration, is suggested by Saccomanno and Buyco. They suggest this technique for assessing the effect of traffic environment on truck accident rates and demonstrate it with 1983 truck accident and exposure data. Exposure data are a necessary ingredient for accident analysis, and Harwood et al. describe ways to estimate wet pavement exposure from existing weather records with a computer model, WETTIME, which can be used by highway agencies. Injury accident models based on geometric and operating characteristics of signalized intersections are described by Lau and May. They apply the Classification and Regression Tree (CART) technique to building injury accident models and compare the technique to others.

Accident data are used to address characteristics of specific groups of drivers in the next two papers. Polus et al. compared accident rates and patterns for male and female drivers in Israel. McKelvey et al. examine the relationship between driver age and highway accidents by using a variation of the induced exposure method. They confirm that the accident involvement of elderly drivers is higher than that of other age group drivers.

The concluding five papers revolve around the critical issue of visibility. Zimmer describes the development and use of a simple, cost-effective, and easily assembled system to measure illuminance of high-mast roadway lighting. The system, housed in a passenger automobile moving at traffic speeds, allows data to be stored on computer disk so that illuminance measures and uniformity ratios can be calculated later. Fog can severely degrade visibility, so when potentially hazardous fog conditions were predicted for a proposed bridge, Potash and Brown conducted a study to review and evaluate alternative mitigation measures. They describe the recommended and approved mitigation system. To optimize post delineator placement, Zwahlen et al. developed and demonstrated a computer program based on visibility. The program optimized height, spacing, and lateral offset of post delineators for tangent sections and horizontal curves on two- and four-lane highways. Janoff, using accident data from the San Mateo Bridge in California, found that flickering lighting systems do indeed produce the adverse nighttime safety effects predicted in earlier work. In the last paper, van der Horst describes a before and after behavioral study of the yellow light interval at signalized intersections in the Netherlands, in which he found that an appropriate yellow interval cut red light violations in half. He also concludes that an improved yellow interval on signals at drawbridges and railroad grade crossings would be equally useful.

Assessment of Existing General Purpose Data Bases for Highway Safety Analysis

KING K. MAK, JOHN G. VINER, AND LINDSAY I. GRIFFIN III

Safety is a continuing concern for highway officials at all levels of government. If the safety impacts of existing and proposed programs and policies for the construction and maintenance of highway systems are to be properly assessed, it is imperative that these officials be provided with the necessary supporting data. A recently completed FHWA-sponsored study critically reviewed a number of large national data bases for applicability and utility to various areas of highway safety that are of prime concern to the FHWA. Conceptual alternatives that would improve or enhance the capability and utility of these data bases from the standpoint of highway safety analyses were developed and evaluated for feasibility and practicality, and appropriate recommendations were made. The study results are to be considered as part of an effort to improve the capability and use of existing data bases and to offer a basis for improvements in ongoing and future data collection efforts so that the information needs of highway safety analyses may be better served.

Highway officials at the federal, state, and local levels are faced with the continuing task of assessing the potential safety impacts of proposed programs, policies, and alternatives in the construction and maintenance of the highway systems. To ensure that their decisions are appropriate and as cost-effective as possible, these officials must be provided with the supporting data needed to conduct the appropriate safety analyses. This work ranges from the identification of problems, causal factors, and countermeasures to the evaluation of the effectiveness, as well as the unintended effects of the countermeasures.

Various data bases have been created and are maintained at the federal, state, and local levels for a variety of reasons. Some of these data bases are intended for record-keeping purposes, with no consideration given to analysis requirements. Others are designed for a specific purpose and are of little use otherwise. A research study sponsored by the FHWA critically reviewed the existing general purpose data bases for their ability to meet the information needs of highway safety analyses (*1*). In this paper, selected major findings and recommendations from this study are presented.

HIGHWAY SAFETY ANALYSIS

The term highway safety, as used in this study, refers to those traffic safety areas that, at the national level, are of prime concern to the FHWA. In consideration of this definition, the emphasis of the study was on large data bases that are national in scope and intended for general purposes. The various components of highway safety analysis that are considered in this study are shown in Figure 1.

Studies of highway safety data bases can be categorized as either analysis or implementation. Analysis refers to the use of the data bases to address problems and questions from the standpoint of research and development, evaluation, and analysis. Implementation, on the other hand, is related to the development of warranting criteria and to project selection based on the warrants. The assessment of the data bases in this study was only from the analysis standpoint and excluded implementation.

As shown in Figure 1, highway safety analysis can be characterized by four factors:

- Type of analysis (problem identification, cross-sectional evaluation, or longitudinal evaluation);
- Unit of analysis (location or accident);
- Purpose of analysis; and
- Specificity (highway, accident, or both).

Type of Analysis

Types of analysis commonly used in highway safety studies are as follows:

- *Problem identification.* The determination of where and why the accidents are occurring;
- *Cross-sectional evaluation.* The study of the effects on or relationships to accidents of various factors, using information during a given time frame; and
- *Longitudinal evaluation.* The study of the effect of a given treatment (e.g., a countermeasure or a modification to a highway or to an environmental factor) on accidents during different time frames.

Unit of Analysis

This may be defined as location or accident. A location is defined as a roadway section, a point on the roadway, or a physical feature of the roadway, such as a bridge. An accident refers to an actual accident or a vehicle involved in an accident. The unit of analysis corresponds to the dependent variable used in the analysis. A location-based analysis is related to accident frequency or rate, whereas an accident-based analysis is related to accident severity.

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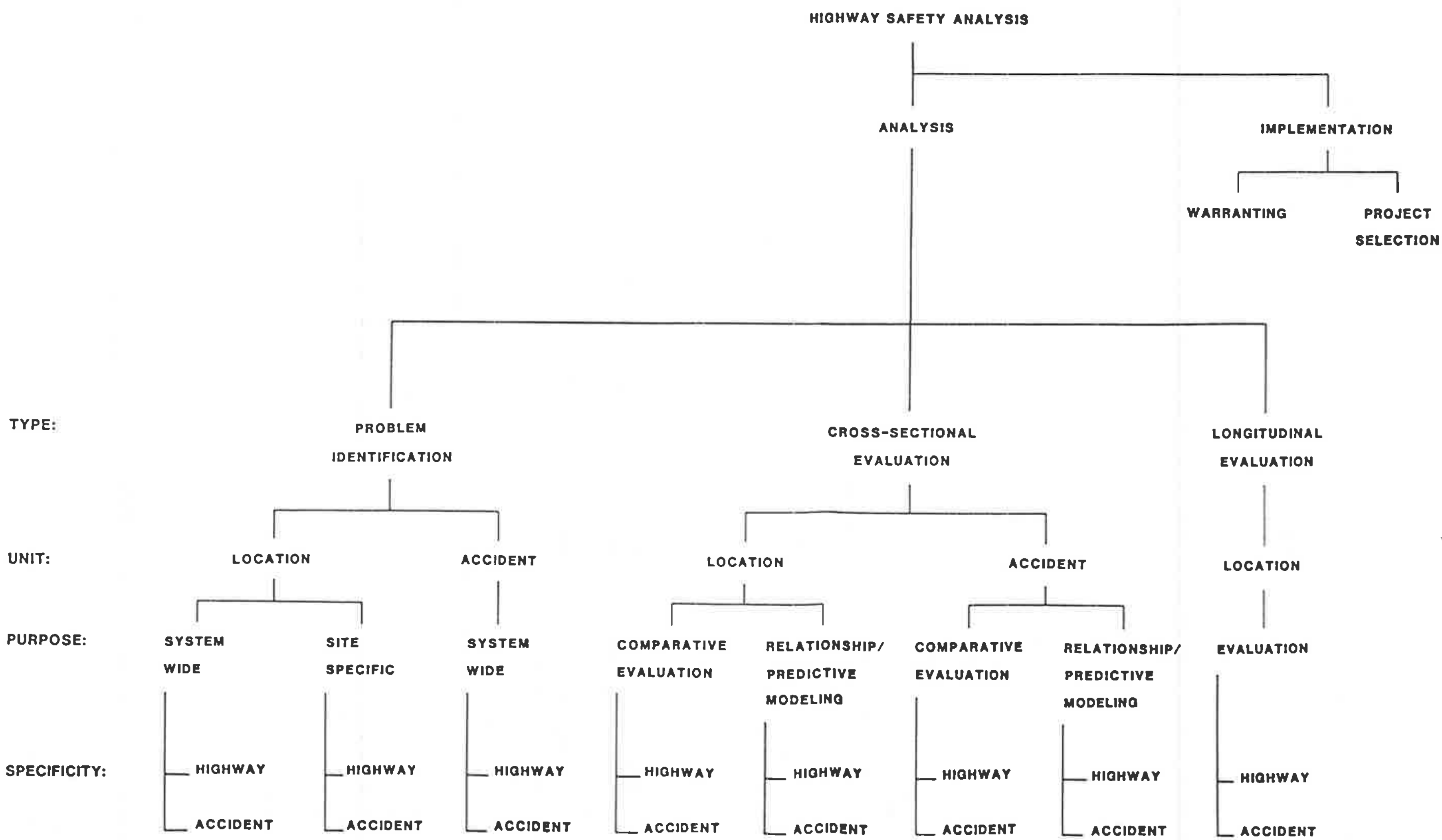

FIGURE 1 Characterization scheme for highway safety analysis.

Purpose of Analysis

This is simply the objective of the analysis, that is, what the analysis is intended to accomplish. The purpose varies according to the type of analysis being conducted.

Specificity of Analysis

This factor reflects the level of detail needed in the analysis. It includes division into subsets or constraint of the data (e.g., fatal accidents only, or two-lane rural highways only), and the inclusion of variables (e.g., highway type, accident type, etc.). The specificity for any analysis purpose can be defined by highway-related variables (e.g., highway type, curve or tangent, number of lanes, etc.), by accident-related variables (e.g., weather and surface condition, accident type, vehicle type, injury severity, etc.), or by both types.

For analysis of problem identification, the unit of analysis can be either location or accident. The purpose can be system-wide (program level) or site specific (project level). Note that accident-based analysis is not applicable at the site-specific level. By definition, a site-specific analysis refers to a location or locations and not to an accident or accidents.

In the problem-identification type of analysis, the datum sought, be it accident frequency, rate, or severity, is always related to the accident experience for a given set of conditions. Also implicit in this question is the comparison with a certain baseline to determine, first, whether a safety problem exists for the given set of conditions under study and, next, the extent of the problem.

For cross-sectional evaluation, the unit of analysis is again location or accident. The purpose of the analysis can be grouped into two general categories:

- *Comparative evaluation.* To compare the safety performance or effects on accidents between two or more different sets of conditions; and
- *Relationship or predictive modeling.* To determine or predict the effect of certain conditions or parameters on the frequency, rate, or severity of accidents.

For longitudinal evaluation, the objective is to assess the safety effects of a given treatment before and after its implementation. From the FHWA's point of view, the unit of analysis must be location, and the purpose is always evaluation.

DATA BASES SELECTED FOR STUDY

A number of candidate data bases were identified at the federal, state, and local levels from completed studies and from ongoing data-collection efforts. The candidate data bases were then categorized according to the following criteria:

- Application (primary or secondary);
- Data base purpose (general purpose or special purpose);
- Unit of analysis (location or accident); and
- Level of detail (police level, enhanced police level, or in depth).

Application

A primary data base is one that can be used directly for safety analysis. In comparison, a secondary data base can only be used indirectly for safety analysis, as a supplement to a primary data base. Only primary data bases were included in the study.

Data Base Purpose

For a primary data base, the purpose is termed either general or special. A general-purpose data base is created for general use and not for any specific application. There are very few general-purpose data bases in existence. Most data bases are special purpose in nature, that is, they were created with a specific purpose in mind. A general-purpose data base is useful for a wide variety of applications but often lacks the specificity desired for study of particular topics or questions. It must contain a large number of data elements to be general in nature, and it may not have enough depth for specific questions.

On the other hand, a special-purpose data base contains all the required data elements for the specific question or topic under study but little else. Sometimes it may be possible for a special-purpose data base to be used for addressing other questions or topics that are similar in nature to the one for which the data base was created, but such applications are usually limited.

Unit of Analysis

The unit of analysis corresponds with the unit used for the data record. Each data record in a location-based data base contains information on a location, which may be a roadway section, a point on the roadway, or a physical feature of the roadway, such as a bridge. In comparison, each data record in an accident-based data base contains information on an accident.

Level of Detail

Police-level accident data are obtained directly from police accident reports. These reports are readily available and are maintained on a continuous basis. However, there are well-documented problems associated with police-level accident data, such as the lack of detail, inaccuracy and inconsistency in definitions and nomenclature, and differences in reporting thresholds.

Enhanced police-level accident data, as implied by their name, are still based on police accident reports but are enhanced by the addition of supplemental information and better quality control. The Fatal Accident Reporting System (FARS) is an example of an enhanced police-level accident data base in which police accident reports on fatal accidents are supplemented with additional data elements, such as vehicle, driver, and medical data.

In-depth accident data are collected by trained accident investigators in much greater detail than are the police-level or enhanced police-level accident data, and they are tightly controlled to ensure accuracy and consistency. The costs of collecting in-depth accident data are very high and thus limit the sample size of the data base. Also, because police accident reports are used as the starting point for sampling the accidents to be investigated in depth, the problem of reporting criteria remains (e.g., unreported accidents, different reporting thresholds within and among the states, etc.).

Figure 2 illustrates how the four parameters are combined into the categorization scheme. Note that there are only seven

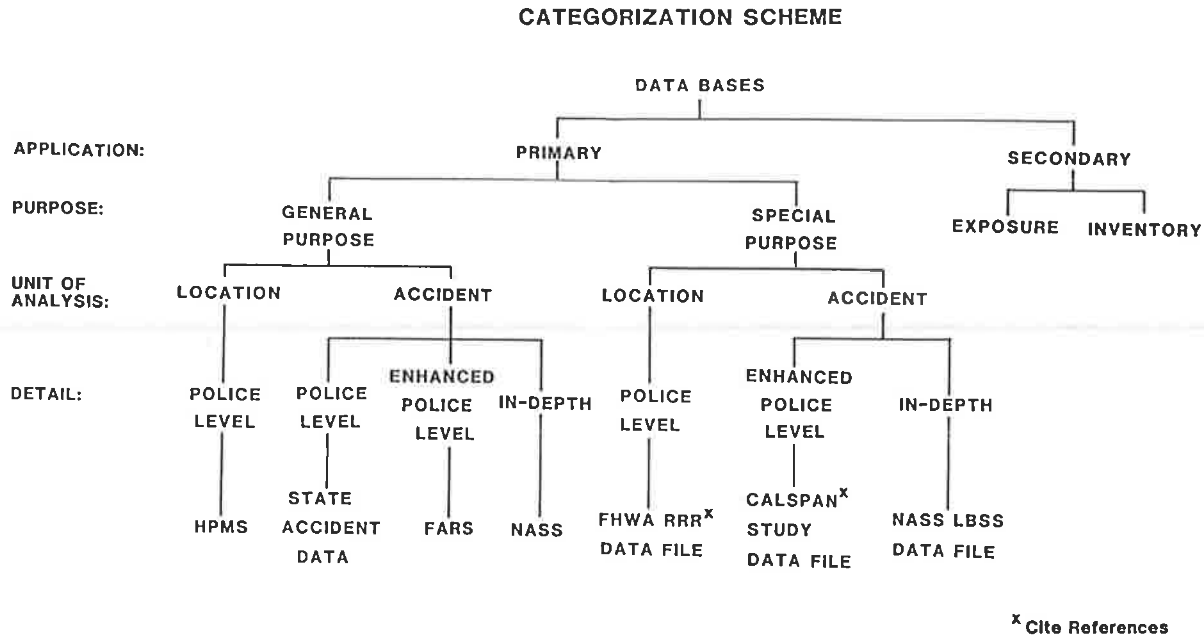

FIGURE 2 Categorization scheme for data bases.

combinations with existing primary data bases. This situation does not mean that combinations not included in the categorization scheme are inappropriate but is instead a reflection of what is currently available.

Only three general-purpose primary data bases currently exist:

- Highway Performance Monitoring System (HPMS): location-based, police-level accident data;
- National Accident Sampling System (NASS) Continuous Sampling Subsystem (CSS): accident-based, in-depth accident data; and
- Fatal Accident Reporting System (FARS): accident-based, enhanced police-level accident data.

State accident data files, particularly those integrated with road log, traffic, and other roadway and roadside data, are adaptable for use in highway safety analysis at the state level, even though they are not part of a national accident data base in the strict sense of the word. With some modifications and integration, it is conceivable that a national data base could be created from state records. State accident data files are therefore included under the category of general-purpose police-level accident data bases.

It should be noted that although the unit of analysis is shown as accident, state accident data files (and to a limited degree, FARS) can also be used for the location-based type of analysis. This is because both of these data bases are censuses of accident data, and state accident data contain location identification variables.

Three special-purpose primary data bases were also included in the evaluation for illustrative purposes:

- FHWA RRR data file: location-based, police-level accident data;
- Calspan study data file: accident-based, enhanced police-level accident data; and
- NASS Longitudinal Barrier Special Study (LBSS) data file: accident-based, in-depth accident data.

Discussions on these special-purpose data bases were presented in the final report for the study but are excluded from this article.

EVALUATION OF SELECTED DATA BASES

The state accident data bases were evaluated on the assumption that the accident data files are linked or integrated with the roadway inventory files. It is believed that such capability exists for most of the states. If the accident and roadway inventory files are computerized and have a common location reference system between the files, it is a relatively simple task to link the files together. Also, the FARS data base is considered only for fatal accidents and fatalities.

Each selected data base was evaluated on its capability to address the 16 specific types of analyses illustrated in Figure 1 for data base applicability and utility. The applicability of a data base refers to the types of highway safety analysis for which the data base can be used. The only criterion used in the assessment is whether the data base can be used for a particular analysis. No consideration is given to the utility of the data base, which is evaluated separately.

Utility refers to how good the data base is at satisfying the information needs of the applicable analyses. This quality was assessed in terms of breadth of representation, sample size, level of detail, accuracy and consistency, and flexibility.

Breadth of representation refers to the geographical distribution and sampling scheme for the data, that is, where and how the data are collected. The geographical distribution can be national, state, or local. The sampling scheme can be a census, statistical sample, or sample of convenience. A census includes the entire population of interest, for example, all fatal accidents in the nation or all reported accidents on state-maintained highways. A statistical sample allows the sampled data to be projected back to the population of interest for representative estimates. Such a sampling scheme could be based on location or accident. A sample of convenience does not provide data that are representative of the population but depends on other controlling factors for analysis.

Sample size of a data base determines the extent and detail of analysis possible with the data base, that is, the categorization of data or classification into subsets. Sample size also affects the precision of any estimates made from the data and the power of statistical tests. For an ongoing data collection effort, either the sample size available on an annual basis or that over the entire program is considered in the evaluation.

Level of detail refers to the amount of information available from a data base. The evaluation criteria include first, the total number of data elements available per record and the number of usable data elements on highway and accident characteristics, and second, the specificity of the data elements, that is, the number of levels available per data element.

Accuracy and consistency refers to the level of quality control in the data-collection effort. In other words, are the collected data accurate, or are there major sources of inaccuracy that could threaten data validity? Also, is the quality of the collected data checked to make sure that the data are accurate and consistent?

Flexibility refers to the ease of use and integration with other data files. The ease of use is evaluated from the user's standpoint for availability, completeness, and understandability of documentation for the data base. Another consideration is the extent of data processing required to create an analysis file from the data base, including case selection and data recording or reformatting. To create a new data file for analysis, it is often necessary to integrate or merge other data files into the data base for variables that are not available from the data base itself. These evaluation criteria include the availability of identification variables for merging with other data bases and the extent of data processing required.

Assessment of the applicability of the 7 selected data bases to the 16 components of highway safety analysis is illustrated in Figure 3. Note that state accident data bases, once they have been integrated with roadway inventory data, are the only data bases that are applicable to all components of highway safety analysis. The three national general-purpose data bases and the special-purpose data bases are limited in their applications.

HPMS provides extensive data on the physical and operational characteristics of the sample panels. The data are summaries because they apply to the entire sample panel, which can be several miles long and vary in length between sample panels. Specificity is thus limited to summary variables. Accident data associated with the HPMS panels are quite limited and provided in summary form only: the data consist solely of counts of total, fatal, and nonfatal injury accidents. Applicability of the data base is thus limited to location-based analyses on systemwide problem identification, cross-sectional evaluation, and possibly longitudinal evaluation. The data base is not applicable for many analyses that are accident based or site specific.

State accident data bases are applicable to all components of highway safety analysis once they have been integrated with the roadway inventory data. As mentioned previously, it is believed that most states have the capability to merge accident and roadway inventory data files, even though these files may not currently be integrated. Without roadway inventory data, the state accident data bases will be limited to only accident-based analyses.

FARS is primarily applicable to accident-based systemwide problem identification. The applicability of the data base is severely limited because only fatal accidents are included, so there is no basis of comparison in terms of severity. Also, there are no built-in exposure data to calculate fatal accident or fatality rates. Information from other sources will be necessary if the FARS data are to be used for any type of safety analysis other than problem identification.

NASS CSS data are not applicable for location-based analyses or for accident-based, site-specific problem identification because the data base has no location information. In addition, the data base does not have any exposure information for calculation of accident rates.

Assessment of the utility of the seven selected data bases can be seen in Table 1. It should be emphasized that the evaluation is based on available documentation and related publications gathered and reviewed in the study and not on actual processing and application of the data bases. Some of the evaluation criteria are rated subjectively on a simple three-point scale of poor, fair, and good.

HPMS is a national data base with information on approximately 100,000 roadway sections from all 50 states, the District of Columbia, and Puerto Rico. The roadway sections are sampled on the basis of a statistical scheme so that national estimates can be made from the data. The large sample size should allow for very detailed analysis. The level of detail is fair to good on highway-related data elements, with information available on general roadway and traffic characteristics. However, because the highway data elements are applicable to the entire section, which may be several miles long, the specificity of the analyses must be general in nature. The level of detail on accident data elements is very poor because only summary accident data are available.

The data are submitted by the states, and the extent of quality control is somewhat limited. Thus the accuracy and consistency of the data base are judged to be only fair. On the basis of review of the documentation available for the data base, the data-processing requirements for the data base appear to be fairly complicated and are thus rated fair. The data base does have the capability, albeit indirectly, of merging with other data bases by using a location-matching process.

State accident data bases are maintained by each state and are a census of all reported accidents. The sample size varies by state and is very large for analysis purposes. If it is assumed that the accident and roadway inventory data are integrated, information on general roadway and traffic characteristics is available, although the level of detail varies from state to state. For example, the Utah accident data base is a fully integrated system and contains much more information than the Texas accident data base, which has only roadway inventory data merged with the accident data.

Police-level accident data are limited in detail and are subject to inaccuracies in areas such as location identification and definitions or interpretation of data elements. Quality control of the data generally ranges from poor to fair. Data processing for state accident data bases requires large computer facilities

Analysis Characterization Scheme				General Purpose Data Base				Special Purpose Data Base		
Type of Analysis	Unit of Analysis	Purpose	Specificity	HPMS	Integrated State Accident Data Bases	FARS	NASS CSS	FHWA RRR Data File	Calspan Study Data File	NASS LBSS
Problem Identification	Location	System Wide	Highway	●	●					
			Accident		●					
		Site Specific	Highway		●					
			Accident		●					
	Accident	System Wide	Highway		●	●	●		●	●
			Accident		●	●	●		●	●
Cross-Sectional Evaluation	Location	Comparative Evaluation	Highway	●	●					
			Accident		●					
		Relationship/ Predictive Modeling	Highway	●	●					
			Accident		●					
	Accident	Comparative Evaluation	Highway		●		●		●	●
			Accident		●		●		●	●
		Relationship/ Predictive Modeling	Highway		●		●		●	●
			Accident		●		●		●	●
Longitudinal Evaluation	Location	Evaluation	Highway	●	●			●		
			Accident		●			●		

FIGURE 3 Applicability of selected data bases to highway safety analysis.

TABLE 1 UTILITY OF SELECTED DATA BASES FOR HIGHWAY SAFETY ANALYSIS

	General-Purpose Data Bases				Special-Purpose Data Bases		
Evaluation Criteria[a]	HPMS[b]	State Accident Data	FARS	NASS CSS	FHWA RRR Data	Calspan Study Data	NASS LBSS Data
1. Breadth of representation							
a. Geographical representation	National	Individual state	National	National	11 states	6 states	National
b. Sampling scheme	Statistical	Census	Census	Statistical	Convenient	Convenient	Convenient
2. Sampling size	~100,000 sections	Varies by state	~45,000 accidents/yr	~12,000 accidents/yr	196 sites (as of 5/83)	7,972 accidents	~300 accidents/yr
3. Level of detail							
a. Highway	Fair/good	Fair/good	Poor	Fair	Fair	Fair/good	Good
b. Accident	Poor (summary data)	Fair	Fair	Good	Poor (summary data)	Fair/good	Good
4. Accuracy and consistency	Fair	Poor/fair	Fair/good	Good	Fair	Fair/good	Good
5. Flexibility							
a. Data processing	Fair	Fair	Good	Poor/fair	Good	Good	Poor/fair
b. Integration	Yes	Yes	No	No	No	No	No

[a]Evaluation criteria 3–5a were graded on a three-point scale: Poor, Fair, Good.
[b]The evaluation is based on the assumption that the state accident and roadway inventory data are integrated in the data base.

because of the massive amounts of data, but the process itself is fairly straightforward. Merging with other data bases is achieved through a matching process that is based on location or other identifiers, such as vehicle or driver license numbers.

FARS contains a census of all fatal traffic accidents in the United States, which is approximately 45,000 accidents per year. The file size is large enough for most analyses but not for analyses involving rare events (e.g., accidents involving crash cushions) or great specificity (e.g., vehicles of certain year, make, and model). The level of detail is similar to that available from police-level accident data. The key advantages of the FARS data base over state accident data bases are the improved accuracy and consistency of the data and the use of standardized coding formats for all states. The FARS data base is fairly simple to process and use. However, it is not possible to merge the FARS data base with any other data base because all identifiers are deleted from the data.

NASS CSS data contain a statistical sample of reported accidents, designed to provide national estimates of accident statistics. Accidents are selected by a method of disproportionate probabilities from 50 localities, known as primary sampling units (PSUs), within the continental United States. Note that not all of these 48 states are included in the sample, but the PSUs are scattered across the country. About 9,000 accidents were sampled each year in the Continuous Sampling Subsystem (CSS) during 1982–1984, and a larger sample size of 12,000–13,000 accidents was collected in later years. The sample size is adequate for making general national estimates but may become insufficient when greater specificity is needed.

The level of detail available on accident data elements is extensive but fairly limited for highway-related data elements. The data are quality controlled exhaustively for good accuracy and consistency. The data base is somewhat difficult to learn and understand for users who are not familiar with the NASS program because of the complexity of the program and the voluminous documentation.

There is also a problem with incompatibility between data from the early years (1979–1981) for some of the data elements, a problem that requires extensive recoding and reformatting to merge between years. This problem of incompatibility between years has been resolved for the years 1982–1984, although a major revision was implemented in 1985. It is not possible to merge the NASS data base with any other data base because all identifiers have been eliminated from the data.

CONCEPTUAL ALTERNATIVES

Numerous conceptual alternatives to improve and enhance the capabilities and utilities of the existing general purpose data bases were considered. Some were rejected out of hand for being infeasible or impractical and others for not being cost effective. Brief discussions on some of the alternatives that were rejected will be presented in this section.

The ideal alternative is to have a single data collection system that would satisfy all the information needs for highway safety analysis. By modifying and integrating the four existing general-purpose data bases, a single data base could, in theory, be created for this purpose. The individual components would serve different functions but would complement each other, so that data needs for various safety analyses might be satisfied by using one or more of the individual components. This alternative would require major, fundamental changes to the various data collection systems, both technically and administratively. In theory, this might be a feasible and even desirable concept, but it is obviously not a practical alternative.

The FARS and NASS data bases are accident based and lack the capability of merging with other data bases. It is not possible to modify these two data bases to a location-based system without totally redesigning the data collection systems. Any improvements or enhancements short of major redesign and restructuring of the data collection systems would not extend the applicability of these two data bases beyond that of an accident-based analysis. Therefore, no alternative was considered for the FARS or NASS data bases. It follows that any alternative to enhance and improve existing data bases to meet the needs of highway agencies would evolve from the HPMS and state accident data bases.

The HPMS data base is location based, and the panels or highway sections are sampled on a statistical basis to provide national representativeness. The data base has a large enough sample size to handle most of the highway safety analyses of interest and the capability of being merged with other data bases, albeit indirectly. However, the HPMS data base also has some major shortcomings:

- Only summary accident data are provided; and
- The roadway and operational data elements apply to the entire sample panel, which varies in length, resulting in a lack of specificity for some of the data elements.

The lack of detailed information on accident data could be remedied by merging the state accident data files with the HPMS data base through a location-matching process. With an appropriate system design, the merged data base could provide for both location- and accident-based analyses.

One alternative is to keep the current HPMS data base unchanged. Accident records would be matched with the HPMS panels through a location-matching process on an individual state basis. No modification would be made on the accident records. This alternative would be easy to implement with a minimum of effort, provided that the states already have the capability to merge their accident files with the HPMS files. Unfortunately, this is not currently the case, which means that a merging process would have to be developed for those states that do not have this capability.

If it is assumed that such a merging capability is developed, the users would have to extract the required data from the merged data base to create an analysis file suitable for use with the intended analysis. The burden of converting the merged data base into a usable analysis file would be borne by the users, a factor that would probably discourage the use of this data base. The level of detail would be limited to that available from the HPMS data base and police-level accident data, and anyone attempting an analysis that required greater detail or specificity would have to resort to special data files created for that purpose.

Another alternative is to create a safety analysis subsystem within the HPMS data base. Appropriate modifications would be made to the HPMS and state accident data collection systems to create a data base that would be suitable for safety analysis. This would involve changes to the HPMS data elements and the state accident data records. The users would be provided with a single safety analysis data base for analytical purposes, with a minimum of data manipulation required. The level of detail is again limited to that available from the HPMS data base and police-level accident data, so special purpose data files would have to be created for any analysis that required more details or greater specificity.

The major obstacle to setting up a single accident data base is the wide variation among the states in their accident report forms and reporting thresholds. The accident report forms vary among the states in terms of available data elements, format, definitions, and coding levels. The effort that would be required to merge all the state accident data bases into a single standardized format is monumental. In some past and ongoing research studies, several state accident data bases were merged into a single data base for the purpose of analysis. The process was very tedious and time consuming. Moreover, the level of detail on the data elements for the merged data base is usually reduced to the lowest level available from the individual states.

The use of varying reporting thresholds among the states is even more problematic. The reporting thresholds used in most states are based on certain levels of property damage (e.g., $250 or above), towed vehicles, or injury to one or more of the involved occupants. However, the reporting thresholds vary among the states; a given accident might be reported in some states but not in others. It is clear that such differences in reporting thresholds would introduce unknown biases into the data base.

The ideal solution would be to have the states standardize their accident record systems by using a standardized accident reporting form and the same reporting threshold, thus eliminating the problem totally. A lot of effort has been devoted to this goal through the years, with only limited success, and there is no reason to believe that the situation will improve in the foreseeable future.

The problem with the lack of uniformity and compatibility in the accident reporting form could be minimized by using an approach similar to that of the FARS system. A subsample of accidents (e.g., a total of 250,000 accidents) occurring within the HPMS panels could be selected on the basis of a statistically representative scheme. These sample accidents would be recoded onto a standardized form and entered into a single data base. Supplemental information on accident, roadway, traffic, and other data elements could be added to the data base. More rigorous quality control checks could be instituted to improve the accuracy and consistency of the data. In short, the data quality would be improved to that of the enhanced police level.

The extra human resources required could be provided through contracts with appropriate state agencies. State personnel, paid by the contracts, would be used for the coordination, recoding, collection of supplemental data, quality control, and data entry functions. The data would then be compiled at a centralized location to create the data base. The number of accidents included in the data base would be a function of funding available.

The problem with differences in reporting thresholds among the states is much more difficult to resolve. Any changes in the reporting thresholds would probably require legislation at the state level. Also, there are variations even within a state: for example, some major metropolitan areas have adopted the policy of reporting only injury and fatal accidents, leaving the reporting of property damage–only accidents to self-reporting by drivers.

Another drawback is the lack of specificity for some roadway data elements. This problem occurs because the data elements apply to entire HPMS panels, which vary in length up to several miles. This problem could possibly be alleviated by merging state roadway inventory files into the HPMS data base or by subdividing the HPMS panels into shorter sections of equal length. Either approach would require considerable effort.

Integrated state accident data bases can be used for both accident- and location-based analyses through a location-matching process. In theory, a national data base could be created by merging across the states. This data base would be applicable to all components of highway safety analysis, as discussed previously. As a census of all reported accidents in the nation, the sample size would be enormous.

The problem of lack of uniformity and compatibility among the states in their accident and inventory record systems makes the merging of state accident data files into a single data base a monumental task. Also, the data base would be too large for most applications. Extensive computer facilities would be required, and the associated data processing costs would be very high because of the large number of records. This would not be a viable alternative, and thus it is not considered any further.

RECOMMENDED ALTERNATIVE

The alternative that was found to be most cost effective and that is thus recommended is to select a small number of states (e.g., four) with integrated accident data bases and merge them into a single data base. The states would be selected on the following criteria:

- Geographical representation;
- Existing ability to integrate the accident data files with the roadway inventory, traffic, and other pertinent data files; and
- Compatibility among the states in their accident and roadway inventory record systems.

Geographical representation is the major concern with this approach, due to the possible lack of credibility because the combined data base does not provide true national representation. This concern can be partially alleviated by dividing the nation into four regions and then selecting one state from each region. Even so, the extrapolation of the analysis results to states other than those included in the data base is inferential and not statistical in nature. Caution should be exercised in interpreting the analysis results, especially if the analysis of interest is susceptible to the influence of regional characteristics, such as weather and terrain conditions, design standards, ages of the facilities, and so forth.

The ability to integrate accident record systems is a requirement of the candidate state data bases. A number of states have established or are in the process of developing integrated accident record systems; examples include Michigan, Montana, New York, North Carolina, Ohio, Texas, Utah, and Washington. This list is by no means all-inclusive, and the degree of integration varies among the states. Of course, it would be desirable to select systems with as high a degree of integration as possible.

Compatibility in terms of the data elements and reporting thresholds among the selected states is important in order to minimize the effort required to merge the state data files into a single data base. It is recognized that true compatibility is not attainable at this time. However, because the number of states involved is small, a high degree of compatibility could be achieved. Also, there is a better chance of improving the degree of compatibility by working closely with the states and possibly by staging demonstration projects.

Another consideration is the relative size of the individual state data bases. It is conceivable that the size of one state, in terms of number of highway miles and accidents, could be so great relative to the other states in the data base that the analysis results would be heavily biased toward that state. This problem could be minimized by selecting states with roughly equal proportions. On the other hand, it may be desirable to have each state in proportion to the relative size of the region in which it is located so that the data base would be more representative on a national basis.

The level of detail of the accident data is limited to police report level, while the level of detail on the roadway and traffic data elements is restricted to what is available from the state roadway inventory files. Such levels of detail are usually adequate for analysis of a general nature but not for specific applications.

It may be desirable to supplement the roadway inventory data with a limited number of data elements that are deemed essential but are unavailable from the existing inventory files in one or more of the states. However, use of this option should be kept to a minimum and should preferably be performed on an ad hoc basis. There is always a tendency to try to satisfy the information needs of as many users as possible. This effort could result in the inclusion of too many data elements that are only used infrequently, and the cost for the data collection and processing could be increased substantially without a corresponding increase in the benefits.

It is probably more cost effective to maintain in the data base on a continuous basis only those data elements that are most essential or those that are currently available from the state data files. Then, special studies could be conducted for particular applications that require more details than are available from the continuous data base.

Data quality would be subject to the same problems as in the individual state data bases because no additional quality control checks are incorporated. One way to improve the data quality to that of enhanced police-level data is by recoding the data and instituting additional quality control checks. The costs associated with this enhancement are substantial and probably not cost effective.

The privacy and accessibility of the data base is another area of concern. Tort liability has been a growing concern among state governments in recent years. It may be difficult to enlist the cooperation and assistance of the states without considerations for the privacy and accessibility of the data. It would be ideal if the data base could be protected under some form of legislation and be restricted to research and development applications only. Otherwise, safeguards against requests by attorneys or nongovernmental agencies through subpoenas or the Freedom of Information Act should be developed to maintain data privacy. The safeguards should also be extended to limit the accessibility of the data base to authorized users only. At a minimum, the data must be sanitized to remove identifiers relating to individual accidents.

A rigorous validation procedure is needed to double-check any analysis results derived from the data base. As an example, suppose that a predictive model is developed by using a data base that includes data from four states. The model should be applied to each of the four states individually to check whether the results are consistent across the states. It would also be desirable to apply the model to one or two states that were not included in the data base as an external check. If the results were consistent across all of the states tested, the results would probably be applicable to other states. However, if there were wide variations in the results among the states, it would be necessary to reanalyze the data to determine the cause(s) of such variations and to develop appropriate adjustment factors to account for the differences.

In other words, analysis results derived from the data base should be validated by applying the results to the individual states included in the data base and, if possible, to other states outside of the data base. This would also reduce biases that had been introduced into the results by the unequal sizes of the states in the data base. This validation process is relatively straightforward and inexpensive because it involves only the application, not the development, of the analysis results.

Proper administration of the data base is essential to its success. The data base should be administered through a single agency, be it the FHWA or a contractor to the FHWA. The data base manager has to have intimate knowledge of the individual state data bases and must constantly keep abreast of any new developments in the states. The manager should also have a good understanding of the requirements for highway safety analysis so that he or she can assist the users in proper analysis of the data base. This user interface is critical to the acceptance of the data base by the users.

This alternative is relatively inexpensive and provides a reasonably good data base for safety analyses. The startup cost consists of identifying and selecting the state integrated accident record systems for inclusion in the data base and acquiring the data bases from the selected states. Because the individual state accident data bases are already integrated, it is only necessary to develop the required software to merge the individual data files into a single data base. A startup cost of $250,000 (1988) should be sufficient for the purpose.

The annual operating cost, which includes the acquisition of the state accident data bases and update of the software for merging the files, is correspondingly low. An annual budget of $200,000 (1988) should be sufficient. This estimate does not include any reporting requirements or applications.

SUMMARY

Existing data bases were identified and grouped into seven categories according to a categorization scheme. Seven data bases, one for each category, were then selected for study. The emphasis of the evaluation is on the four general-purpose data bases that are national in scope. The three special-purpose data bases are included to illustrate the use of special studies to address specific questions that cannot be answered with the general-purpose data bases.

The various components of highway safety analysis, as used in this study, are defined and characterized. The applicability and utility of these selected data bases for use in highway safety analysis were evaluated on the basis of available documentation. The three national general-purpose data bases (HPMS, FARS, and NASS CSS) and the special-purpose data bases are rather limited in their applications. State accident data files, when integrated with roadway inventory data, are the only data bases that are applicable to all components of highway safety analysis. The utility of the data bases varies somewhat among the general-purpose data bases, but none of them had any major problems. In devising conceptual alternatives, the emphasis is thus on improving the applicability (instead of the utility) of the data bases.

Conceptual alternatives that would improve or enhance the capabilities and utility of the existing general-purpose data bases to better serve the information needs of highway safety analysis were developed. These identified alternatives were studied and analyzed for their feasibility and practicality, and appropriate recommendations were made.

The recommended alternative involves the selection of a small number of states (e.g., four) with existing integrated accident data bases and merging these data bases into a single data base. Geographical representation is approximated by dividing the nation into four regions and then selecting one state from each region. Because the individual state accident data bases are already integrated, it is only necessary to develop the software required to merge the individual files into a single data base. A startup cost of $250,000 (1988) is estimated. The annual operating cost is estimated at $200,000 (1988), including the acquisition of the state accident data bases and update of the software for merging the files.

The recommended data base would provide, at a reasonable cost, the needed information for most highway safety analyses of a general nature from the standpoint of FHWA. It should be recognized, however, that the data base also has its limitations. Some analyses are better addressed with specially designed studies, and others are simply not answerable with accident analysis.

EPILOGUE

A functional prototype of a merged multistate integrated accident data base, the recommended alternative of this study, is being pursued in a current FHWA research contract (*2*). The findings of this effort will be used to determine the feasibility of developing such an operating system.

The FHWA decided to delete accident data from the HPMS system, partially on the basis of the results of this study. In an unrelated event, the National Highway Traffic Safety Administration decided to restructure the NASS CSS effort during 1988. This restructured NASS CSS program is limited to data on passenger vehicle and light van crashworthiness. The applicability of the resulting data file to highway safety analyses is much less than the former NASS CSS system. These two events increased the importance of the development of an alternate source for national-level highway safety data. The system recommended in this paper is one such alternative.

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Accident Data as a Tool for Highway Risk Management

DANIEL S. TURNER AND CECIL W. COLSON

Transportation agencies at all levels of government have experienced a rash of suits involving alleged negligence. The devastating increase in the number of suits and in the magnitude of financial losses has been overwhelming to many of these transportation agencies. In an effort to minimize these losses, many agencies have organized "risk management" programs. The risk in this case is the probability that the agency will be sued following a highway accident. If all highway accidents could be eliminated, the risk would become zero. Because this is impossible, the next most desirable option is to reduce the number of accidents (especially high-severity collisions) and thus reduce the probability of being sued. Accident data offer an excellent technique for reducing risk by identifying those sites that are of greatest risk to the motorist and thus most deserving of safety treatment. In this paper, several innovative accident data programs are described, and sample computer listings of accident data for several of them are presented. The federal aid safety program, accident inventory listings, high-accident locations, wet pavement accidents, daylight-dark accidents, roadway defect investigations, high-exposure accidents, railroad grade crossings, roadside objects, and bridge collisions are a few of the topics included.

America's romance with the automobile has recently taken an ugly turn, and transportation agencies at all levels are suffering the legal consequences. Why are agencies currently so vulnerable to litigation? How can they protect themselves in the future? In this paper, a promising risk-reduction procedure is offered as a solution.

A NATION ON WHEELS

The classic American love affair may be condensed to two simple elements, a man and his car. We have become a people born to travel and have come to feel that we have a "constitutional right" to our individual mobility. Fifty years ago there was an average of almost five people in every automobile on the road. For today's typical trip to work, there are only 1.3 of us per vehicle (*1*). Other data show that we had 159 million drivers in 176 million vehicles traveling 1.8 trillion miles in 1986 (*2*). That means that for every five men, women, and children in the United States, there were four registered vehicles. Over 70 percent of the American population, regardless of age, was registered to drive. Americans love to travel, and they spend an average of $3,000 per year on each automobile in this country (*3*).

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Disadvantages of Travel Mania

There are dark sides to this frenzy for travel. First, 45,600 were people killed on our highways in 1985, and an additional 1.7 million suffered disabling injuries (*4*). Second, steady increases in vehicular travel have been the norm for almost 50 years, and the public has exerted an ever-increasing demand for more, better, and safer roads. Almost all state transportation agencies are in a mad scramble to find sufficient funds to maintain existing roads, provide new roads, and improve the safety of their highways.

Growing Threat to Transportation Agencies

Transportation agencies are under the shadow of another dark cloud. The specter of tort liability has raised an unparalleled threat of financial devastation in the courtroom. The number of suits against transportation agencies and the consequent financial losses have skyrocketed. Our nation's inherent belief in the right to travel is being paralleled by another inherent belief, the right to sue.

A few examples will illustrate the severe nature of the problem. Almost all states enjoyed sovereign immunity in the 1960s, but over the next 20 years, this status was overturned in the majority of states. Today, only a handful of states still enjoy

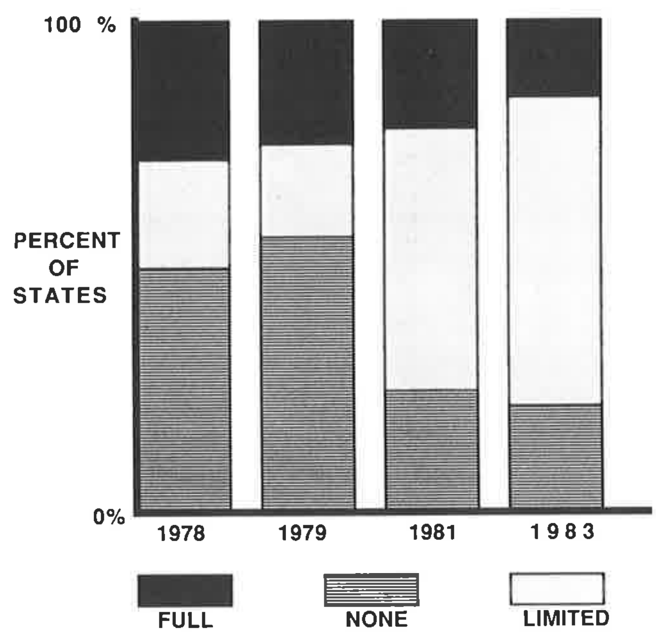

FIGURE 1 Survey of sovereign immunity (*5*).

sovereign immunity, although many of those who lost it have since found ways to return to limited immunity (claims courts, etc.). Figure 1 indicates the trend over several years, with more and more states being forcefully converted from full immunity to limited or none. In 1983 an AASHTO survey estimated pending tort liability claims reported by 40 states at almost $7 billion (*5*). Almost 7,000 suits were filed in 1982 alone, as shown in Table 1.

In the 5 years after loss of sovereign immunity, the Pennsylvania Department of Transportation spent $80 million defending and settling tort liability actions. By 1985, the amount was more than $20 million for a single year. A similar story may be told about Louisiana. In 1984, initial judgments and settlements against the Department of Transportation (LDOT) reached $38 million, while interest on these losses cost another $14 million (fortunately, several of these judgments were overturned on appeal). Table 2 gives a very revealing look at the nature of the claims against LDOT during 1979–1983. The types of suits in Louisiana are to a large extent representative of those all across the nation.

The usual assertion in these suits is that the governmental unit has failed to perform its duty in a reasonable manner, that is, it was guilty of negligence. On the basis of the data in Table 1, it would seem that 157 Louisiana plaintiffs have claimed that a lower shoulder or shoulder dropoff constituted a hazardous condition that caused or contributed to their collisions and the Louisiana DOT was negligent in allowing the shoulder condition to exist and contribute to the accident.

DUTY OF GOVERNMENT TOWARD "SAFE" ROADS

Because the failure of governmental units to perform their duty in a reasonable manner (negligence) is the basis for many suits, that duty needs to be understood by governmental employees. The function of government is to provide security and services

TABLE 1 CLAIMS AND SUITS FILED AGAINST STATE TRANSPORTATION AGENCIES (*5*)

STATE	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982
AL						233				262	172
AK							163	157	152	37	63
AZ	71	100	70	90	98	282	291	321	293	293	214
AR	125	98	100	83	132	132	200	122	154	170	165
CA	870	1,013	1,042	1,239	3,533	3,575	1,818	2,079	489	523	444
CL	2	8	7	11	7	23	12	65	55	89	126
CO	286	300	376	414	453	583	164	150	126	900	1,200
DE											
FL		5	65	71	35		447	189	89	92	73
GA							8		4		
HI							57	96			
ID	19	65	93	117	108	48	181	178	210	223	193
IL		9	30	65	142	60		118		45	114
IN	72	88	308	435	622	702	653	828	599	607	773
IA	80	108	90	129	126	136	152	185	338	184	182
KS					16	17		6	8	11	12
KN	242	249	241	275	250	333	405	590	383		
LA	22	80	111	118	145	147	420	612	520	448	514
MA							2	5	8	28	6
ME							5		6,277		
MS							121	60	150	150	166
MI				201	108	95	114	126	110		
MN					42	165	192	211	162	133	181
MS											
MO	0	3	2	1	1	3	2	15		27	39
MT							90	64			
NB	34	64	38	44	40	42	50	101	65	63	92
NV	20	19	22	19	13	40	8	43	69		
NH	2	3	0	2	4	1	8	8	7	26	24
NJ											
NM			13	9	17	18	21	101	10	20	30
NY		151	162	158	130	226	202	256	302	326	
NC											
ND											
OH				92	142	226	200	158	129	130	128
OK									14	8	7
OR							30	39		466	588
PA							212	622	520		
RI								14		100	100
SC	265	302	364	366	406	407	295	571	659	319	372
SD	0	0	1	0	1	1	0	2	2	0	0
TN							275	447			400
TX	12	15	19	41	28	27	37	37	59	58	69
UT						10	52	62		16	4
VT									90	90	90
VA							5		5		
WA	45	58	69	66	77	70		314	867	64	88
WV							200	282	280	308	228
WI	1	2	5	6	4	4	8	128	62		
WY			2	1	0	1	4	0	9	55	72
TOTAL	2,168	2,740	3,230	4,053	6,680	7,607	7,104	9,362	13,276	6,271	6,929

TABLE 2 LOUISIANA DEPARTMENT OF TRANSPORTATION SUMMARY OF CLAIMS RELATED TO HIGHWAY TORT LIABILITY FOR 1979–1983 (*6*)

Condition	Claim Amount ($)	No. of Claims
Shoulder	203,935,706	157
Design, etc.	201,049,525	107
Surface	123,683,633	161
Work site	121,102,215	107
Signs	94,664,421	96
Property	94,365,486	45
RR crossing	59,835,430	39
Bridge	59,713,449	55
Drainage	48,569,651	16
Signal	36,309,772	126
Marking	29,136,161	26
Sight distance	27,425,450	23
Traffic control	26,125,700	7
Maintenance	24,816,773	28
Left turn	10,893,211	18
Lighting	7,614,655	14
Equipment	6,400,870	4
Debris	6,386,497	13
Ferry	5,204,479	3
Mowing	4,062,350	4
Guardrail	3,511,109	6
Tunnel	2,350,000	1
Other	2,000,000	1
Steel cable	1,110,000	2
DOTD operator	227,000	1
Under $100,000	286,867	9
Total	1,200,780,410	1,069

for its citizens. Transportation is one of the services that governmental officials and employees are charged with providing. Normally, the goal of governmental transportation efforts should be the safe and efficient movement of people and goods, within reasonable fiscal constraints.

The courts have universally held that although governments are providing these transportation services, governments are not the absolute ensurers of the safety of a highway user. The total resources of any government are limited, and it would not be realistic to expect that the bulk of all funding be devoted to keeping the roads in an absolutely sound and safe condition. However, the courts have consistently held that governments are required to maintain streets and roads in a *reasonably safe* manner. Failure to do so may result in liability if a user suffers injury.

NEGLIGENCE

Negligence is generally defined as failure to use reasonable care in dealing with others. In other words, what would a reasonable person have done in the circumstances and situations that constitute the current court case? Negligence in one form or another is usually the key to tort liability cases, and officials should understand its general principles and applications. To win a negligence case, the plaintiff must prove that

1. The defendant had a duty of reasonable care toward the plaintiff;
2. The defendant breached that duty;
3. The defendant's negligence was the cause of plaintiff's injury;
4. The plaintiff was not guilty of contributory negligence that caused the injury (or was guilty of comparative negligence in some states); and
5. The plaintiff incurred resulting damages.

Officials should be interested in breaking the five-step chain of factors. Removing all negligence (the second factor, or link) would be the ideal way to prevent highway-related tort liability losses. The best defense to a lawsuit is a preventive defense.

EACH TRAFFIC ACCIDENT IS A POTENTIAL SUIT

Roadway liability almost always begins with a traffic accident. Each accident victim is a potential plaintiff in a lawsuit, and there are a great many of them. Most of us are overwhelmed when we first learn of the magnitude of traffic accidents in a typical year.

Over the last 40 years, there have been 40,000 to 50,000 traffic accident fatalities each year, and 1 to 2 million people per year have been injured. Fortunately, there have been decreases in both the number of people killed and the rate of fatalities per million miles driven; however, it would be wise to remember that there were 46,400 fatalities, 1.7 million disabling injuries, and 33 million drivers involved in accidents in 1985 (*4*). That amounts to millions of possible plaintiffs from suits generated by traffic accidents.

RISK MANAGEMENT PRINCIPLES

Because of the rapid increase in suits and the corresponding increase in financial losses, most transportation agencies have looked for ways to minimize their losses. The concept of risk management has been borrowed from the private sector. A successful risk management program involves the implementation of both risk finance (insurance) and risk control techniques. A well-designed risk management program achieves the following important goals:

- Minimize the potential number of lawsuits being filed;
- Minimize the number of lawsuits lost; and
- Minimize the damages from lawsuits lost.

Risk finance techniques (insurance) are generally most useful in achieving the third goal, which involves minimizing monetary damages to the agency from lost lawsuits. Risk control techniques, on the other hand, are useful in achieving all three goals. Risk control involves

1. Identifying the risk;
2. Measuring the risk (probability, severity, frequency);
3. Putting a plan in place to reduce or control the risk; and
4. Monitoring and adjusting the plan.

Many transportation agencies have recently attempted to minimize their liability through risk control. In general, this involves setting up a program specifically aimed at recognizing and reducing liability factors. Several of the most frequently used procedures will now be briefly outlined.

Importance of Good Records

The ideal situation for any agency under suit is for the plaintiff's attorney to discover documentation in the defendant's files that proves the defendant's position. The chance of this occurrence is greatly increased if the transportation agency is careful to keep accurate, complete, and timely records of its actions. These records are especially important when the agency deals with individual members of the public (complaint calls, requests for service) or when the agency deals with chronic problem areas (continuous congestion, chronic maintenance problems, etc.).

A good transportation manager can periodically review the agency's records to learn which departments and which employees are conscientiously executing their duties. The records serve not only as potential evidence for the courtroom but also as a tool for the agency manager.

Accident data make up one type of record of great importance in establishing and conducting a risk management program. Before discussing the use of accident records, several other types of records will be outlined.

Notice of a Defect and Documentation of Complaints

Once a public entity has received notice of a defect, it has a duty to repair the defect or to warn the public until the defect can be repaired. A prominent part of the plaintiff's negligence case is often an attempt to prove that the highway agency had notice of a defect.

The notice of a defect can take place in three ways. First, it can be actual notice, such as a complaint call. Second, it can be constructive notice. That is, a defect could exist long enough that a reasonable person would have found it. Third, the agency may receive notice if its own actions (improper repair, etc.) caused the defect.

Because notice of a defect is such a strong portion of a negligence case, the transportation agency should use due care in how it receives and handles complaint calls. Procedures should be set up to record key information, determine the severity of the reported defect, and take appropriate action on the defect. These records should be carefully preserved for possible later use in court.

Examples of good procedures for recording complaint call information are illustrated by Figures 2 and 3. In each case, key facts are recorded about the call (date, time, location, caller, receiver, nature of call, etc.), the name of the individual or unit to whom the request was assigned, and the disposition of the action. Both of these forms require explicit data entries.

Maintenance Records

Records of maintenance and construction activities include work undertaken, names of supervisors, materials used, and dates and times of activities. These records may later prove to be the agency's strongest allies in defending a court case. Witnesses tend to forget specific times, dates, and details, and they are sometimes tempted to exaggerate on the stand to emphasize their testimony. A good system of maintenance record keeping may often provide key pieces of data to refute erroneous testimony or to strengthen the defense's case by giving specific facts to the jury. These records may also be used to establish that the agency took reasonable action in addressing a specific problem at a specific site.

Inventory Records

Future suits may be deterred by recognizing existing defects through field inventories and then removing the defects. For example, signs and traffic signals are two items that are frequent topics of suits. The highway agency might prevent many future suits by carefully comparing each existing sign to the *Manual on Uniform Traffic Control Devices* (*8*). Once the inventory is completed, the agency should routinely replace those signs and signals that do not meet requirements, updating the inventory as they do so.

Other types of inventories are also useful in court. Video logs and photo logs are two that are often used as evidence in support of the defense.

ACCIDENT DATA AND ACCIDENT REDUCTION PROGRAMS

The heart of any good risk management system should be a program to reduce accidents, injuries, and fatalities. Realistically, it must be recognized that all traffic accidents can never be eliminated, but it may be possible to decrease the number of collisions by altering the roadway environment. Specifically, emphasis should normally be placed on improving situations and locations that have demonstrated potentially high risk to the motorist.

The accident reduction program might proceed in the following manner:

1. Ensure that local police know why accident data are needed, that accident reports are correctly filled out, and that they are filed in a manner that facilitates cross-classification and retrieval;
2. Prepare a high-accident situation or location list;
3. Look for patterns of accident types and causes;
4. Develop alternative corrective measures for each site and determine the most cost-effective treatment;
5. Develop a priority list among competing sites and program corrective actions on the basis of the list;
6. Erect warning signs at sites that cannot immediately be repaired or take routine maintenance actions to improve safety at the site;
7. Review projects after completion;
8. Periodically reassess the priority list and the need for warning or minor improvements at sites not yet completed; and
9. Keep good records of all portions of the program.

Obviously, there are many details that might be added to this list to specify the manner in which the individual tasks are performed. The details vary with the type of highway, degree of hazard, and other factors.

High-accident locations can be identified by reviewing accident data. In the simplest case, police accident reports may be examined and accident locations marked with pins on a street map. On the other hand, most transportation agencies have

STATE OF ALABAMA
HIGHWAY DEPARTMENT
INCIDENT REPORTING/RESPONSE RECORD

___ ___ ___ ___ ___ ___ ___
Div Dist Seq.No.

TYPE INCIDENT:

Complaint or Request
___ Routine Maintenance
___ Hazardous, Routine Repair
___ Hazardous, Safety-Related

___ Catastrophic Event*
___ Hazardous Material Accident*
___ Natural Disaster*
___ Other (describe)________________

HAZARDOUS MATERIALS ONLY
___ State Troopers Notified
___ ADEM has been Notified
___ Division has been Notified
___ Highway has been closed
___ Highway Dept. assisted in Clean-up

* Report immediately to Division

DATE:________ TIME RECEIVED:________ RECEIVED BY:________

REPORTED BY:________________ TELEPHONE NO.:________

HOW REPORTED:________________

LOCATION:________________

DESCRIPTION:________________

ASSIGNED TO:________ DATE ASSIGNED:________ TIME ASSIGNED:________

ACTION TAKEN:________________

DATE COMPLETED:________ TIME COMPLETED:________

STATUS WHEN CLOSED OUT:
___ Completed ___ Scheduled As Routine Maintenance
___ No Action Required
___ Further Study, Forwarded To: ________
___ Other (describe):________

FIGURE 2 Complaint form used by the State of Alabama Highway Department.

automated records of accidents and use computers to determine high-accident locations. There are excellent computer programs for use in accident reduction efforts, including those that calculate accident rates for all state routes, county and city accident totals, high-accident locations, and collision diagram information.

Once the high-accident situations or locations are known, patterns of accidents should be identified and matched to causes, if possible. This process may be as simple as reviewing a few reports to see the types of accidents that occur at an intersection, or it may require using supporting data (collision diagram, condition diagram, traffic counts, warrant analysis,

METROPOLITAN DADE COUNTY, PUBLIC WORKS DEPARTMENT

W.O. 2700 *SERVICE REQUEST*

LOCATION:

01 REQUEST NO 07

8 DATE REC'D | SEC | AVENUE | TYPE | STREET | TYPE | JUR | AVENUE

STREET | OTHER | 8 DATE DUE

REQUEST RECEIVED FROM: RECEIVED BY: ______

NAME ______ ORGANIZATION ______

ADDRESS ______ TELEPHONE NO. ______

VIA: ☐ TELEPHONE ☐ LETTER ☐ MEMO ☐ OTHER ______

DESCRIPTION OF REQUEST: ______

HISTORICAL FILE REVIEW FOR ANALYSIS WORK:

PREVIOUS WORK: ______

VOLUME COUNTS ON FILE: (Date, Type, Data) ______

ACCIDENT ANALYSIS: ______

COMMENTS: ______

UNIT ASSIGNED ______

DATE ASSIGNED 8 MANHOURS

DISPOSITION OF REQUEST:

☐ NO FURTHER ACTION ☐ WORK ORDER ISSUED ☐ WORK COMPLETED BY ANALYST IN FIELD

☐ WORK ORDER PREVIOUSLY ISSUED SRF# ______

☐ OUTSIDE JURISDICTION: FORWARD REQUEST TO ______

COMMENTS: ______

PROBLEM REPORTED

PROBLEM FOUND ______ 10 11

INSPECTED BY ______ DATE ______, APPROVED BY ______ DATE ______

CONTACT AND CORRESPONDENCE:

PERSON CONTACTED: ______

RESPONSE DATE: ______ TYPE: ☐ PHONE ☐ LETTER ☐ OTHER ______

13 DATE TERMINATED 8 17 | 18 COMMENTS 43

130 01.04 Rev. 12/82 PAGE 1

* SECTION CODE

NW, W	1
NE, E	2
SW	3
SE	4

STREET CODE:

Road	1	Court	4	Street	7
Ave	2	Lane	5	Drive	8
Place	3	Terrace	6	Named Street	9

FIGURE 3 Sample complaint form (7).

summary of key facts, field observations, etc.) for complex locations. Procedures for making these studies are well documented elsewhere. Likewise, processes for matching corrective measures to accident patterns and for choosing the most cost-effective improvements are well documented in the same references.

In summary, good accident reduction programs may take many different forms. Discretion should be exercised in devising a program to fit the local situation and to maximize the use of public funds.

Federal Aid Safety Program

The primary safety effort of most state transportation agencies is the Federal Aid Safety Program. Section 209 – Hazard

Elimination Program funds are used to make safety improvements at high-hazard locations. Section 203 – Rail Highway Safety Program funds are used specifically to improve grade crossings. Support for highway improvements made under these two programs is 90 percent federal funds and 10 percent state or local agency funds. Projects funded by these two sections offer prime opportunities to reduce tort liability exposure by removing roadway hazards.

Consideration of accident data is a requirement and an essential element for operating the Federal Aid Safety Program. Use of accident data to identify locations and to set priorities for the best use of safety funds is an integral portion of the safety program and thus of a good risk management program.

In addition to normal safety program uses, there are several innovative uses of accident data that may go far toward reducing an agency's tort liability exposure. The following section includes a discussion of several of these.

High-Accident Locations

A prominent part of most accident reduction programs is the deliberate, well-planned feedback of information to managers and field forces. This information is used for both general education and site-specific (or characteristic-specific) studies. It is educational in that field forces become familiar with general accident trends and characteristics in the state. This procedure allows employees to recognize unusual characteristics that should be addressed for safety treatment. Specific information is usually received in the form of lists of sites that exhibit unusual characteristics, such as a high number or rate of accidents.

The general education function can be fulfilled by summaries of accident characteristics. One example is demonstrated in Figure 4. This report is a very simple listing of the number of accidents that happened in one highway department jurisdiction. Care should be used in dispensing these types of reports. Employees should be made aware that the number of accidents is not the best criterion for selecting treatment sites. These listings do, however, allow employees to maintain a general feeling for the local accident situation.

The second type of information feedback is site specific. The best methods for selecting treatment sites are those that involve examination of both the number of accidents and the accident rate at any given site or a more sophisticated process, such as the rate/quality control technique. Figure 5 is a listing that was derived by the latter method, which produces a statistically sound sample of sites for each type of roadway and provides emphasis on individual sites that have accident numbers and rates higher than other roadways of similar character. This technique allows comparison of freeway segments to other freeway segments, two-lane rural routes to other two-lane rural routes, and so on, and provides the safety engineer with a strong tool for choosing the sites most worthy of treatment.

The transportation agency's overall safety program is based on the premise that good accident data are available and that soundly conceived procedures are used to review these data and to select sites for treatment. The rate/quality control procedure is the most widely accepted technique for selecting statistically valid samples of sites. It should be adopted in those cases for which the transportation agency has the data and expertise to utilize the procedure.

Wet Pavement Accidents

Many pavements become smoother under the wear and tear of traffic. As the aggregates "polish" and become smoother, they decrease the ability of motorists to stop quickly under emergency situations. This is especially true if the pavement is wet. Most transportation agencies now measure this skid resistance through a standard friction test on wet pavement and classify

STATE OF ALABAMA HIGHWAY DEPARTMENT
DESIGN BUREAU - TRAFFIC ENGINEERING
ACCIDENT IDENTIFICATION SECTION
INTEGRATED MODEL TRAFFIC RECORDS SYSTEM

TIME PERIOD: 01/01/86 TO 12/31/86

INVENTORY OF HIGH ACCIDENT SEGMENTS
URBAN AREAS - ALL ROADS
DEFINING CRITERIA: MIN OF 5 ACCIDENTS

SEQ. NO.	COUNTY	CITY	LINK	BEGIN NODE	END NODE	ACCIDENT SEVERITY TOTAL	P.D.O.	INJURY	FATAL	PERSONS NON-FATAL	FATAL
1	LEE	AUBURN	5015	314	315	16	15	1		1	
2	LEE	AUBURN	5047	315	316	12	11	1		1	
3	LEE	AUBURN	5093	313	934	12	12				
4	LEE	AUBURN	5015	315	591	12	10	2		2	
5	LEE	AUBURN	5015	73	75	11	7	4		7	
6	LEE	AUBURN	5015	341	351	10	7	3		3	
7	LEE	AUBURN	5047	315	933	10	10				
8	LEE	AUBURN	5015	92	375	10	7	2	1	3	1
9	LEE	AUBURN	5015	86	93	9	5	4		5	
10	LEE	AUBURN	5015	64	73	9	8	1		1	
11	LEE	AUBURN	5136	316	590	7	7				
12	LEE	AUBURN	5015	322	331	6	6				
13	LEE	AUBURN	5015	824	849	6	6				
14	LEE	AUBURN	5089	614	615	6	5	1		1	
15	LEE	AUBURN	5093	313	319	6	6				
16	LEE	AUBURN	5093	300	934	6	6				
17	LEE	AUBURN	5155	74	75	6	6				
18	LEE	AUBURN	5015	702	834	5	5				
19	LEE	AUBURN	5046	578	588	5	4	1		1	

FIGURE 4 Example of a "general education" accident summary report.

STATE OF ALABAMA HIGHWAY DEPARTMENT
DESIGN BUREAU/TRAFFIC ENGINEERING SECTION
ACCIDENT INDENTIFICATION & SURVEILLANCE BRANCH
INTEGRATED MODEL TRAFFIC RECORD SYSTEM
INTERSTATE, URBAN, 4 LANES

CRITICAL ACCIDENT RATE ANALYSIS OF SEGMENTAL ACCIDENTS
FROM: 04/01/83 TO 12/31/85

RANKING	COUNTY	ROAD	BEGIN MILE-POST	END MILE-POST	SECTION LENGTH	SECTION ADT	ACTUAL ACC. RATE	CRITICAL ACC. RATE	DEVIATION	*** NUMBER OF ACCIDENTS *** TOTAL	P.D.O.	INJURY	FATAL
1	MOBILE	I010	28.00	28.04	0.04	33127	9.02	0.54	8.48	12	8	4	0
2	ETOWAH	I059	181.00	181.26	0.26	9701	3.95	0.65	3.30	10	6	4	0
3	MOBILE	I010	14.80	15.25	0.45	26066	2.46	0.56	1.90	29	21	8	0
4	MONTGOMERY	I065	169.50	174.72	5.22	24806	2.42	0.56	1.86	315	229	85	1
5	MONTGOMERY	I065	168.80	169.40	0.60	24806	2.34	0.56	1.78	35	29	6	0
6	JEFFERSON	I020	132.80	133.10	0.30	37586	2.12	0.53	1.59	24	20	4	0
7	MOBILE	I010	19.00	19.25	0.25	37725	2.11	0.53	1.58	20	11	8	1
8	MONTGOMERY	I065	168.00	168.25	0.25	24806	2.09	0.56	1.53	13	11	2	0
9	MOBILE	I065	13.00	13.40	0.40	18035	2.07	0.59	1.48	15	9	5	1
10	LEE	I085	59.80	60.33	0.53	12698	2.07	0.62	1.45	14	6	8	0
11	MONTGOMERY	I065	175.00	175.40	0.40	31038	1.76	0.54	1.22	22	16	6	0
12	MOBILE	I065	9.00	9.30	0.30	30797	1.72	0.54	1.18	16	14	2	0
13	MOBILE	I065	9.80	11.50	1.70	27874	1.56	0.55	1.01	74	49	25	0
14	JEFFERSON	I020	131.00	132.25	1.25	37613	1.53	0.53	1.00	72	55	17	0
15	MONTGOMERY	I065	176.00	176.21	0.21	31038	1.38	0.54	0.84	9	5	4	0
16	TUSCALOOSA	I059	76.00	76.60	0.60	16561	1.40	0.59	0.81	14	10	4	0
17	MOBILE	I010	20.00	20.25	0.25	37725	1.16	0.53	0.63	11	6	5	0
18	MOBILE	I010	17.75	18.00	0.25	34381	1.16	0.54	0.62	10	5	5	0
19	MONTGOMERY	I085	5.90	7.20	1.30	34797	1.14	0.53	0.61	52	38	14	0
20	MOBILE	I010	13.80	14.20	0.40	26066	1.15	0.56	0.59	12	10	2	0
21	MOBILE	I010	10.50	11.25	0.75	21324	1.12	0.57	0.55	18	8	8	2

FIGURE 5 Sample rate/quality control listing of potential sites for safety treatment.

their pavements on a scale of 0 to 100. Friction numbers in the 40s or above are normally believed to give good stopping resistance.

A second way to classify the skid resistance of pavements is to analyze the percentage of accidents that happen in wet weather. This type of analysis is shown in Figure 6. A site with particularly smooth pavement will often have a large number of wet weather accidents. The most useful analysis is to compute the percentage of accidents that happen in wet weather. Normally, a site might be investigated if the wet weather percentage is twice the systemwide average.

For example, in the southeastern states, approximately 20 to 25 percent of accidents occur in wet weather, even though the pavement is wet only 5 to 7 percent of the time. If 50 percent of the accidents at a given site occurred in wet weather, that site could normally be assumed to have pavement that was "slicker" than other pavements and might need additional attention.

Figure 6 presents the total number of accidents at any site, the number that occurred on wet pavement, and the percentage that occurred on wet pavement. A skid-reduction program should concentrate on "wet" accident sites, so one way to prioritize a treatment program would be to start with the sites with the greatest number of wet accidents, as long as these sites had a rate greater than twice the system average.

DESIGN BUREAU/TRAFFIC ENGINEERING SECTION
ACCIDENT IDENTIFICATION & SURVEILLANCE BRANCH
INTEGRATED MODEL TRAFFIC RECORDS SYSTEM

SKID TESTS/ACCIDENT DATA FOR RURAL, 2 LANES
STATE ROUTES IN THE 2ND DIVISION

COUNTY	RT	BEGIN MILE-POST	END MILE-POST	SECT LENGTH	ADT	SEG ACC PER MVM	NIGHT SEG ACC/MVM	WET SEG ACC/MVM	SKID NO.	TYPE MIX	ACCS/ MILE	% WET ACCS	NUMBER OF SEG. ACCS.* TOT.	P.D.O.	INJ.	FATAL
LAWRENCE	33	31.00	32.00	1.00	2621	5.93	1.40	1.40	0	416A	17.00	24	17	8	9	0
LAWRENCE	33	30.20	31.00	0.80	2863	3.99	1.20	0.80	0	416A	12.50	20	10	6	4	0
LAWRENCE	33	22.70	23.00	0.30	2492	3.67	0.00	1.22	0	416A	10.00	33	3	2	1	0
LAWRENCE	157	44.00	45.00	1.00	3800	2.89	0.96	0.96	0	416A	12.00	33	12	10	2	0
LAUDERDALE	101	28.00	29.00	1.00	3436	4.52	1.06	1.06	0	411A	17.00	24	17	11	6	0
LAWRENCE	101	24.00	24.50	0.50	3649	6.01	1.50	1.50	0	411A	24.00	25	12	8	3	1
WINSTON	5	213.00	214.00	1.00	4645	1.97	0.59	0.98	0	411A	10.00	50	10	8	2	0
MARION	17	288.00	288.42	0.42	2049	9.56	1.06	4.25	0	411A	21.43	44	9	4	4	1
MARION	5	218.00	219.00	1.00	4674	4.30	0.98	1.56	0	411A	22.00	36	22	17	5	0
MARION	5	217.15	218.00	0.85	7834	1.51	0.27	0.55	0	411A	12.94	36	11	11	0	0
WINSTON	74	30.00	30.68	0.68	1998	4.71	0.67	1.35	37	301E	10.29	29	7	5	2	0
LAUDERDALE	17	342.00	343.00	1.00	3751	3.41	1.22	0.49	46	411A	14.00	14	14	9	5	0
LAWRENCE	24	57.00	57.80	0.80	4228	3.78	0.27	0.54	47	416A	17.50	14	14	6	7	1
LAWRENCE	24	63.91	65.00	1.09	6676	3.01	1.13	0.50	49	416A	22.02	17	24	17	7	0
LAUDERDALE	17	341.00	342.00	1.00	4023	3.18	0.45	0.68	49	411A	14.00	21	14	11	3	0
LAWRENCE	24	63.00	63.91	0.91	5720	2.11	0.18	0.70	50	416A	13.19	33	12	3	9	0
LAUDERDALE	13	338.00	339.00	1.00	4766	3.26	1.34	0.96	51	424	17.00	29	17	10	7	0

FIGURE 6 Sample computer report for a wet pavement accident analysis.

A second point is illustrated by Figure 6. The accident data follow a random, statistical process. Small samples (i.e., sites with few accidents) are subject to great variability from year to year. More confidence can be placed in sites with large numbers of accidents, and engineers usually choose a minimum (threshold) number of accidents necessary to consider a site for improvement.

The most certain way to analyze loss of skid resistance is to combine wet weather accident records with friction test results. Any site that has low friction numbers and a high percentage of wet weather accidents should be a candidate for further investigation.

Daylight-Dark Accidents

Analyses of the percentage of accidents happening during nondaylight hours can be used to identify areas where motorists are having difficulty seeing the roadway or determining which driving maneuver to execute. These analyses are conducted in the same manner as wet pavement analyses. That is, any site that is having twice the percentage of accidents in nondaylight hours than the system average becomes a candidate for investigation to determine whether the installation of street lights is necessary. Lighting should only be installed in those cases for which improved visibility would reduce the types of accidents or the severity of accidents occurring at that particular site.

A sample daylight-dark analysis is given in Figure 7. The purpose of this computer report is similar to that of the wet pavement report (Figure 6), and the report is utilized identically.

Roadway Defect Investigations

Most accident investigation forms include a section on any roadway defects that might have contributed to the accident. Typically, law enforcement officers might enter comments regarding high shoulders, low shoulders, malfunctioning traffic control devices, potholes, or other defects. Transportation agencies might utilize these data for their risk management programs.

If defects do exist and contribute to traffic accidents, they become a liability for the transportation agency. The courts might find that a reasonable action for the agency would have been to find and remedy the defects before they caused additional accidents.

A roadway defect program might consist of the following steps:

1. Accident data are screened for roadway defects.
2. When defects are found, field forces are notified of the date and location of the accident and supplied with the accident report.
3. Field forces investigate the site, making observations, measurements, and photographs of conditions.
4. If a defect is found, its extent is noted and any possible contribution to the accident is documented. The defect is remedied or scheduled for routine maintenance.
5. If no defects are noted, the law enforcement officer is contacted to discover the reasons for the incorrect accident report.

This program offers three distinct advantages. First, it documents field conditions for possible future use in defending court cases. Second, field forces become acquainted with roadway situations that contribute to accidents and consequently learn to minimize these situations. Third, field forces and law enforcement officials are forced into contact with each other. Frequently, the law enforcement officials become better informed about the use of accident data and the result is an improvement in the quality of these data.

A sample road defect computer printout is presented in Figure 8. It contains enough data about the accident that field forces can conduct a simple field investigation without having a

ANALYSIS OF ACCIDENTS OCCURRING DURING DARKNESS
SYSTEM WIDE TOTAL ACCIDENTS: 5069, % DURING DARKNESS: 0.076, FOR THE PERIOD: 10/01/81 - 9/30/82

5 OR MORE ACCIDENTS					LESS THAN 5 ACCIDENTS				
STREET CODE	STREET NAME	TOTAL ACCIDENTS	TOTAL DARK	PERCENT DARK	STREET CODE	STREET NAME	TOTAL ACCIDENTS	TOTAL DARK	PERCENT DARK
1216	PORKY STREET	5	3	0.600	2460	HERTZ DR	1	1	1.000
488	EDGEVILLE RD	6	3	0.500	2378	LITTLE SQUAW SE	1	1	1.000
981	BLUE AVE NW	8	3	0.375	2299	SPORTING LANE	1	1	1.000
748	MUDD ISLAND RD	9	3	0.333	2045	LAURA DRIVE	1	1	1.000
80	NEW PARKWAY CIRCLE S	36	10	0.277	1956	WINNER DRIVE	1	1	1.000
974	MASON DR	11	3	0.272	1742	FAIRGROUND AVE	1	1	1.000
1040	DEVILLE ST	12	3	0.250	1727	WILLOW COVE	1	1	1.000
996	MEADOW DR	14	3	0.214	1607	BIGTOP BLVD	1	1	1.000
2357	HILLTOP CIRCLE	5	1	0.200	1572	JVC CIRCLE	1	1	1.000
1428	BLUE RIDGE AVE NW	15	3	0.200	1530	LITMUS AVE	1	1	1.000
1353	SEVENTH ST	5	1	0.200	1494	JERICO AVE	1	1	1.000
1215	POTTER AVE	5	1	0.200	1393	MARJORIE ST	1	1	1.000
942	BRIDGE AVE	5	1	0.200	1349	LEE ANN DRIVE	1	1	1.000
689	HALLEY AVE	5	1	0.200	1193	PHILPOT AVE	1	1	1.000
467	BATTLE VIEW AVE	5	1	0.200	1118	OAKDALE COURT	1	1	1.000
316	CLOVERDALE MALL ST	5	1	0.200	1068	THRILL LANE	1	1	1.000
222	HIGH AVE	5	1	0.200	1016	MESA BUTTE RD	1	1	1.000
181	BRANDON AVE	5	1	0.200	988	LIMMA VIEW DR	1	1	1.000
43	TUNA ST	5	1	0.200	921	BIG BEND AVE	1	1	1.000

FIGURE 7 Sample computer report for a daylight-dark accident analysis.

STATE OF ALABAMA HIGHWAY DEPARTMENT
DESIGN BUREAU/TRAFFIC ENGINEERING SECTION
ACCIDENT IDENTIFICATION & SURVEILLANCE BRANCH

FROM: 01/01/87 TO 07/02/87

INTEGRATED MODEL TRAFFIC RCORDS SYSTEM

ACCIDENTS WITH REPORTED ROAD DEFECTS ON INTERSTATE AND STATE ROUTES
DIVISION: 1 COUNTY: DEKALB CITY: ---RURAL---

SEQ. NO.	CONTRB RD DEF	ACCIDENT NUMBER	DATE	TIME	LIGHT	WEATHER	ST	NODE 1	NODE 2	DIST	NODE	MILE POST	DIR TRL	PRIM CAUSE	FIRST HARM EVENT	EST SPD	NO. VEH	INJ I	INJ K
1*	OTHER	7003403	12387	2215	DK-NL	CLDY	I059	1076	7087	3.25	2	228.00	S	DR NOT CTL	HIT CLVRT	50	1	0	0
2*	SH LOW	7020646	31287	1445	DAY	CLEAR	S035	987	7185	0.75	2	15.50	N	DUI	HIT NPK VH	60	1	2	0
3*	SH LOW	7036555	43087	100	DK-NL	CLEAR	S035	35	7846	0.50	2	27.30	W	USN OBJ/PN	FGN MAT RD	55	1	0	0
4*	SH LOW	7001949	11687	2253	DK-NL	RAIN	S227	7647	7648	1.50	2	26.50	N	VH LEFT RD	HIT CLVERT	50	1	0	0
------------------- ACCIDENTS ENTERED SINCE LAST REPORT -------------------																			
5	SH LOW	7055335	62087	220	DK-NL	CLDY	S007	7989	7992	0.50	1	244.00	S	DUI	HIT UTL PL	50	1	0	0

FIGURE 8 Sample computer report for a program to investigate accidents with reported road defects.

copy of the police accident report. However, in many cases the police report is necessary to supply enough detail about the collision so that field forces may conduct a comprehensive site investigation.

High-Exposure Accident Investigation

Some types of accidents, by their very nature, are likely to result in suits. Accidents with fatalities or serious injuries are the types that usually breed suits. Others, like those that generate large amounts of publicity, are also likely to result in suits. When the transportation agency becomes aware of these types of accidents, it is a good idea to visit the site, make key measurements, and document conditions through photographs and interviews with witnesses or local residents. This information is then placed in a file for possible later use. Although the majority of these accidents may not result in suits, the data gathered in such investigations will prove invaluable in those few instances in which suits are filed.

Most states allow plaintiffs to wait 1 to 2 years after the accident before filing the suit. In this period of time, evidence such as skid marks, debris, damaged vegetation, and so on, will disappear from the site. Unless the transportation agency has conducted an investigation soon after the accident, it will be very difficult to conduct a valid defense, due to lack of supporting field evidence.

The State of Alabama Highway Department obtains accident data 24 to 72 hours after the local law enforcement officials complete their investigation. These data are screened daily by computer. Each fatal accident is identified, and the field office closest to the accident is notified via computer printout soon after the accident data are coded. The field office then goes to the site and conducts a proper investigation. Figure 9 is a copy of the computer output used to notify field offices of the need to conduct an investigation.

Roadside Objects

Over the past few years, the most popular topics for highway liability suits have been low shoulders and improper traffic control devices. The most rapidly increasing topics of concern now appear to be single-vehicle accidents involving roadside objects and accidents that occur due to limited site distance. The former category is relatively easy to identify by a simple scan of accident data.

Because collisions with roadside objects are normally very severe and frequently result in fatalities and severe injuries, transportation agencies might benefit by targeting them for safety programs. The most common types of these accidents involve trees, poles, drainage devices, mailboxes, or bridges, barriers, or safety hardware. In general, the closer these objects are to the edge of the roadway, the less safe the driving environment becomes. Accident data can be used to identify roadway locations or situations where these accidents occur most frequently. Safety improvements can be made by pinpointing these locations and removing the roadside objects entirely or moving them farther from the roadway. Locations that have experienced the worst safety records become the leading candidates for roadway improvements.

Bridge Accidents

A special category of fixed objects consists of bridges, bridge barriers, and bridge approach barriers. Because the majority of bridges in the United States are over 50 years old, many of them are narrow, on poor alignment, or otherwise more likely to experience accidents than other roadway locations. Once a plaintiff has been awarded a large judgment, a series of these lawsuits usually follows because these bridges are highly visible and prominent on the roadways. They thus become easy targets for lawsuits.

STATE OF ALABAMA HIGHWAY DEPARTMENT
DESIGN BUREAU/TRAFFIC ENGINEERING SECTION
ACCIDENT IDENTIFICATION & SURVEILLANCE BRANCH
INTEGRATED MODEL TRAFFIC RECORDS SYSTEM

NOTIFICATION OF FATAL ACCIDENTS OCCURRING ON STATE
ROUTES & INTERSTATES IN ALL DIVISIONS

SEQUENCE NUMBER	DATE	ACCIDENT NUMBER	COUNTY	CITY	ROUTE	NODE 1	NODE 2	MILE POST	INJURIES KILLED	INJURIES INJURED
				---------- 1ST DIVISION ---------						
1	6/25/87	7056129	MADISON	---RURAL---	S001	7285		352.6	1	2
2	6/17/87	7056128	MADISON	---RURAL---	S001	8221	8406	329.2	1	2

---------- 2ND DIVISION ---------

THERE WERE NO FATAL ACCIDENTS IN THE 2ND DIVISION ENTERED INTO THE COMPUTER SINCE THE LAST REPORT

---------- 3RD DIVISION ---------

THERE WERE NO FATAL ACCIDENTS IN THE 3RD DIVISION ENTERED INTO THE COMPUTER SINCE THE LAST REPORT

---------- 4TH DIVISION ---------

THERE WERE NO FATAL ACCIDENTS IN THE 4TH DIVISION ENTERED INTO THE COMPUTER SINCE THE LAST REPORT

FIGURE 9 Sample computer report for a program to investigate "high-exposure"–type accidents.

STATE OF ALABAMA HIGHWAY DEPARTMENT
DESIGN BUREAU - TRAFFIC ENGINEERING
ACCIDENT IDENTIFICATION

TIME PERIOD: 04/01/83 TO 12/31/85
SORTED BY: ACCIDENT RATE
INCREMENT LENGTH: 0.15

INTEGRATED MODEL TRAFFIC RECORDS SYSTEM
ACCIDENT RATES FOR BRIDGES OF
THE ALABAMA INTERSTATE SYSTEM

SEQ NO	COUNTY	CITY	ROUTE	MILE POST	BRIDGE LOCATION	ADT &	YR	RDWAY BDG WIDTH	BRIDGE LENGTH	SEG ACC PER MV	**SEGMENTAL ACC** TOT=	PDO	+INJ	+FAT	BDG ACC
277	CULLMAN	--RURAL--	I065	298.54	MARIOT CREEK	17160	83	0.00	71	0.29	5	4	0	1	0
278	MOBILE	MOBILE	I010	19.50	MOORES CREEK	17780	83	28.00	204	0.28	5	3	1	1	0
279	LEE	--RURAL--	I085	68.00	HALLAWAKEE CREEK	10360	83	28.00	204	0.28	3	2	1	0	0
280	MONTGOMERY	MONTGOMERY	I065	17.24	BRANCH	7085	83	0.00	31	0.28	2	2	0	0	0
281	MOBILE	MOBILE	I065	18.82	US 43	7085	83	51.20	445	0.28	2	1	1	0	0
282	SHELBY	LEEDS	I020	144.29	LITTLE CAHABA RIVER	17500	83	0.00	38	0.28	5	1	4	0	0
283	SHELBY	--RURAL--	I065	140.68	BRANCH	11005	83	0.00	30	0.27	3	2	1	0	0
284	MOBILE	PRICHARD	I065	8.38	ILLINOIS CENTRAL GULF RR	14515	83	38.50	229	0.27	4	3	1	0	0
285	BLOUNT	--RURAL--	I065	289.44	BRANCH	17850	83	0.00	42	0.27	5	4	1	0	0
286	JEFFERSON	FAIRFIELD	I059	119.18	41ST STREET	22050	83	38.50	170	0.27	6	2	4	0	0
287	BALDWIN	--RURAL--	I010	60.82	ALLEN SPRING BRANCH	7500	83	0.00	37	0.26	2	2	0	0	1
288	SHELBY	--RURAL--	I065	229.26	SOUTHERN RR	7385	83	28.00	169	0.26	2	2	0	0	1
289	JEFFERSON	--RURAL--	I065	264.95	SOUTHERN RR	15248	83	39.20	393	0.26	4	4	0	0	0
290	JEFFERSON	BIRMINGHAM	I065	266.10	US 31	15245	83	39.20	360	0.26	4	2	1	1	0
291	JEFFERSON	BIRMINGHAM	I065	259.72	5TH AVE SOUTH	52555	83	50.50	193	0.26	14	13	1	0	0
292	ESCAMBIA	--RURAL--	I065	57.44	BRANCH	7410	83	0.00	43	0.26	2	2	0	0	0
293	MOBILE	MOBILE	I065	49.94	FLETCHER CREEK	3705	83	28.00	272	0.26	1	1	0	0	0
294	ESCAMBIA	--RURAL--	I065	51.14	BRANCH	3705	83	0.00	28	0.26	1	1	0	0	0

FIGURE 10 Sample computer report for a program to investigate bridge-related accidents.

Figure 10 is a sample computer summary of bridge accidents. It displays high-accident sites, high-accident rate sites, and high-severity sites as part of one state's risk management program. Care must be used in establishing a bridge accident summary report because these collisions are relatively rare events. A very large volume of data must be used to pinpoint individual structures that need treatment. Difficult decisions must be made regarding the criteria for identifying bridge accidents. For example, are accidents accepted for analysis if they occur on a bridge approach? If so, how close to the structure must they be to count as a "bridge hit"? Because very few structures are hit with great frequency, it is often difficult to isolate clear trends. States may have to group similar bridges together to get enough data to choose the types of structures for treatment.

Railroad Grade Crossing Accidents

A final example category of accident reports involves rail-highway grade crossings. Because of the nature of the vehicles involved, these collisions are normally of high severity for the occupants of automobiles. The potential for fatalities and serious injuries and the virtual certainty of publicity marks this as one category of accidents with a high probability of resultant court cases. Plaintiffs' attorneys have been relatively successful in convincing juries that the large conglomerate railroads have been the cause of injuries to occupants of highway vehicles and have thus collected a sizable number of substantial judgments and settlements.

Rail-highway grade crossings have been the subject of a federal safety emphasis for a number of years. Section 203 – Rail Highway Safety Program funds have been used specifically to address safety deficiencies in this area. As a result, most states have already developed computer programs to review rail-highway accident data. If that is the case, the agency's emphasis might be best directed at developing and maintaining realistic site selection criteria. A priority list using these concepts is an excellent risk management tool—if the list is actually followed. If the priority list was the basis for the expenditure of safety funds, it is easy to demonstrate to a court that the agency was doing all that could be reasonably expected.

SUMMARY

Almost all transportation agencies have experienced a rapid and significant increase in monetary losses from tort liability suits. Usually, alleged negligence on the part of the agency is the basis for the suit.

Traffic accident records play a critical part in minimizing these suits. For example, these data may be used in routine safety programs to identify and remove hazardous locations. A critical element in a good risk management program is the demonstration that the agency is effectively doing all that it can to remove these safety hazards through a well-planned and well-executed safety program.

Specialized accident reduction efforts can be very beneficial in minimizing tort liability losses. Wet pavement analyses, daylight-dark investigations, roadway defect investigations, high-exposure accident monitoring, and roadside object analysis programs were used as examples of these special studies.

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The Transportation System Management (TSM) Association, a research, service, and education group located in the College of Engineering of the University of Alabama, has been extremely active in defining and installing risk management programs for state and city agencies in the past few years. The materials in this paper are typical of those produced under the professional education activities of TSM. The author gratefully acknowledges TSM's role in the production of this document.

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Generalized Loglinear Models of Truck Accident Rates

F. F. SACCOMANNO AND C. BUYCO

Several methods for calibrating statistical models of truck accident rates are considered. A loglinear approach is suggested for assessing the effect of traffic environment on truck accident rates. A number of concerns associated with using a weighted least squares algorithm for estimating β parameters in the loglinear expression are noted, including the presence of reduced cell membership in the contingency tables of accidents and input variable incompatibilities between continuous exposure and categorical accident measures. An alternative form of generalized linear interactive model (GLIM) is proposed for calibrating loglinear expressions of truck accident rates. GLIM uses maximum likelihood techniques for estimating β parameters in loglinear expressions. As in the classical weighted least squares algorithm, this approach permits a stepwise statistical analysis of higher-order interactions in the traffic environment as related to accident frequencies, while adjusting directly for continuous measures of exposure. The results of a calibration of GLIM loglinear expressions are presented using 1983 truck accident and exposure data for Ontario as a basis.

Truck accidents are caused by complex interactions of environmental and operational factors that are present at a specific time and location on the road network. Higher-order interactions between mitigating factors in an accident situation can be identified through a calibration of statistical models of truck accident rates. These rates are usually expressed as the ratio of the number of accidents divided by the amount of travel for a comparable mix of mitigating factors. The amount of travel or exposure measure reflects the number of opportunities available for each accident to take place (*1*).

Recent attempts to calibrate reliable statistical models of truck accident rates have been hampered by two basic concerns: (a) incompatibility between continuous exposure information and categorical accident data and (b) the absence, in most jurisdictions, of comprehensive information on truck exposure. Incompatibilities in variable inputs restrict the methodology for analyzing truck accident rates to procedures that can incorporate both categorical and continuous information directly into the analysis. The absence of suitable exposure information has restricted the classification of accident environment to basic conditions for which travel information is available. Frequently, this has resulted in an analysis of factors that ignores second- and third-order interactions that affect truck accident rates. A number of recent studies on truck accidents illustrate the problems associated with lack of exposure data and incompatibilities in factor inputs for truck accident rate models (*2–4*).

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In this paper, some previous methods of analysis for truck accident rates are explored. The discussion is methodological in nature, with emphasis on the major limitations associated with each approach. Some important statistical concerns for the use of classical loglinear models in truck accident rate analysis are addressed. Alternatively, a generalized linear interactive model (GLIM) form of loglinear expression that incorporates both categorical accident involvement data and continuous exposure measures is suggested. As does the classical loglinear model, this approach allows for a stepwise statistical analysis of higher-order interaction effects for truck accidents, while adjusting directly for the continuous exposure factor.

ALTERNATIVE METHODS OF ANALYSIS

In recent years, the issue of truck safety has been receiving considerable attention. However, both the methods of analysis and many of the conclusions from these studies have lacked consistency. A special report on twin trailer trucks (doubles) by TRB (*5*) reviewed the available literature on truck safety in the United States and reported that ". . . several studies are extremely variable and conflicting." A recent presentation of results to an Organization for Economic Cooperation and Development meeting of member countries (*6*) and an earlier review by Freitas (*7*) arrived at similar conclusions. Much of the blame for a lack of consistency in studies of truck accidents has been attributed to limitations in the available data in various jurisdictions.

The focus of the discussion in this section of the paper is on methodology rather than results in recognition that the results may not be transferable to all jurisdictions. A lack of consistency in the choice of analysis procedure may have contributed as much to conflicting evidence on truck accident causation as problems with the data bases did.

In this section, three different procedures for calibrating truck accident rate expressions are discussed. The purpose of these expressions is to give a mathematical representation of the relationship between factors that influence accidents and the resultant truck accident rates. Mitigating factor inputs for these expressions are obtained from a statistical screening of candidate variables by multivariate techniques, such as analysis of variance and factor analysis.

Multiple Linear Regression

Wright and Burnham (*8*) used multiple linear regression and factor analysis to study the effect of selected roadway features on truck accident and severity rates. Initially, 15 roadway features were considered in the analysis. These features were subsequently grouped by factor analysis into four distinctive and uncolinear attributes: percent total mileage with two lanes, percent two-lane mileage with substandard horizontal curvature, percent two-lane mileage with substandard vertical curvature, and percent mileage with substandard pavement width. In Table 1 and below, the results of one of the best accident rate regression expressions using these four roadway attributes as independent variables are summarized:

$$Y_2 = 9.85X_{10} + 6.83X_{13} + 36.08X_{14} + 11.73X_{15} - 555$$

where

Y_2 = accident rate for truck, accidents/100 × 10^6 vehicle-mi;
X_{10} = percent of total mileage with two lanes;
X_{13} = percent of two-lane mileage with substandard horizontal curvature;
X_{14} = percent of two-lane mileage with substandard vertical curvature; and
X_{15} = percent of two-lane mileage with substandard pavement width.

F ratio = 3.03
Standard error $S(Y_2)$ = 565
Correlation coefficient (R) = 0.60
R^2 = 0.36

TABLE 1 ACCIDENT RATE MODEL FOR TRACTOR-SEMITRAILER TRUCKS: STATISTICAL DATA FOR REGRESSION COEFFICIENTS

	Variable			
	X_{10}	X_{13}	X_{14}	X_{15}
Level of significance	0.260	0.547	0.004	0.333
Standard error	8.51	10.43	11.27	11.86

Wright and Burnham acknowledged that on the basis of an R^2 value of 0.36, much of variation in the predicted truck accident rates for this expression remains unexplained. Furthermore, many of the calibrated coefficients were found to lack statistical significance. Only the variable X_{14} (percent of two-lane mileage with substandard vertical curvature) is statistically significant at the 0.05 level. Furthermore, the intercept term (−555) in this expression is too large and negative. Intuitively, this is unacceptable because accident rates must be positive.

The use of multiple linear regression for analyzing the causes of truck accidents may be inappropriate because the relationship itself does not always reflect linear behavior. Furthermore, multiple linear regression cannot account for the non-negative nature of accident occurrence.

Poisson Regression Models

Jovanis and Chang (*9*) suggested applying Poisson regression to the analysis of truck accident data to overcome the non-negativity shortcomings of linear regression. The basic assumption of Poisson regression is that accident occurrence follows a Poisson distribution, with assumed independence between accidents. The expected value of accident involvement for a given time interval i is expressed as

$$\lambda_i = f(\beta, X_i) \tag{1}$$

where β is the vector of parameters to be estimated and X_i is the vector of independent causal variables. The probability of k accidents in the time interval t is given by the Poisson expression

$$P_k = (\exp \lambda_i)(\lambda_i)^k/k! \tag{2}$$

A likelihood expression for β can be obtained by maximizing Equation 2 with respect to β, such that

$$L(\beta) = \prod_{i=1}^{n\to\infty} \; \prod_{k=0} P_k^{D_{ik}} \tag{3}$$

where D_{ik} is a dummy variable (1 if an accident occurred, 0 otherwise). The logarithmic form of Equation 3 is referred to as the log likelihood value of the Poisson regression. This expression serves as a basis for estimating the regression coefficients and testing the degree of fit of each calibrated expression.

In this study, accident and exposure data were collected for trucks on the Indiana Toll Road in 1978. Exposure was estimated from the number of tolls collected at each exit booth. During the study period, 700 truck accidents were observed. The independent variables included daily VMT (vehicle miles traveled) for trucks and hours of snow and rain during the study. The dependent variable was expressed as the expected number of truck accidents per day.

Because this study was based on a disaggregate analysis of individual shipments, the approach is limited to analysis of closed systems in which extensive monitoring of the traffic is possible. Exposure can thus be estimated for a specified period of time and a given set of mitigating accident factors, such as weather or traffic distribution. However, on most sections of the road network (e.g., roads that are not Interstates) this level of disaggregate information is not available.

Loglinear Models and the Weighted Least Squares Algorithm

Loglinear models are most appropriate for analyzing the effects of selected categorical variables on accident involvement. The strength of association between various categories is expressed by the calibrated β parameters, such that

$$\ln Y_i = \beta\,(X_i) \qquad i = 1, 2, \ldots, n$$

where Y_i is the expected cell frequency or accident counts for factor combinations i and X_i is the covariate vector i. The β parameters in Equation 4 measure the strength of association between the factors X_i and indicate the magnitude of contribution to accident involvement associated with each factor combination. The theoretical background of loglinear models is discussed in depth by Bishop et al. (*10*).

Several considerations support the use of loglinear models for truck accident analysis. First, variables that affect truck accidents can be expressed categorically, for example, road type and vehicle type. Because truck accidents are discrete in nature, categories can be developed to allow maximum representation of differences in accident response from the data base. Second, a loglinear approach allows the statistical significance of partial and marginal association to be tested for a given combination of categorical factors. Third, the non-negativity characteristic of accident occurrence is handled through a maximization of a Poisson log likelihood expression, similar to Equation 3.

Philipson et al. (*11*) used a loglinear approach to derive inferences concerning truck accident causation. Loglinear models were developed on the basis of truck accident involvement, without adjusting the results for exposure. Interactions among causal factors in these models were assumed to be unique to the accident data alone. The analysis is similar to comparing the absolute frequencies of accidents for various categories of contributing factors.

Chira-Chavala and Cleveland (*4*) adopted a loglinear approach using large truck accident data from the United States for 1977 to study the effect of various causal factors on truck accident rates. Incompatibilities between continuous exposure measures and categorical accident causes were addressed by fitting separate loglinear expressions to the accident involvement and the exposure data.

In their analysis, Chira-Chavala and Cleveland selected "best fit" loglinear expressions on the basis of the relationship of selected causal factors to accident involvement (frequency) alone. The same model configuration was then applied to the exposure data. These loglinear expressions for exposure reflected ***by design*** the same factors that were considered to affect accident involvement. Exposure, in this study, was expressed in truck vehicle-miles for each combination of mitigating factors.

Chira-Chavala and Cleveland combined involvement and exposure loglinear expression to yield an accident rate loglinear expression of the form

$$\log (m_{ij}/e_{ij}) = W + W_i + W_j + W_{ij} \tag{5}$$

where m_{ij} is the expected number of accidents, e_{ij} is the expected volume of truck travel, and

$$W = U - V \qquad W_i = U_i - V_i$$
$$W_j = U_j - V_j \qquad W_{ij} = U_{ij} - V_{ij}$$

where the *U*s and *V*s are the parameter estimates of accident and exposure models, respectively. Equation 5 was applied to variables with known, compatible accident and exposure measures.

Chira-Chavala and Cleveland noted that "exposure does not affect the goodness of fit or the selection of the 'best' accident rate model," hence the loglinear structure of the exposure expression can be based on the results of the loglinear analysis for accident involvement. Buyco and Saccomanno (*12*) have shown that the use of separate but structurally similar loglinear expressions for accident and exposure data does not provide stable estimates of accident rates. Exposure plays a significant and distinctive role in fitting accident rate models. Furthermore, by using the Chira-Chavala and Cleveland study as a basis, Buyco and Saccomanno demonstrated that the classical weighted least squares algorithm (WLSA) for calibrating loglinear models produces high residuals for cells that are characterized by low cell memberships in the contingency table of causal factors (i.e., sampling zero cells). The WLSA approach generally requires large samples. As noted by Koch and Imrey (*13*), parameter estimates based on the WLSA approach are sensitive to small observed and expected cell counts. Given the nature of accident data, sampling zeros are problematic in most cases.

In the next section, an alternative approach for calibrating loglinear models of truck accident rates is presented. This approach uses maximum likelihood techniques for calibrating β parameters and incorporates exposure directly into the loglinear expression as an offset.

GENERALIZED LOGLINEAR MODELS OF TRUCK ACCIDENT RATES FOR ONTARIO

An approach using was developed by Baker and Nedler (*14*) of the British Royal Statistical Society to fit loglinear expressions to various factor relationships. In the GLIM approach, the dependent variables in a contingency table are considered to behave in a Poisson-like process with values ranging from 0 to infinity. The algorithm for calibrating loglinear models of accident rates permits the inclusion of exposure as a continuous covariate in the expression.

In this section of the paper, the GLIM approach for calibrating loglinear models is presented. Loglinear expressions of truck accident rates are obtained, using Ontario truck accident data for 1983 as a basis.

Theoretical Background

The β parameters of loglinear models that use the GLIM approach are calibrated on the basis of maximum likelihood techniques. This differs from the traditional weighted least squares algorithm used in most loglinear statistical packages [e.g., BMDP (*15*)]. Maximum likelihood permits the inclusion of continuous covariates in the loglinear expression.

A quantitative covariate whose β parameter is known beforehand is referred to as an offset (*16*). In accident analysis, it is assumed that accident frequency is directly related to exposure, such that the β parameter for exposure is assumed to be 1.0. This assumption must be checked in testing the significance of alternative expressions. The offset, which is declared in calibration, is subtracted initially from the linear predictor, and the result is regressed on the remaining categorical variates. The dependent variable in these expressions is accident rate, which is expressed as the number of incidents per unit of exposure.

Model calibration of accident rates using the GLIM procedure involves fitting two separate loglinear expressions.

Model A

This model uses exposure as an offset for accident frequency, such that

$$\text{LOGNOACC} - k * \text{LOGEXP} = 1 + R + A + \ldots + RAM \tag{6}$$

where R, A, . . ., RAM are various combinations of factor inputs in the loglinear expression.

The logarithm of truck travel exposure (LOGEXP) is treated as an offset and subtracted initially from the logarithm of the number of accidents (LOGNOACC) before fitting the expression. The resultant dependent variable is regressed on the remaining covariates, representing factor interactions in the accident causation model. The use of exposure as an offset in the accident rate loglinear expression (Equation 6) assumes that the k term is equal to 1.0.

Model B

In this model, exposure is treated as a covariate for an accident frequency expression, such that

$$\text{LOGNOACC} = 1 + R + A + \ldots + RAM + b * \text{LOGEXP} \quad (7)$$

Modification of Equation 7 in terms of accident rate on the left-hand side yields an expression of the form

$$\text{LOGNOACC} - \text{LOGEXP} = 1 + R + A + \ldots + RAM + (b - 1) * \text{LOGEXP} \quad (8)$$

Equation 7 is used to check the validity of the assumption that k in Equation 6 is equal to 1.0 (or alternatively, that b is not significant). The accident frequency expression, Model B, serves to test the acceptability of the accident rate loglinear expression, Model A.

The "best fit" model is selected on the basis of the ratio of the likelihoods of the fitted model to the full or saturated model. It is possible to test the significance of specific interaction effects by adding or deleting individual terms from the fitted expression with a stepwise procedure. A complete derivation of the GLIM approach for loglinear calibration and testing is included in works by Koch and Imrey (*13*), McCullagh and Nedler (*16*), and Bishop et al. (*10*).

Application of GLIM to Ontario Truck Accident Data

The Ontario truck accident data base was modified to exclude observations made in the northern sector of the province and at intersections and ramps, as well as those for which load status is unknown. The modified data base consists of 1,955 large truck accidents for 1983. Multivariate techniques, such as n-level analysis of variance, were applied to the truck accident data to produce a contingency table of categorical factors affecting truck accident involvement (Table 2). The approach serves as an initial screening of candidate factors for calibrating loglinear expressions of truck accident causation. The contingency table of categorical factors in Table 2 consists of 960 cells, of which 132 are considered to be structurally empty (no observed exposure).

Truck travel over the entire road network was estimated from provincial link-specific truck counts, adjusted by weighing station estimates from the 1983 Commercial Vehicle Survey (CVS) (*17*) for Ontario. Weighing stations at specific locations of the road network were classified according to road and land development characteristics on the basis of Ontario road inventory data. The proportion of trucks of a given type at each weighing station group was estimated directly from the CVS counts. These proportions were applied to total truck flows on similar classes of roads to yield VMTs for trucks of different configurations on these roads. Total truck flows on the Ontario highway network were obtained directly from provincial volume data. The application of CVS truck proportionalities to total flows assumes that the truck characteristics that are observed at each weighing station group are representative of the distribution of trucks on all other similar roads. This approach is discussed in detail by Buyco and Saccomanno (*18*).

TABLE 2 VARIABLES AND CATEGORIES IN THE CONTINGENCY TABLE FOR TRUCK ACCIDENT RATE MODEL

Variable	Symbol	Category	Description
Road type	R	1	Freeway
		2	Nonfreeway
Traffic pattern	P	1	Commuter
		2	Noncommuter
Traffic volume	A	1	Low
		2	High
Truck type	T	1	Truck
		2	Truck and trailer
		3	Tractor
		4	Tractor and trailer
		5	Tractor and two trailers
Load status	L	1	Empty
		2	Loaded
Model year	M	1	Post-1977
		2	Pre-1977
Hour of day	N	1	1800–600
		2	600–1800
Driver age	D	1	<25 years old
		2	25–54 years old
		3	>54 years old

Table 3 summarizes the hierarchical steps used in fitting a loglinear expression to the contingency table of factors affecting truck accident involvement. By using Model A (Equation 6) as a basis, terms can be added and deleted in a stepwise analysis of individual factor interactions. The results can also be compared with the corresponding loglinear expression for accident frequency with the exposure term as a covariate (Model B, Equation 7). As indicated in Table 3, the "best fit" expression is obtained for step $3a$, where the third-order term RAM is added. This expression indicates that the addition of the term RAM is statistically significant at the 5 percent level. The model itself is not statistically different from the saturated expression. The format using exposure as a covariate (Model B) indicates that term b for step $3a$ is not significant. From this analysis, the "best fit" truck accident rate expression using exposure as an offset is

$$\begin{aligned}\log AR = {} & 1 + R + P + A + T + L + M + N + D \\ & + RA + RP + PA + RT + PT + PL + TL \\ & + RM + AM + TM + RN + PN + AN + TN \\ & + LN + MN + TD + RAM \end{aligned} \quad (9)$$

where AR is the expected accident rate in truck involvements per thousand truck-km and $R, P, \ldots.$ RAM are the β parameters used in estimating $\log AR$.

TABLE 3 STEPWISE MODEL SELECTION

Model	No. of Terms 2nd	No. of Terms 3rd	No. of Terms Total	Model A Scale Dev. (S.C.)	Model A Diff. (S.C.)	Model A DOF	Model A Diff. DOF	Model A Crit. χ^2 (5% LOS)	Model A Conclusion	Model B Scale Dev.	Model B DOF	Model B LOGEXP	Model B *T* Test
1. *R+P+A+T+L+M+N+D*	0	0	9	1,144.4		815		882.5	S.				
2. *(R+P+A+T+L+M+N+D)* · *(R+P+A+T+L+M+N+D)*	28	0	37	663.2		757		821.8	N.				
(add 2nd level int.)					481.2		58	76.8	S.				
3. Model 2–*D*–int–M.*(A+P+L)* –*A.(R+T+L)*–*R.L+T.D+D*	15	0	24	686.5		779		844.8	N.	654.12	778	0.758	S.
[del *D*–int.,*M.(P,A,L)*, *A.(R,T,L),R.L*]					–23.3		–22	33.9	N.				
4. Model 3–*T.D*	14	0	23	713.9		787		853.1	N.				
(del *T.D*)					–27.4		– 8	15.5	S.				
1*a*. Model 3+*R.A+A.M*	17	0	26	681.3		777		842.7	N.	551.57	776	–0.003	N.
(add *R.A, A.M*)					5.2		2	6.0	N.				
2*a*. Model 1*a*.–*R.A*	16	0	25	681.5		778		843.7	N.	651.57	777	0.765	S.
(del *R.A*)					– 0.2		– 1	3.8	N.				
3*a*. Model 1*a*.+*R.A.M* (add *R.A.M*)[a]	17	1	27	669.1		776		841.6	N.	541.21	775	–0.022	N.
					12.2		1	3.8	S.				
4*a*. Model 1*a*.+*P*T*L*	18	1	28	642.1		775		840.6	N.	620.63	774	0.796	S.
(add *P*T*L*)					39.2		2	6.0	S.				
5*a*. Model 1*a*.+*R.P.A*	17	1	27	675.0		776		841.6	N.	537.75	775	–0.060	N.
(add *R.P.A*)					6.3		1	3.8	S.				
6*a*. Model 1*a*+*P*T*L+R.P.A* +*R.A.M*	17	3	29	622.6		772		837.5	N.	517.91	771	–0.003	N.
(add *P*T*L,R.A.M,R.P.A*)					58.7		5	9.5	S.				

NOTES: Total number of terms includes main effects. Refer to Table 2 for variable symbols. DOF = degrees of freedom.
[a]Model 3*a* is selected as the "best" model.

Standardized residuals for Equation 9 were inspected graphically for different values of exposure. Large residual values (greater than 5.0) were obtained for eight cells, as summarized in Table 4. These cells reflect very high observed accident rates, with low exposure values and an observable number of accidents.

A dispersion factor of the form

$$\sigma^2 = \chi^2/(N - p) \qquad (10)$$

[where χ^2 is the standardized Pearson chi square value and $(N - p)$ gives the degrees of freedom] was used to reflect variability in the data. The deletion of the eight cells with high standardized residuals resulted in a reduced dispersion value for Model A from 2.44 to 1.01. This indicates that the eight cells account for much of the dispersion in the loglinear accident rate expression (Equation 9). The β parameters for this expression, however, remained stable (Table 5). Furthermore, doubling exposure for the eight high-residual cells did not alter the β parameters for this model to any appreciable extent.

Information on the first category associated with main and higher-order interaction effects in the model is excluded from the results in Table 5. These categories are intrinsically aliased; that is, their estimates are set to zero. Estimates of all other nonaliased categories for a given level of interaction are relative to these aliased categories (*14*).

The β parameters in the loglinear expression, which are summarized in Table 5, reflect the degree of association for different levels of interaction among the categorical factors that

TABLE 4 CELLS WITH LARGE STANDARDIZED RESIDUALS

Cells *R*	*P*	*A*	*T*	*L*	*N*	*M*	*D*	Observed No. of Accidents	Observed Exposure (10^3 truck-km)	Observed Accident Rate (10^6 truck-km^{-1})	Expected No. of Accidents	Standardized Residuals
1	1	1	5	2	1	2	2	2	162.0	12.35	0.050	8.707
2	1	1	2	2	1	1	2	1	101.7	9.83	0.003	17.091
2	1	1	4	1	1	1	2	10	2,219.7	4.51	2.205	5.249
2	1	2	1	1	1	1	2	3	50.2	59.76	0.031	16.773
2	1	2	4	1	1	2	2	3	139.0	21.58	0.299	4.942
2	2	1	1	2	1	2	2	6	1,707.5	3.51	0.968	5.114
2	2	1	2	2	1	1	1	1	145.6	6.87	0.028	5.805
2	2	1	3	1	2	1	2	1	6.1	163.93	0.002	20.869

NOTE: Refer to Table 2 for parameter symbols and categories.

TABLE 5 COMPARISONS OF PARAMETER ESTIMATES

Parameter Symbol	Level	Model 1 (no data adjusted)	Lambda Values, Model 2 (8 cells excluded)	Difference Between Models 1 and 2	Model 3 (8 cells exposure doubled)	Difference Between Models 1 and 3
Mean		−7.0390	−7.1720	0.1330	−7.0500	0.0110
R	2	−0.2130	−0.4501	0.2371	−0.2516	0.0386
P	2	0.6735	0.7910	−0.1175	0.6839	−0.0104
A	2	0.3838	0.4504	−0.0666	0.3941	−0.0103
T	2	−2.4860	−3.0300	0.5440	−2.5010	0.0150
T	3	−0.9406	−0.6271	−0.3135	−0.9378	−0.0028
T	4	−0.1221	−0.1281	0.0060	−0.1338	0.0117
T	5	−1.4350	−1.3970	−0.0380	−1.4390	0.0040
L	2	−1.2110	−1.1470	−0.0640	−1.2020	−0.0090
M	2	0.1495	0.0667	0.0828	0.1401	0.0094
N	2	0.6900	0.8437	−0.1537	0.7127	−0.0227
D	2	−0.8897	−0.9212	0.0315	−0.8933	0.0036
D	3	−0.9849	−0.9849	0.0000	−0.9849	0.0000
RP	22	−0.4081	−0.3459	−0.0622	−0.3891	−0.0190
RA	22	0.3783	0.3776	0.0007	0.3910	−0.0127
PA	22	−0.5136	−0.5622	0.0486	−0.5186	0.0050
RT	22	1.0890	0.7175	0.3715	1.0860	0.0030
RT	23	0.7122	0.4034	0.3088	0.7268	−0.0146
RT	24	0.6179	0.6613	−0.0434	0.6190	−0.0011
RT	25	1.1830	1.3440	−0.1610	1.1990	−0.0160
PT	22	0.1360	0.1799	−0.0439	0.1417	−0.0057
PT	23	0.3350	−0.0100	0.3450	0.3357	−0.0007
PT	24	−0.2899	−0.2453	−0.0446	−0.2769	−0.0130
PT	25	−0.6251	−0.5150	−0.1101	−0.6119	−0.0132
PL	22	0.4044	0.3261	0.0783	0.3915	0.0129
TL	22	0.4993	0.3006	0.1987	0.5028	−0.0035
TL	32	0.0000	0.0000	0.0000	0.0000	0.0000
TL	42	0.2734	0.2978	−0.0244	0.2806	−0.0072
TL	52	2.6410	2.5690	0.0720	2.6380	0.0030
RM	22	1.0580	1.1140	−0.0560	1.0600	−0.0020
AM	22	0.6930	0.7374	−0.0444	0.6935	−0.0005
TM	22	−0.1569	0.0270	−0.1839	−0.1488	−0.0081
TM	32	0.1504	0.2964	−0.1460	0.1575	−0.0071
TM	42	−0.9007	−0.8752	−0.0255	−0.8933	−0.0074
TM	52	−1.2790	−1.4730	0.1940	−1.2770	−0.0020
RN	22	−0.4838	−0.2939	−0.1899	−0.4594	−0.0244
PN	22	−0.7740	−0.8637	0.0897	−0.7880	0.0140
AN	22	0.3604	0.3308	0.0296	0.3502	0.0102
TN	22	0.2150	0.7420	−0.5270	0.2152	−0.0002
TN	32	0.0041	−0.2711	0.2752	−0.0105	0.0146
TN	42	−0.4285	−0.4915	0.0630	−0.4299	0.0014
TN	52	−1.5730	−1.6030	0.0300	−1.5770	0.0040
LN	22	0.4029	0.3557	0.0472	0.3933	0.0096
MN	22	0.3176	0.3410	−0.0234	0.3219	−0.0043
TD	22	−0.0553	0.2006	−0.2559	−0.0450	−0.0103
TD	23	−0.3286	−0.0433	−0.2853	−0.3218	−0.0068
TD	32	0.4436	0.4011	0.0425	0.4471	−0.0035
TD	33	0.3637	0.3637	0.0000	0.3637	0.0000
TD	42	0.7311	0.7536	−0.0225	0.7329	−0.0018
TD	43	0.1083	0.1083	0.0000	0.1083	0.0000
TD	52	0.9828	0.9810	0.0018	0.9856	−0.0028
TD	53	−0.1468	−0.1468	0.0000	−0.1468	0.0000
RAM	222	−0.9901	−1.0800	0.0899	−0.9991	0.0090

NOTE: Refer to Table 2 for parameter symbols. See Equation 9 for accident rate model.

TABLE 6 TRUCK ACCIDENT RATES FOR AN ONTARIO CORRIDOR

Highway Number	Link Number	Road Characteristics[a] R	P	A	Dist. (km)	AADT	Observed Truck Accident Rate (no./10^6 truck-km)[b]	Estimated Accident Rates for Truck Configurations (no./10^6 truck-km) Truck	Truck + Trailer	Tractor[c]	Tractor + 1 Trailer	Tractor + 2 Trailers
6	1	2	1	2	0.8	26,300	–	0.490	0.234	1.369	1.431	2.961
	2	2	1	2	2.2	25,000	1.7	0.490	0.234	1.369	1.431	2.961
5	3	2	1	1	2.8	9,900	1.4	0.159	0.076	0.446	0.466	0.964
	4	2	1	1	0.9	10,000	–	0.159	0.076	0.446	0.466	0.964
	5	2	1	1	2.0	9,800	–	0.159	0.076	0.446	0.466	0.964
	6	2	1	1	0.5	9,800	–	0.159	0.076	0.446	0.466	0.964
	7	2	1	1	3.5	9,800	2.4	0.159	0.076	0.446	0.466	0.964
	8	2	1	1	6.1	9,800	0.3	0.159	0.076	0.446	0.466	0.964
	9	2	1	1	2.1	9,500	6.8	0.159	0.076	0.446	0.466	0.964
	10	2	1	1	7.4	10,000	1.0	0.159	0.076	0.446	0.466	0.964
	11	2	1	1	2.9	9,500	2.6	0.159	0.076	0.446	0.466	0.964
	12	2	1	1	0.6	9,500	–	0.159	0.076	0.446	0.466	0.964
403	13	1	1	2	4.9	31,200	0.7	0.674	0.108	0.923	1.061	1.247
	14	1	1	2	1.6	29,500	1.1	0.674	0.108	0.923	1.061	1.247
	15	1	1	2	4.6	43,700	0.2	0.674	0.108	0.923	1.061	1.247
	16	1	1	2	2.1	47,000	0.7	0.674	0.108	0.923	1.061	1.247
	17	1	1	2	2.8	35,300	0.3	0.674	0.108	0.923	1.061	1.247
	18	1	1	2	2.9	38,800	0.4	0.674	0.108	0.923	1.061	1.247
401	19	1	1	2	2.2	124,200	1.4	0.674	0.108	0.923	1.061	1.247
	20	1	1	2	3.8	158,600	1.2	0.674	0.108	0.923	1.061	1.247
	21	1	1	2	1.8	81,700	1.8	0.674	0.108	0.923	1.061	1.247
Total					58.5							

[a]Refer to Table 2 for parameter symbols and categories. Accident rates are estimated using Equation 9 and Table 5 with load status = loaded (2), model year = post-77 (1), hour of day = 600–1800 (2), and driver age = 25–54 (2).
[b]Observed truck accident rates taken from *1983 Traffic Volumes–Provincial Highways*, Ministry of Transportation and Communications, Ontario.
[c]Tractor is considered as empty in this model. See Figure 1 for the highway corridor.

influence truck accident rates. Truck accident rates for various combinations of influencing factors can be obtained by using these β parameters directly in the loglinear expression that includes these factors (Equation 9).

The significance of the loglinear expression and the corresponding β parameters is illustrated by estimating the accident rates for two types of trucks (single and double trailer configurations) on a given highway link. The highway link is assumed to have the following characteristics:

- Road type: freeway;
- Traffic pattern: commuter;
- Traffic volume: high (i.e., AADT greater than 20,000 vehicles).

Other vehicle and driver characteristics are assumed to be as follows:

- Vehicle model/year: after 1977;
- Time period of shipment: 600–1800;
- Age of driver: between 25 and 54 years;
- Load status: fully loaded.

Application of the appropriate β parameters for these characteristics in Equation 9 yields the corresponding accident rate for a single trailer combination vehicle, such that

$$\begin{aligned}\log AR\ (11242122) &= -7.039 + 0.3838 - 0.1221 - 1.2110\\ &\quad + 0.6900 - 0.8897 + 0.2734 + 0.3604\\ &\quad - 0.4285 + 0.4029 + 0.7311\\ &= -6.8487\\ AR\ (11242122) &= \exp -7.6351 = 0.001061/10^3 \text{ truck-km}\\ &= 1.061 \text{ truck involvements}/10^6 \text{ truck-km}\end{aligned}$$

If all other factors are held constant, the corresponding accident rate for a tractor–two trailer combination is estimated as

$$\begin{aligned}AR\ (11252122) &= \exp -6.6868 = 0.001247/10^3 \text{ truck-km}\\ &= 1.247 \text{ truck involvements}/10^6 \text{ truck-km}\end{aligned}$$

The ratio for the two truck types for the same set of road, vehicle, and driver characteristics is estimated as

$$\text{Ratio}_{R=1} = 1.061/1.247 = 0.85$$

This ratio indicates that for the assumed set of conditions, loaded single trailer units experience lower accident rates, in general, than double trailer units. For roads that are not freeways, the ratio of accident rates between singles and doubles becomes even more pronounced, such that

$$\text{Ratio}_{R=2} = 1.43/2.96 = 0.48$$

Table 6 summarizes the accident rates for different truck configurations as estimated on a typical highway corridor in

Ontario (Figure 1). These values indicate that accident rates for large trucks are sensitive to road and vehicle characteristics. In general, doubles reflect the highest accident rates (per kilometer) for all truck types, especially on high-volume roads that are not freeways. Single trailer units experience the lowest accident rates on low-volume highways that are not freeways. On high-volume freeways the lowest accident rates are associated with single unit trucks and truck-trailer combinations. These rates are based on the assumption that the vehicle is fully loaded and being driven in a commuter-type traffic pattern by a 25–54-year-old driver during the 600 to 1800 time period. Changes in these assumed conditions result in a different distribution of accident rates by truck type for the same highway corridor.

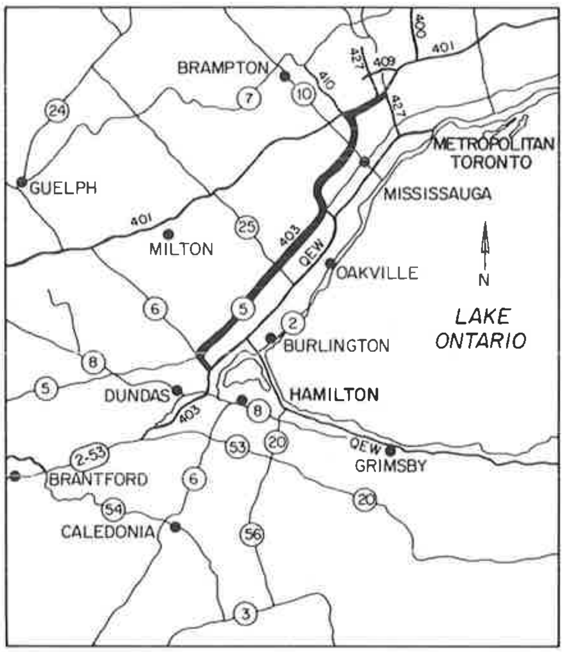

FIGURE 1 Reference map of an Ontario (Canada) highway corridor.

CONCLUSIONS AND RECOMMENDATIONS

Loglinear models of truck accidents explore higher-order interactions in the causal variables. The importance of these higher-order relationships in truck accident causation has been underscored in this analysis of Ontario truck accident data. Calibrated loglinear parameters in the truck accident rate model allow an assessment of the importance of individual factors in accident causation for a mix of mitigating factors.

The incompatibility between categorical accident data and continuous measures of exposure is a major problem for calibrating loglinear models of truck accident rates. The use of separate but structurally similar loglinear expressions for accident frequency and exposure does not result in stable estimates of accident rates. Exposure plays a significant and distinctive role in fitting accident rate models.

Loglinear calibration frequently makes use of a weighted least squares algorithm for estimating model parameters. For a contingency table of mitigating factors, the weighted least squares algorithm is highly sensitive to small cell counts. This approach for calibrating loglinear models is especially problematic for assessing factors that affect truck accident rates. This situation occurs because small cell counts, mostly sampling zeros, are not unexpected in truck accident data. A loglinear expression of truck accident rates should reflect categories in the contingency table for which truck travel or exposure is observable even in the absence of any accident involvement.

A generalized linear model for loglinear calibration (GLIM) is recommended on the basis of an analysis of truck accident and exposure statistics for Ontario. The β parameters for the truck accident rate expressions are calibrated by maximum likelihood techniques, which permit the inclusion of a continuous exposure variable as an offset in the loglinear expression. The algorithm can also accommodate cells in the contingency table with observed exposure but zero accident counts. A stepwise statistical analysis of higher-order interactions is also possible. The algorithm can accommodate cells in the contingency table with an observable level of exposure despite an absence of accident involvement.

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Estimation of Wet Pavement Exposure from Available Weather Records

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The estimation of wet pavement exposure is critical to the effective management of programs aimed at reducing wet pavement accidents. Without a measure of wet pavement exposure, highway safety engineers cannot tell whether differences in wet pavement accident frequencies between sites or over time represent actual safety problems or merely result from site-to-site or year-to-year variations in rainfall frequency. Laboratory and field tests were conducted to investigate two key aspects of wet pavement exposure estimation: (a) the conditions under which pavement wetness reduces pavement surface friction and (b) the time required for pavements to dry after rainfall. The results of these tests were used to develop an improved method of estimating wet pavement exposure from available weather records. This method has been incorporated into a computer model, known as the WETTIME model, for application by highway agencies. The model estimates the number of hours with wet pavement conditions on monthly and annual bases.

Wet weather accidents are an important element of the safety problem on U.S. highways. Recent research has found that approximately 13.5 percent of fatal accidents (*1*) and as many as 25 percent of all accidents (*2*) may occur under wet pavement conditions. The presence of water on the pavement reduces the available friction at the tire-pavement interface and may increase accident rates associated with maneuvers involving high friction demand, particularly accelerating, braking, and cornering.

The estimation of wet pavement exposure is critical to the assessment of wet weather accident experience. Wet pavement exposure estimates are needed both to allow traffic engineers to assess the overall priority that should be assigned to wet weather accidents in highway safety programs and to provide a reliable means of comparison for wet weather accident rates of highways located in different climatological regions. This paper presents a method for estimating wet pavement exposure from available weather records.

DEFINITION OF EXPOSURE MEASURES

Exposure estimates are used in highway safety studies as a measure of the opportunities for traffic accidents to occur. Typical exposure measures for traffic accidents include the number of sites considered, the duration of the time period for which accident data are available, the total length of the sites, or the total vehicle-miles of travel on those sites. The greater the exposure, the greater the number of accidents that would be expected to occur. To determine whether there are more accidents than expected at a site (or a group of sites), both an accident frequency and an exposure measure are needed. Thus safety measures used in accident surveillance often combine both accident and exposure measures for a given time period into an accident rate:

$$R = A/E \qquad (1)$$

where

R = accident rate (accidents/10^6 vehicle-miles);
A = number of accidents; and
E = exposure (10^6 vehicle-miles).

Thus accidents form the numerator and exposure forms the denominator of the accident rate expression.

Wet pavement exposure measures represent the portion of the total exposure that occurs under wet pavement conditions. If the numerator of the accident rate expression is annual wet pavement accidents, then the denominator should be annual vehicle-miles of travel under wet pavement conditions:

$$R_w = A_w/E_w \qquad (2)$$

where

R_w = wet pavement accident rate (accidents/10^6 vehicle-miles);
A_w = number of wet pavement accidents; and
E_w = wet pavement exposure (vehicle-miles).

Wet pavement accidents are defined by the road surface condition at the time of the accident as recorded by police officers, motorists, or both on the accident report form. The categories used for road surface condition on accident report forms are typically (a) dry, (b) wet, and (c) ice and snow. Wet pavement accidents are those that occur when the road surface is wet, whether or not it is actually raining at the time of the accident.

The annual wet pavement exposure (E_w) can be estimated most directly as the product of total annual vehicle-miles of travel (E) and the proportion of annual hours during which the pavement is wet. Previous wet pavement exposure estimation methods have focused on how to estimate this latter proportion.

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NEED FOR WET PAVEMENT EXPOSURE ESTIMATES

Most highway agencies currently have a program for identifying and treating locations with large numbers of wet pavement accidents, often as part of their computerized accident surveillance system. The locations identified by the program are reviewed through engineering studies to determine whether a correctable safety problem exists. These locations become candidates for improvement projects to increase tire-pavement friction (such as pavement resurfacing) and improvement projects to reduce the need for tire-pavement friction (such as realignment or other geometric modifications).

Accident surveillance programs identify potential improvement locations by comparing the accident frequencies or rates at specific locations with average values or with selected critical values. Typically, the computer analysis may evaluate the accident experience of a fixed-length section (say, 0.3 mi) that moves along the highway in 0.01-mi increments. For example, a 0.3-mi highway section might be classified as a high-accident section if it had either a wet pavement accident rate at least 20 percent higher than the average wet pavement accident rate or more than five wet pavement accidents per year.

A review of wet pavement accident surveillance programs by the National Transportation Safety Board (NTSB) (*1*) in 1980 found that most states do not use any wet pavement exposure measure in their wet pavement accident surveillance programs. If no wet pavement exposure measure is available, the wet pavement accident rate is typically defined with wet pavement accidents in the numerator and total exposure in the denominator, as follows:

$$R'_w = \frac{A_w}{E} \tag{3}$$

where

R'_w = modified wet pavement accident rate (accidents/10^6 vehicle-miles);
A_w = number of wet pavement accidents; and
E_w = total exposure under all pavement conditions (vehicle-miles).

This hybrid measure (R'_w) has been used both in state accident surveillance systems and in past wet pavement accident research. The potential problem with R'_w as a measure of wet pavement accident rate is that it is not sensitive to the geographic variations in climate within a state or the variations in climate from year to year. An accident surveillance program that monitors R'_w or the raw frequency of wet pavement accidents will tend to identify as problem sections those highways in areas that get the most rainfall. Highway sections with low pavement surface friction that are located in drier areas might go untreated even if they experience unusually high accident rates when the pavement is wet.

Precipitation amounts also vary from year to year within each state and across the nation. It would be erroneous to interpret an increase in wet pavement accident frequency as a developing safety problem if it resulted, in fact, from an increase in rainfall from one year to the next. Thus wet pavement exposure estimates are also needed to account for year-to-year changes in climate.

AVAILABLE WEATHER RECORDS

The development of an explicit method to estimate wet pavement exposure requires detailed weather records. The most detailed weather records that are normally available for major weather stations are recorded on an hourly basis. All previous efforts to estimate wet pavement exposure have been based on the hourly weather data available from the National Climatic Data Center (NCDC) in Asheville, North Carolina.

The most commonly used weather records for exposure estimation are the hourly precipitation data, which are available on computer tape from NCDC. These data contain a record of the hourly precipitation amount (in inches) for each hour in which a measurable amount of precipitation (at least 0.01 in.) occurred. The hourly precipitation data are available for thousands of stations throughout the United States but are generally complete and reliable only for first-order weather stations, which are typically located at major airports (*3*).

Most previous attempts to estimate wet pavement exposure have been based solely on the hourly precipitation data. However, there are other forms of weather records available on an hourly basis that provide a valuable supplement to the hourly precipitation amounts. These are the hourly surface observations that are available on computer tape from NCDC. These data are also available for first-order weather stations and include hourly data on

- Air temperature;
- Dew point temperature;
- Relative humidity;
- Wind speed;
- Cloud cover; and
- Occurrence of rain, snow, or fog.

These data provide a more complete understanding of hourly weather than precipitation amounts alone.

EXISTING WET PAVEMENT EXPOSURE ESTIMATION METHODS

There have been several previous attempts by highway agencies and researchers to develop a wet pavement exposure estimation method. It has been obvious to all investigators that annual precipitation totals by themselves are not adequate for estimation of wet pavement exposure. Some climatic regions commonly experience cloudbursts, in which large amounts of rain fall in a very short time period. Other regions experience drizzle, in which small rainfall amounts are spread over a long time period. Therefore all previous attempts to estimate wet pavement exposure have, in one way or another, examined the number of hours in which rainfall occurred.

A 1972 study by the California Department of Transportation (*4*) defined wet pavement exposure as the total number of hours during which a measurable amount (0.01 in. or more) of rainfall occurred. Trace amounts of rainfall were not considered. This method was used to estimate wet pavement exposure from the NCDC hourly precipitation data. A similar definition of wet pavement exposure has been used in studies by NTSB (*1*) and by the states of Arizona (*5*) and Michigan (*6*).

An alternative wet pavement exposure estimation technique was developed by Midwest Research Institute (MRI) in the late

1970s. The MRI method was developed in NCHRP Project 6-11, "Economic Evaluation of the Effects of Ice and Frost on Bridge Decks" (*7*). The method was further refined by MRI in the 1978 FHWA study "Effectiveness of Alternative Skid Reduction Measures" (*2*). The technique differed from the California/NTSB technique in that it included explicit consideration of the drying period during which pavements remain wet after rainfall ceases and the period during which pavements are wet due to melting of snow and ice. The original MRI technique also considered wet time due to trace amounts of rainfall (less than 0.01 in./hr) that are part of a longer period during which measurable rainfall occurs but ignored periods of rainfall composed entirely of trace amounts. The development of the technique included field observations of pavement drying times, and the technique was validated with wet pavement exposure data from a moisture sensor implanted in an Interstate highway bridge near Iowa City, Iowa.

One major weakness of the California/NTSB approach is that it makes no distinction between frozen and nonfrozen precipitation. This distinction is important in an exposure measure because accidents classified by road surface conditions have separate categories for wet pavements and for ice- and snow-covered pavements. Thus the California/NTSB method is only applicable to snow-free areas or to data from which the winter months have been excluded. This limitation may not be critical in California, where snowfall is rare in most populated areas, but it is important for the nationwide application attempted by NTSB (*1*).

The original MRI model attempted to account for many of the weaknesses described above but did so imperfectly because of the lack of research into the role of these factors in pavement wetness. The MRI model was more complex than the California/NTSB model because it considered both NCDC's hourly precipitation data and data on the type of precipitation, also available from NCDC.

Each of these previous attempts to estimate wet pavement exposure had limitations resulting from the lack of valid research findings concerning the role of key meteorological and pavement factors in wet pavement exposure. These limitations have been addressed in the development of a new wet pavement exposure estimation model, known as the WETTIME model.

MODEL SCOPE

The WETTIME model examines precipitation and weather data on an hour-by-hour basis and classifies all or portions of each hour as DRY time, WET time, or ICE and SNOW time. Monthly and annual totals of the number of hours of exposure to each type of pavement surface condition are obtained.

The WETTIME model is intended to provide a tool for use by highway agencies to estimate wet pavement exposure. The elements of the model draw on the strengths of both the California/NTSB model (*1, 4*) and the original MRI model (*2, 7*) and correct the weaknesses of these models through laboratory and field testing, as well as analytical and observational studies. The model is based to the greatest possible extent on valid research findings rather than on engineering judgment.

The model development recognized the need to distinguish clearly between wet pavement exposure time and ice and snow exposure time. The NCDC hourly precipitation data cannot be used alone for this purpose because these data do not distinguish frozen from nonfrozen precipitation. The most readily available data source in which this distinction can be made is the NCDC hourly surface observations. As mentioned earlier, these observations also include hourly measurements of air temperature, dew point temperature, relative humidity, wind speed, and cloud cover that can be used to enhance the accuracy of the exposure estimation model. However, the need for both types of input data limits the direct application of the model to first-order weather stations. There are only about 4 to 10 first-order stations in each state, typically located at major airports. A method for extending the WETTIME model estimates to additional weather stations was developed so that isoexposure contour maps of selected states could be developed. This process is discussed later in this paper.

MODEL ELEMENTS

The following elements have been incorporated in the WETTIME model:

- Minimum level of wetness that reduces pavement surface friction;
- Rainfall intensity and duration;
- Runoff period following rainfall;
- Pavement drying period following rainfall and runoff;
- Pavement wetness due to fog; and
- Estimation of exposure to ice and snow conditions.

Each element of the model is discussed in this section.

Minimum Level of Wetness That Reduces Pavement Surface Friction

None of the earlier wet pavement exposure estimation models explicitly addressed whether the rainfall amounts considered by the model were sufficient to reduce pavement surface friction to the point of slipperiness. The existing exposure estimation techniques assume that 0.01 in. of rainfall in an hour, or in some cases a trace amount of rainfall in an hour, is sufficient to result in slipperiness. However, neither of the existing models provides any justification for this assumption on the basis of valid research findings.

A critical review of the literature related to the relationship between tire-pavement friction and water film thickness was undertaken as part of the development of the WETTIME model. The relationship between pavement friction and water film thickness for thin water films has been investigated by a number of researchers including Besse (*8*), Giles (*9*), Gegenbach (*10*), Veith (*11*), Pelloli (*12*), Williams and Evans (*13*), and Rose and Gallaway (*14*). These sources do not provide a satisfactory relationship between pavement friction and water film thickness; several of the sources suggest that this relationship may have a negative exponential form at speeds above 25 mph, but no relationship was found at lower speeds.

Because no satisfactory relationship was found in the literature, laboratory and field studies were undertaken to determine the minimum level of wetness that substantially reduces pavement surface friction. The results of these studies, which have been reported by Harwood et al. (*3*) and Kulakowski et al. (*15*), suggest that as little as 0.001 in. of water on a pavement surface can, in some cases, reduce the friction coefficient 75 percent of the way from the dry friction to the wet friction value. This minimum level of wetness is likely to be exceeded during any hour in which there is at least 0.01 in. of rainfall. Thus all measurable amounts of rainfall in the available NCDC hourly precipitation data are likely to exceed the minimum level of wetness and should be considered as wet pavement exposure in the WETTIME model.

Rainfall Intensity and Duration

Existing wet pavement exposure models make no distinction between hours of precipitation based on rainfall intensity or duration. These models operate under the assumption that the pavement was wet for the entire hour during any hour in which at least 0.01 in. of precipitation fell. This is unrealistic because it would be expected that, on the average, the more rain that falls during an hour, the longer the rainfall would last; however, no data were previously available to quantify this phenomenon.

To determine the relationship between rainfall amounts and rainfall duration, Harwood et al. (*3*) obtained and analyzed the U.S. Geological Survey (USGS) urban stormwater data base (*16*). This data base contains rainfall amounts by 5-min periods for 717 selected periods of rainfall at 99 stations located in 22 metropolitan areas throughout the United States. Table 1, which is based on the USGS data from Florida, Maryland, Missouri, and Washington, presents the relationship that was developed. The duration of rainfall in Table 1 extends from the first 5-min period in which rain fell to the last 5-min period in which rain fell during a particular hour. Thus the duration could include short periods during which the rain ceased, but it is likely that the pavement would remain wet during such periods. The relationship in Table 1, which has been incorporated in the WETTIME model, indicates that the duration of rainfall increases with the hourly rainfall amount up to 0.05 in. of rainfall in 1 hour. Above that level, pavement wetness due to rainfall typically lasts for the entire hour.

TABLE 1 VARIATION IN DURATION OF RAINFALL DURING AN HOUR WITH HOURLY RAINFALL AMOUNT (*3*)

Hourly Rainfall Amount (in.)	Duration of Rainfall in WETTIME Model (min)
0.01	15
0.02	30
0.03	45
0.04	45
0.05 and over	60

Runoff Period After Rainfall

There is a period after the end of active rainfall when the pavement remains wet while water is running off the pavement. For purposes of the WETTIME model, it was assumed that pavement drying does not begin until runoff is complete.

The time required for water to flow off the pavement while rain is falling can be determined from the kinematic wave method (*17*) as a function of the rainfall intensity (in inches per hour), the pavement surface texture (represented by the Manning coefficient), the length of the drainage path, and the slope of the pavement surface. Estimates of runoff time calculated for typical ranges of values of these parameters indicated that runoff time is usually less than 10 min and is often 5 min or less. Although the kinematic wave approach is not directly applicable to a period after the rain has stopped, the runoff time at the end of rainfall would be similar to the runoff time for a very low rainfall intensity, such as 0.10 in/hr.

On the basis of the available data, it was found that typical runoff times after rainfall would range from 0 to 10 min. The differences in runoff times between sites would not be expected to have a major effect on the annual percentage of pavement wet time, so to keep the model simple, a uniform runoff time of 5 min after the end of rainfall was incorporated in the WETTIME model.

Pavement Drying Period Following Rainfall and Runoff

Previous studies have estimated the typical pavement drying time following rainfall and runoff at 30 min (*1, 2*); however, these estimates were based on limited observation and did not account for the possible influence of environmental and pavement variables on pavement drying time. Therefore laboratory and field studies of pavement drying time were undertaken as part of the development of the WETTIME model.

The laboratory study of pavement drying time was conducted in an 8 × 8 × 8–ft chamber constructed so that environmental conditions could be controlled. Air temperature was controlled by a heater and an air conditioner, and relative humidity was controlled by a humidifier. Solar radiation was simulated by an array of solar lamps, and wind was simulated by a large fan. Instruments, including a thermometer, hygrometer, and anemometer, were installed in the chamber to monitor the environmental conditions.

A Class A evaporation pan filled with water was placed in the chamber. Various asphalt and portland cement concrete (PCC) pavement samples were wetted and placed in the evaporation pan so that the pavement surface was above the water surface. The time required for the pavement surface to dry was monitored for selected combinations of environmental conditions. The thickness of the film of water on the pavement surface was monitored with a micrometer depth gauge at 5-min intervals during the drying period, and the water level in the evaporation pan was also monitored with a hook gauge. The independent variables considered in the pavement drying tests and the levels considered for each variable were as follows:

Solar Radiation

Nighttime or overcast (0 Langleys/min);
Partly cloudy day (0.75 Langleys/min); and
Bright, cloudless day (1.15 Langleys/min).

Wind Speed

No wind (0 mph);
2 mph;

8 mph; and
15 mph.

Air Temperature

60°F;
75°F; and
90°F.

Relative Humidity

45 percent;
60 percent;
75 percent; and
90 percent.

Pavement Type

Asphalt concrete; and
Portland cement concrete (PCC).

The pavement drying tests were conducted in accordance with an experimental design based on a Greco-Latin square that allowed assessment of the effects of the independent variables. To keep the required number of pavement drying tests to a minimum, the experimental design chosen could evaluate the main effects of each of the independent variables but could only evaluate selected interactions between pairs of independent variables. In all, 132 pavement drying tests were conducted.

The test data were analyzed, and a predictive model for pavement drying time was developed (*3*). This model is presented in Table 2, which shows the deviation from the mean drying time of 31.6 min for each level of each factor. The expected drying time for a particular combination of factors can be determined from the table; for example, the expected drying time for an asphalt pavement on a partly cloudy day with an air temperature of 75°F, 75 percent relative humidity, and wind speed of 5 mph would be

$$31.6 - 0.7 - 1.6 + 5.6 - 11.6 + 3.9 = 27.2 \text{ min.}$$

The model results indicate that pavement drying time can range from a minimum of about 5 min to a maximum of about 60 min, depending on conditions.

The two variables with the strongest influence on pavement drying time were solar radiation and wind speed. Either solar radiation equivalent to a bright, cloudless day (1.15 Langleys/min) or wind speeds of 1.5 mph or more were enough to cause very fast drying times. In contrast, pavement drying times were relatively long under nighttime conditions with no wind. The effects of air temperature and relative humidity on pavement drying time were also statistically significant but were not as important as the effects of solar radiation and wind speed. Pavement drying time was found to increase as relative humidity increased and to decrease as air temperature increased. Finally, pavement type was found to have a small but statistically significant effect on pavement drying time. Portland cement concrete pavements were found to dry, on the average, about 8 min faster than asphalt pavements.

TABLE 2 PARAMETER ESTIMATES FOR PAVEMENT DRYING TIME MODEL (*3*)

Factor	Mean Drying Time[a] (min)	Deviation From Overall Mean Drying Time (min)[b]
Temperature		
Below 67.5°F	35.3	+3.7
67.5–82.5°F	30.9	− 0.7
Above 82.5°F	28.6	− 3.0
Relative humidity		
Below 50 percent	27.1	− 4.5
50–82.5 percent	30.0	− 1.6
Above 82.5 percent	37.7	+6.1
Solar radiation		
Night or overcast	43.2	+11.6
Partly cloudy day	37.2	+5.6
Clear day	14.4	−17.2
Wind speed		
No wind	43.2	+11.6
Wind present	20.0	−11.6[c]
Pavement type		
Asphalt concrete	35.5	+3.9
Portland cement concrete	27.7	− 3.9

[a]The mean drying times represent the effects of each factor taken one at a time, independent of the values of the other factors.
[b]Deviation from overall mean drying time of 31.6 min.
[c]Use this parameter estimate only if the parameter estimate for the solar radiation factor has a positive value.

Field tests were conducted to verify the predictive ability of the pavement drying time model in Table 2. These tests included drying time observations for artificially wetted pavements and after actual rainstorms. The model was found to provide realistic estimates of pavement drying time except in a few cases. In these instances, observed drying times were much longer than the predicted times. This phenomenon was probably due to nearly saturated atmospheric conditions that inhibited evaporation from the pavement surface. As a result, a feature was added to the WETTIME model to delay the beginning of pavement drying when the dew point temperature was within 2°F of the ambient air temperature.

Pavement Wetness Due to Fog

The WETTIME model includes consideration of pavement wetness due to fog. In some situations, a pavement can become wet merely due to the pavement condensation or misting conditions associated with fog. Existing wet pavement exposure estimation models varied greatly in their consideration of pavement wetness due to fog. The California/NTSB approach does not consider the possibility of pavement wetness due to fog at all. The original MRI model classified all periods of fog as resulting in pavement wetness.

In the WETTIME model, pavement wetness due to fog is considered only when the ambient air is nearly saturated. Nearly saturated conditions are identified on the same basis for fog as for the delay in the beginning of pavement drying discussed earlier; that is, pavement wetness due to fog occurs only when the NCDC hourly weather observation data indicate that fog was observed and that the dew point temperature is within 2°F of the ambient air temperature.

```
WET-PAVEMENT EXPOSURE ESTIMATE                                    PAVEMENT TYPE: 1
REVISED MRI RULE                                                  ASPHALT CONCRETE
STATION:   KANSAS CITY MO
YEAR:      1984
```

NUMBER OF HOURS BY EXPOSURE TYPE

	WET	ICE & SNOW	COMBINED	DRY	TOTAL	MISSING
JAN	8.1 1.1%	30.0 4.0%	38.1 5.1%	705.9 94.9%	744.0	0.0
FEB	59.6 8.6%	15.0 2.2%	74.6 10.7%	621.4 89.3%	696.0	0.0
MAR	103.0 13.8%	35.0 4.7%	138.0 18.5%	606.0 81.5%	744.0	0.0
APR	98.3 13.7%	0.0 0.0%	98.3 13.7%	621.7 86.3%	720.0	0.0
MAY	41.7 5.6%	0.0 0.0%	41.7 5.6%	702.2 '94.4%	744.0	0.0
JUN	51.8 7.2%	0.0 0.0%	51.8 7.2%	668.2 92.8%	720.0	0.0
JUL	19.7 2.7%	0.0 0.0%	19.7 2.7%	724.2 97.3%	744.0	0.0
AUG	13.8 1.9%	0.0 0.0%	13.8 1.9%	730.2 98.1%	744.0	0.0
SEP	60.0 8.3%	0.0 0.0%	60.0 8.3%	660.0 91.7%	720.0	0.0
OCT	155.0 20.8%	0.0 0.0%	155.0 20.8%	589.0 79.2%	744.0	0.0
NOV	41.4 5.8%	8.0 1.1%	49.4 6.9%	670.6 93.1%	720.0	0.0
DEC	79.0 10.6%	19.0 2.6%	98.0 13.2%	646.0 86.8%	744.0	0.0
TOT	731.5 8.3%	107.0 1.2%	838.5 9.5%	7945.5 90.5%	8784.0	0.0 0.0%

FIGURE 1 Sample output from the WETTIME model: wet pavement exposure for Kansas City, Missouri, in 1984.

Pavement wetness due to fog is most likely when the pavement temperature is colder than the ambient air temperature. Unfortunately, there is no way to determine pavement temperature from available weather data, so this aspect of pavement wetness due to fog is not considered in the WETTIME model.

Estimation of Exposure to Snow and Ice Conditions

The consideration of snow and ice conditions is important in the WETTIME model in two ways. First, it is important that snow and ice conditions be considered separately and not be included in wet time, as is done in the California/NTSB approach. Second, pavement wetness can result from ice and snow conditions, especially during periods when melting of ice or snow on the pavement may occur or when meltwater might run onto the pavement.

Very little is known about predicting exposure to snow and ice conditions, but the NCDC hourly weather observation data can be used to classify precipitation as frozen or nonfrozen and to determine whether the air temperature is above or below the freezing point. Because there is no better information, ice and snow exposure due to frozen precipitation is estimated by the WETTIME model in a manner similar to the estimation of wet pavement exposure.

Model Summary

A summary of the exact rules that are used in the WETTIME model to classify an entire month or year into DRY time, WET time, and ICE and SNOW time will now be presented. The rationale for each of these rules was explained in the previous section.

An hour with no precipitation is counted as DRY, unless there is still pavement drying under way from the previous hour.

If nonfrozen precipitation of 0.01 in. or more occurs during an hour, then the time while the rain is falling and the subsequent drying time are counted as WET.

- For an isolated hour of precipitation (no precipitation in either the previous or the following hour), the duration of pavement wetness due to the rainfall is determined as follows:

Total Amount of Rainfall During the Hour (in.)	*Duration of Wetness*
0.01	15 min + runoff + drying time
0.02	30 min + runoff + drying time
0.03–0.04	45 min + runoff + drying time
0.05 or more	60 min + runoff + drying time

The rainfall period, whatever its duration, is assumed to be centered within the hour. For example, a 30-min rainfall period is assumed to start at 15 min past the hour and end at 15 min to the next hour.

- For the first hour of 2 or more consecutive hours of precipitation, the duration of wetness is determined as described previously. Whatever the duration of the rainfall period, it is assumed to occur at the end of the hour.
- For the last hour of 2 or more consecutive hours of precipitation, the duration of wetness is also determined as described previously. Whatever the duration of the rainfall period, it is assumed to occur at the beginning of the hour.
- For a middle hour of a period of 3 or more consecutive hours of precipitation, the rainfall is assumed to last for the entire hour.
- The pavement remains wet during a runoff period of 5 min after the end of rainfall.
- Pavement drying usually begins at the end of rainfall and runoff and continues until the pavement is dry or a new storm begins. If the pavement is still wet at the end of an hour, pavement drying continues into the next hour.
- The start of pavement drying may be delayed if the ambient air is nearly saturated (as indicated by a dew point temperature within 2°F of the ambient air temperature). During the daytime, the delay in the start of pavement drying will last a maximum of 2 hr or until the air is no longer saturated. At night, the delay in the start of drying will last until the air is no longer saturated or until drying by solar radiation begins shortly after dawn.
- The duration of pavement drying is determined from a statistical model that predicts drying time (presented in Table 2). The factors that predict pavement drying time are solar radiation, wind speed, air temperature, relative humidity, and pavement type. The predicted pavement drying time is rounded to the nearest 5 min. The program user can specify the pavement type (asphalt or PCC). If no pavement type is specified by the user, asphalt concrete is assumed as the default.
- The environmental factors in the pavement drying model are determined from weather data in the following manner:

Solar Radiation. Determined from a solar ephemeris routine that predicts solar radiation, levels considering month of year, time of day, and sky cover.
Wind Speed. No wind present: 0 or 1 mph;
Wind present: 2 mph and over.
Air Temperature. 67°F or below;
68°–82°F;
83°F or above.
Relative Humidity. 49 percent or below;
50–82 percent;
83 percent or above.

If fog occurs during an hour, the air is nearly saturated (dew point temperature within 2°F of ambient air temperature), and the wind speed is 3 mph or less, then the hour is counted as WET. Pavement drying after a period of fog follows the same rules as it does after a period of nonfrozen precipitation.

If frozen precipitation of 0.01 in. or more occurs during an hour, then the hour is counted as ICE and SNOW. The pavement drying time after a period of frozen precipitation is counted as WET.

MODEL APPLICATION

The application of the WETTIME model is fully explained in the "User's Guide for the WETTIME Exposure Estimation Model" (*18*). This guide includes an explanation of how to obtain the weather data needed to run the model. Figure 1 presents a sample of the printed output obtained from the model. Monthly and annual estimates of the hours of exposure to different pavement surface conditions are presented.

Test Cases for Several Geographic Regions

A number of test cases have been run with the WETTIME model to illustrate its application to a variety of geographic and

TABLE 3 EXPOSURE SUMMARY BY PAVEMENT SURFACE CONDITION FOR 1984 AT SELECTED FIRST-ORDER WEATHER STATIONS

	Percentage of Annual Exposure		
	Wet	Ice and Snow	Dry
Florida			
Apalachicola	5.5	0.0	94.5
Daytona Beach	5.9	0.0	94.1
Fort Myers	13.6	0.0	86.4
Gainesville	23.0	0.0	77.0
Jacksonville	21.6	0.0	78.4
Key West	3.7	0.0	96.3
Miami	6.9	0.0	93.1
Orlando	13.6	0.0	86.4
Tallahassee	19.6	0.0	80.4
Tampa	6.0	0.0	94.0
Vero Beach	7.8	0.0	92.2
West Palm Beach	9.2	0.0	90.8
Missouri			
Columbia	10.1	1.7	88.2
Des Moines, Iowa	8.7	1.9	89.4
Kansas City	8.3	1.2	90.5
Memphis, Tenn.	10.9	0.2	88.9
St. Louis	12.9	1.7	85.4
Springfield	6.6	1.1	92.3
Pennsylvania			
Allentown	8.7	1.8	89.5
Erie	8.4	5.7	85.9
Harrisburg	10.9	1.5	87.6
Philadelphia	9.4	0.8	89.8
Pittsburgh	9.5	3.7	86.8
Wilkes-Barre	9.7	2.5	87.8
Washington			
Astoria, Oreg.	21.9	0.0	78.1
Lewiston, Idaho	5.1	0.5	94.4
Olympia	23.5	0.3	76.2
Portland, Oreg.	15.2	0.1	84.7
Quillayute	38.5	0.2	61.3
Seattle	15.0	0.4	84.6
Spokane	9.9	2.7	87.4
Stampede Pass	20.9	11.3	67.8
Yakima	4.2	0.8	95.0

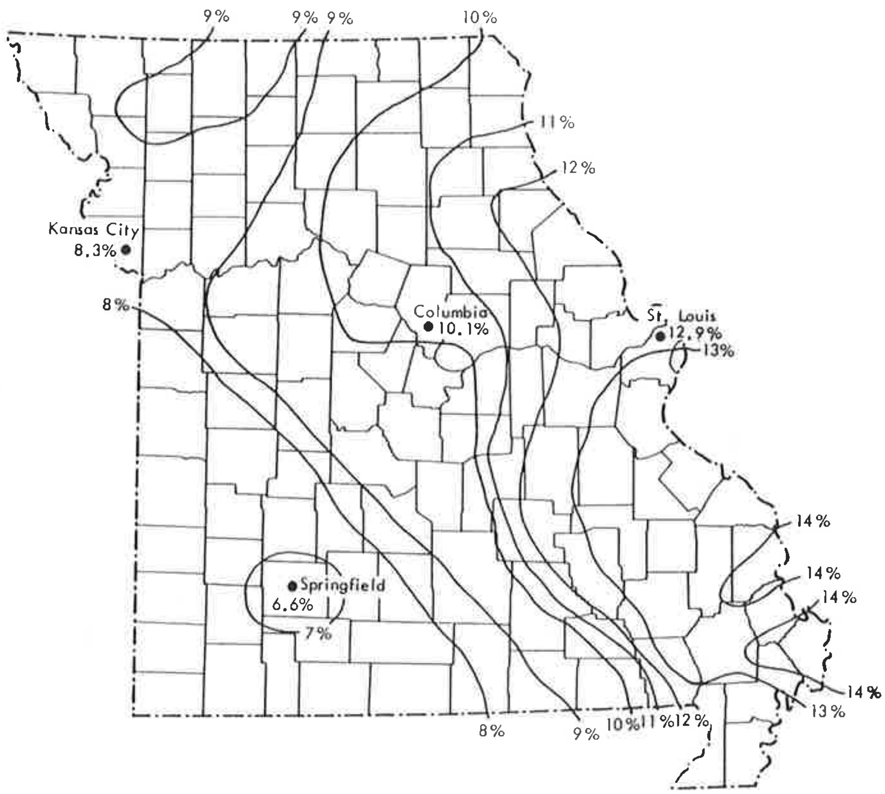

FIGURE 2 Isoexposure contour map for wet pavement exposure in Missouri (*3*).

climatic regions. Four states were selected for these test cases: Florida, Missouri, Pennsylvania, and Washington. Missouri and Pennsylvania were selected because they are typical of the climates in the midwestern and northeastern regions of the United States. Florida was selected because of its pattern of short, frequent rainfalls, and Washington was selected because of its variety of climates, ranging from desert to rain forest.

Table 3 presents the annual distribution of wet pavement exposure, ice and snow exposure, and dry pavement exposure for 1984 for each first-order weather station in the four selected states and for a few stations in adjoining states. Data are presented in the table for a total of 33 weather stations. Table 3 shows the broad range of wet pavement exposure in the selected states. Across these four states, wet pavement exposure time for 1984 ranges from a low of 4.2 percent in the desert areas of Washington to a high of 38.5 percent in the Washington rain forest. Ice and snow exposure time for 1984 ranges from a low of zero in Florida to a high of 11.3 percent in the Cascade Mountains of Washington. The data in Table 3 apply to highway sections with asphalt pavements; wet pavement exposure for PCC pavements would be slightly lower because they dry more quickly.

Isoexposure Contour Maps

The data provided by the WETTIME model can be used to construct contour maps showing the variations in exposure over an entire state or region. The data needed to apply the WETTIME model are available for only 4 to 10 first-order weather stations in each state. However, a procedure has been developed to use the estimates from the WETTIME model for the first-order stations within a state, together with total annual rainfall patterns, to obtain estimates of wet pavement exposure for minor stations (*3*). With wet pavement exposure estimates for a sufficient number of stations within a state, isoexposure contour maps can be constructed. Figures 2 and 3 present typical isoexposure contour maps for the states of Missouri and Washington, respectively.

Isoexposure contour maps like these could be an important guide to the management of wet pavement accident reduction programs by highway agencies. The maps indicate how widely wet pavement exposure can vary across a state even in a state such as Missouri, which has a relatively homogeneous climate. Even in Missouri, wet pavement accident rates, based on a statewide average for wet pavement exposure, could be high or low by as much as 50 percent.

Seasonal Variations

In general, the total annual vehicle-miles of travel under wet pavement conditions can be determined directly from total travel and the output of the WETTIME model:

$$E_w = E(P_w/100) \tag{4}$$

where

E_w = annual wet pavement exposure (vehicle-miles);
E = total annual exposure (vehicle-miles); and
P_w = annual percentage of hours with wet pavement conditions.

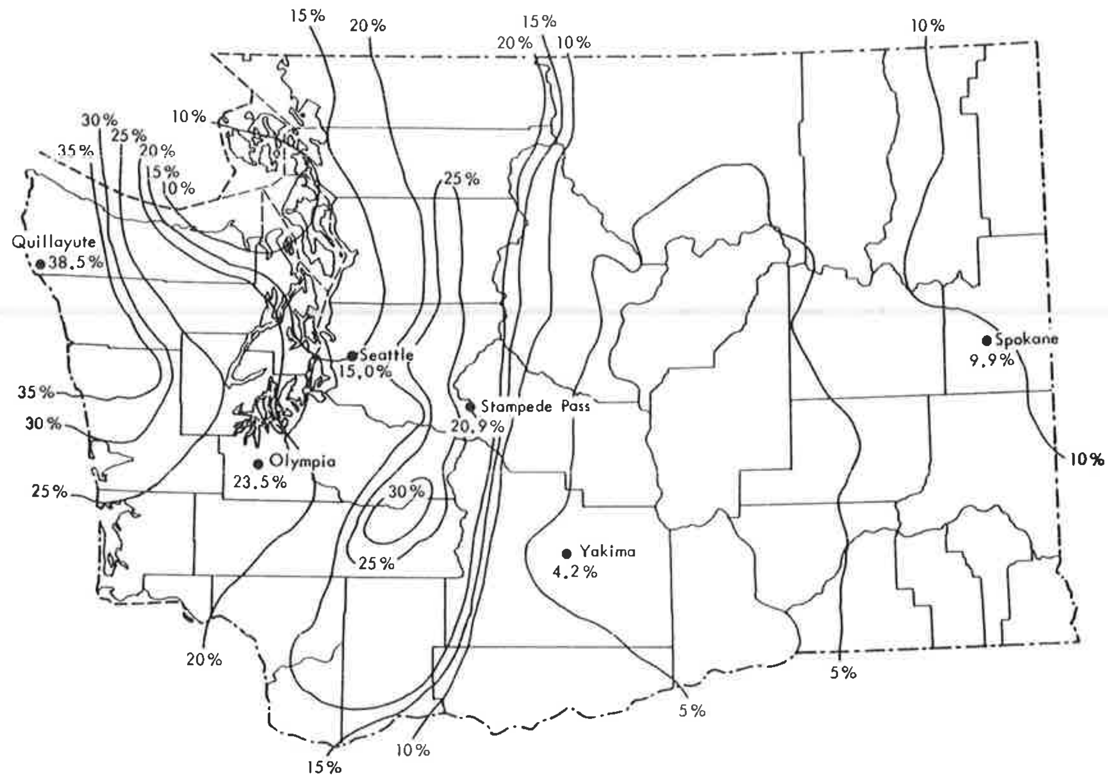

FIGURE 3 Isoexposure contour map for wet pavement exposure in Washington (3).

However, in some regions of the United States, it may be desirable to consider both seasonal variations in wet pavement exposure and seasonal variations in traffic volumes. For example, the California Department of Transportation (4) has observed that their seasonal patterns of rainfall and traffic volume are opposite, with more traffic and less rainfall in the summer months. The month-by-month output of the WETTIME model, as illustrated in Figure 1, can be used as a basis for adjusting the annual wet pavement exposure estimate for monthly variations in travel, in the following manner:

$$P_w = \sum_{i=1}^{12} P_{w_i} \frac{VMT_i}{VMT} \quad (5)$$

where

P_{w_i} = monthly wet pavement exposure percentage in month i;
VMT_i = monthly vehicle-miles of travel in month i; and
VMT = annual vehicle-miles of travel.

However, this method is only recommended for states with particularly large seasonal variations in precipitation patterns or in travel.

SUMMARY AND RECOMMENDATIONS

This paper presents an improved method for estimating wet pavement exposure from available weather records. The method is based on the results of laboratory and field tests that examined the conditions under which pavement wetness reduces friction and the time required for pavements to dry following rainfall. The improved method has been incorporated into a computer program, known as the WETTIME model, for use by highway agencies.

The WETTIME model is recommended as a tool for use in effective management of wet pavement accident reduction programs. Isoexposure contour maps such as those presented in the paper provide a basis for calculation of more accurate wet pavement accident rates for use in accident surveillance. Further research could automate the preparation of isoexposure contour maps and make this information more accessible to highway agencies.

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Evaluation of the Accident Rates of Male and Female Drivers

ABISHAI POLUS, IRIT HOCHERMAN, AND EHUD EFRAT

The goal of this study was to conduct a comparative evaluation of accident rates and patterns for male and female passenger automobile drivers. Two sections of road in Israel, one urban and one rural, were selected for the study. Counts of passenger automobiles by sex of driver were carried out on each section of road. The relative accident rates for male and female drivers on the two roads were assessed by estimating the relative exposure of the two groups and matching it with relative accident frequencies. Accident patterns in terms of severity and type were also compared for the two sexes. The same comparison was made for accidents that occurred on all urban and rural roads in Israel during 1986. It was found that, on the average, accident rates are similar for men and women, for both urban and rural driving conditions. No significant differences in the severity of accidents were found. On rural roads, women were found to be involved in more single-vehicle accidents, whereas men were involved in more collision accidents. Evidence for the lack of driving experience for women was found both in the literature and in the study data. In contrast, women were found to drive considerably more slowly on rural roads. It is suggested that the lesser experience and greater caution of women may counterbalance each other and this was the reason for the similarity found in the accident rates of men and women.

Comparative evaluation of the accident rates of male and female automobile drivers has important practical applications in driver training, education, and safety-enhancement programs. Normally, a comparison of driving characteristics and safety records between the sexes cannot be conducted easily because of the lack of a common basis for evaluation. Differences in the type of vehicles used, the type of trips performed, and the time of day in which travel takes place hamper meaningful comparison.

The majority of accidents usually reflects a problem in two and sometimes in all three of the major attributes of the transportation environment: traffic, road system, and human attributes. In other words, at the time of an accident, the driver is not performing satisfactorily for the conditions on the road, for the characteristics of his or her vehicle, or both. Often, a fundamental problem in the analysis of accident characteristics is the determination of the functional distribution of system demands and driver capabilities, and the implication frequently made is that the two functions are independent (*1*). Another major difficulty occurs in attempts to determine the exposure of a given category, such as class of vehicle, type of driver, or specified time frame. Therefore a comparison of accident rates may be impossible unless these obstacles are overcome.

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The purpose of this study was to conduct a comparative evaluation of accident rates for male and female automobile drivers, that is, to analyze the involvement of each sex relative to the appropriate amount of travel for its group. The number of accidents for both men and women can be determined relatively easily from existing police or statistical bureau files. The amount of travel conducted by each group, however, is unknown, and therefore the accident rates for each cannot be easily computed. Thus the aim of the investigation was to provide a valid macroscopic comparison of accident rates between male and female drivers.

The analysis focused on two sites in Israel, one an urban street in Jerusalem and the second a major rural freeway. This division was selected for two reasons: First, it is believed that urban and rural driving tasks are different. Driving in rural areas consists mainly of routine maneuvers performed at high speed, whereas driving on urban streets requires more complex decisions because the amount of information to be processed is huge, even though the speed is low. Second, it was initially assumed (and later confirmed) that the relative percentage of female drivers is considerably higher in urban areas.

The determinations of the number of accidents and the calculation of exposure were undertaken on exactly the same highways to eliminate any ambiguity in the results. In addition, a macroscopic comparison was conducted on national accident data. This comparison supported the general findings of the study.

BACKGROUND OF THE STUDY

Several studies have dealt exclusively with a comparison of accident rates for male and female automobile drivers. One such study, conducted in England (*2*), evaluated observed differences in accidents. It was concluded that, basically, little difference existed in the proportion of male and female drivers who were regarded as being at fault in accidents. The causes given for the accidents, however, differed considerably. Male drivers tended to drive too fast for the conditions, were more likely to be impaired by alcohol, and took risks more readily. Female drivers, on the other hand, made errors by being distracted and by not seeing hazards. It was also found that a female driver involved in an accident was likely to be less experienced in comparison with a male driver.

In an Australian study, Foldvary hypothesized that because of differences between the sexes, female drivers on the average will be more cautious, more hesitant, and less affected by alcohol (*3*). He found that the overall involvement rate for women was 80 percent of that for men. Women were also found to be slightly more involved in casualty than in non-casualty accidents. Their casualty rate was particularly high on Thursday, which is the main shopping day for a large sector of the population in metropolitan Brisbane, Australia. One of Foldvary's important conclusions was that middle-aged women appear to have, on the average, less driving experience than men of the same age group.

Additional evidence for the lower experience of female drivers was provided by Toomath and White, who surveyed driver exposure to risk (*4*). They found that for each age group, the total amount of distance driven by women is considerably lower than that driven by men.

The notion of experience was also investigated in Canada by Chipman (*5*), who suggested that experience is an important benefit of high exposure for many drivers because it may protect them from accidents in some circumstances. She emphasized that the positive effects of experience are related not so much to years of experience as to current exposure. Chipman suggested that women therefore enjoy a lower observed-to-expected ratio for city accidents than do men (because of their higher exposure to city driving), whereas men have the lower ratio for accidents on limited access highways. It was further suggested that the relationship between accident occurrence and driving experience could be used in driver education programs and in driver training.

A study of risky driving related to driver and vehicle characteristics was recently conducted in the United States by Evans and Wasielewski (*6*), who analyzed headways (time intervals between successive vehicles) in freely moving freeway traffic in Michigan and in Toronto, Canada. Women were found to be underrepresented in the shortest, most risky headways and overrepresented in the more cautious headways, which were about 1.5 seconds. Although the study did not deal directly with accidents, the researchers concluded from their data set that there is "clear evidence for sex-dependent driving behavior." Further evidence for this sex-dependent difference was found in an analysis of seat belt use. In both Michigan, United States, and Ontario, Canada, a higher proportion of women used seat belts, although only the Michigan values were statistically significant (16.6 percent for women and 12.2 percent for men). This finding was also consistent with a Canadian study that found seat belt use to be more prevalent among women (*7*). Kirkham and Landauer found that Australian women were underrepresented in all types of traffic offenses, even after allowing for exposure (*8*).

Additional care on the part of female drivers was reported previously by Parry in his 1968 study on aggression on the road (*9*). In a clear differentiation between male and female drivers, Parry found that whereas at least 50 percent of the male drivers he surveyed admitted to (for example) "cutting across if in the wrong lane of traffic," not a single woman did so. He suggested, tentatively (because of the relatively small sample size), that this could be due to higher anxiety while driving and, consequently, greater caution on the part of women. Generally, male drivers tended to be more aggressive and female drivers more anxious. Weber, on the other hand, found that men and women drive under very different circumstances and that the difference between their accident rates does not necessarily reflect difference in quality of male and female driving performance (*10*).

From this background review, it may appear that there is a distinct difference in driving behavior and experience between the sexes. The exposure to accident risk may therefore be variant. It can further be hypothesized that two conflicting elements control the behavior of women on the road: first, they are on the average more cautious, less willing to take risks, and less aggressive, and second, they have less experience and less exposure to driving. A comparison of road accident rates for men and women should be of considerable interest and may lead to some practical conclusions.

METHODOLOGY AND DATA COLLECTION

Any valid comparison of accident involvement among different groups of road users must take into account differences in exposure, that is, differences in mileage driven, in patterns of driving, and so on. This is also true when a comparison of the accident involvement rates of men and women is wanted. On the average, women drive less than do men, they make shorter trips, and a higher percentage of their trips take place during the day and on urban roads. On the rural network, the distribution of trips made by women is not similar to that of men; for example, the percentage of female drivers is higher near cities than on remote roads. The design of the current study attempted to control these differences in exposure.

The study was executed separately for urban and rural settings because traffic composition and driving conditions and, consequently, accident patterns are different on the two types of roads. If women do indeed drive in a different manner than do men, their relative risk may be different on each road type. Because women drive less frequently on rural roads, they have less experience in this type of driving. Their accident involvement rate on such roads is thus possibly affected.

Two sections of road were selected for the study, one urban and one rural. The sections were chosen so that their geometry and traffic composition, as well as their rate of female drivers, remained similar along the entire section (the last assumption was later verified). Another criterion was that the number of accidents occurring on each road section was sufficient for statistical analysis. The rural road chosen was Route 1, 60 km of four-lane divided highway connecting the metropolitan area of Tel Aviv to the capital, Jerusalem. The urban route was 8 km of arterial road going through Jerusalem.

The relative accident rates for male and female drivers on the two roads were assessed by estimating the relative exposure of the two groups and matching it with relative accident frequencies. Relative exposure of female and male drivers was observed in the following manner. On each of the two sections of road, an observation site was picked, and the number of male and female passenger automobile drivers passing this site in each direction was counted separately. (The observers encountered no difficulty in determining the gender of the driver.) The counts covered the hours between 6:00 a.m. and 6:00 p.m. on each day of the week, a total of 84 hours for each site. Control counts were performed at other sites along the two routes to check the assumption that the rate of female drivers is constant along these routes. All counts were performed over several weeks during the period April 1985–February 1986.

Accident data were collected from the files of the Israel Central Bureau of Statistics for the years 1982–1986 (*11*). These files contain only data on accidents involving casualties. Israeli law requires notifying the police of any accident involving casualties, but there is no obligation to report damage-only accidents. The authors do not believe that this has limited the generality of these findings because injury accidents cover a wide range of accident severity. As with exposure, only accidents involving passenger automobiles and occurring between 6:00 a.m. and 6:00 p.m. were considered. The analysis was restricted to passenger automobiles to eliminate any bias resulting from differences in the types of vehicles driven by men and women.

To test whether the involvement of female drivers in road accidents is different from that of male drivers, the percentage of women among drivers involved in accidents was compared with the percentage of women among all drivers. This comparison does not require knowledge of the absolute number of drivers of each sex; only the rate of female drivers is necessary. Thus the counts did not have to represent all traffic. The layout of the counts allowed for a changing proportion of female drivers during the day and among the days of the week; however, due to lack of contradictory data and the belief that no significant change in proportion of female drivers occurred, it was assumed that a constant proportion of women prevails throughout the year for the same day and hour.

In addition, the relative risk of female to male drivers was also computed. Relative risk is used for assessing risk in retrospective studies. The definition of relative risk (RR) in the current context is as follows: the accident rate per kilometer for female drivers divided by the accident rate per kilometer for male drivers. Along a fixed route, this translates into the following: RR = accident rate per female drivers/accident rate per male drivers, or

$$\frac{A_f/A_m}{D_f/D_m}$$

where A_f, A_m are the numbers of accidents in a given period for female and male drivers, respectively, and D_f, D_m are the numbers of female and male drivers, respectively, for the same period. Again, only the proportion of female drivers is needed. Accident patterns in terms of severity and type were also compared for the two sexes. The same comparison was made for accidents that occurred on urban and rural roads in Israel during 1986.

RESULTS

More than 90,000 automobiles were counted on each of the two road sections. The rate of women among all drivers on the rural road was 14.9 percent; on the urban road, 22.8 percent of the drivers were women. Table 1 presents the overall exposure rate, accident involvement, and relative risk for female versus male drivers at the two sites.

During the 5-year period 1982–1986, 384 drivers were involved in casualty accidents on the rural road. Of these, 51 were women, a rate of 13.3 percent. The 95 percent confidence limits for the proportion of female drivers involved in accidents is (9.9; 16.4). This interval contains the rate of exposure 14.9, so that the involvement rate of female drivers in accidents is not significantly different from their proportion on the road. The relative risk of accident involvement for female drivers is 0.88, with 95 percent confidence interval of (0.65; 1.17).

TABLE 1 INVOLVEMENT IN ACCIDENTS AND EXPOSURE OF MALE AND FEMALE DRIVERS, 1982–1986

	Urban Road	Rural Road
Percentage of women in traffic	22.8	14.9
Number of drivers involved in accidents	260	384
Percentage of women in accidents	22.7	13.3
Confidence limits for percentage of women in accidents	(17.6; 27.8)	(9.9; 16.7)
Relative risk	0.99	0.88
Confidence limits for relative risk	(0.74; 1.32)	(0.65; 1.17)

On the urban road during the same 5-year period, 260 drivers were involved in accidents, 59 (22.7 percent) of them women. The 95 percent confidence limits for the proportion of women drivers involved in accidents is (17.6; 27.8). Again, the accident-involvement rate of women drivers is clearly not significantly different from their proportion on the road. The relative risk of accident involvement for female drivers is 0.99, with 95 percent confidence interval of (0.74; 1.32).

Table 2 displays the distribution of accidents by severity for male and female drivers on the rural section of road. As can be seen, there was no significant difference between the sexes. On the urban road, 93 percent of the accidents were slight, so such a comparison could not be made.

Table 3 presents the male-female distribution by accident type for each road type. On the rural road, female drivers suffered relatively more single-vehicle accidents, whereas male drivers were involved more in collision accidents ($\chi^2_2 = 9.14$; $p = 0.01$). The same trend was found on the urban road, although the difference was not significant.

Similar results were obtained for the whole country for both rural and urban roads, as presented in Table 4. On the national

TABLE 2 DISTRIBUTION OF DRIVERS INVOLVED IN ACCIDENTS BY SEVERITY OF ACCIDENTS AND BY SEX OF DRIVER, 1982–1986

	Number of Drivers			Females
Severity	Female	Male	Total	(%)
Rural Road				
Slight	38	242	280	13.6
Severe	10	80	90	11.1
Fatal	3	11	11	21.4
Total[a]	51	333	384	13.3
Urban Road				
Slight	57	183	240	23.8
Severe	2	17	19	10.5
Fatal	–	–	–	–
Total[b]	59	200	259	22.8

[a] $\chi^2_2 = 1.19$ (N.S.).
[b] $\chi^2_1 = 1.08$ (N.S.).

TABLE 3 DISTRIBUTION OF DRIVERS INVOLVED IN ACCIDENTS BY TYPE OF ACCIDENT AND SEX OF DRIVER, 1982–1986

	Number of Drivers			Females
Type of Accident	Female	Male	Total	(%)
Rural Road				
Pedestrian	3	12	15	23
Single vehicle	21	76	97	20.2
Collision	27	245	272	9.9
Total[a]	51	333	384	13.3
Urban Road				
Pedestrian	8	35	43	18.6
Single vehicle	5	13	18	27.8
Collision	46	153	199	23.1
Total[b]	59	201	260	22.7

[a] $\chi^2_2 = 9.14$; $p = 0.01$.
[b] $\chi^2_1 = .69$ (N.S.).

TABLE 4 DISTRIBUTION OF DRIVERS INVOLVED IN ACCIDENTS IN THE WHOLE COUNTRY BY TYPE OF ACCIDENT AND SEX OF DRIVER, 1986

	Number of Drivers			Females
Type of Accident	Female	Male	Total	(%)
Rural Road				
Pedestrian	17	82	99	17.1
Single vehicle	61	234	295	20.7
Collision	226	1,204	1,430	15.8
Total[a]	304	1,520	1,824	16.7
Urban Road				
Pedestrian	407	1,307	1,714	23.7
Single vehicle	87	191	278	31.3
Collision	1,199	3,880	5,079	23.6
Total[b]	1,693	4,378	6,071	27.9

[a] $\chi^2_2 = 4.2$; $p = 0.12$.
[b] $\chi^2_2 = 8.6$; $p = 0.01$.

system, women were found to be more involved in single-vehicle accidents, while men were involved more in collision accidents.

The distribution of drivers involved in accidents by severity and sex for all urban and rural roads in Israel during 1985 is presented in Table 5. Women were found to be less involved in severe and fatal accidents than men. This trend was not confirmed on the single road studied, and this may be explained by the small sample.

Analysis of accidents by day of the week for the rural and urban sites was also conducted (Table 6). It can be observed that at both sites, the involvement rate for women on Saturdays was considerably lower than their relative exposure. On Saturday, which is the Sabbath in Israel, driving patterns are different than they are on weekdays. On urban roads, the involvement rate on Thursday was particularly high; this in accordance with Foldvary's 1979 finding, which was explained by the increased shopping activity that takes place on Thursdays (*3*). Weber also reported that women have a greater portion of their accidents on weekdays than do men and a lower portion on the weekend (*10*).

TABLE 5 DISTRIBUTION OF DRIVERS INVOLVED IN ACCIDENTS IN THE WHOLE COUNTRY BY SEVERITY OF ACCIDENTS AND SEX OF DRIVER, 1986

	Number of Drivers			Females
Severity	Female	Male	Total	(%)
Rural Road				
Slight	248	1,117	1,425	17.4
Severe	53	313	366	14.5
Fatal	3	53	56	5.4
Total[a]	304	1,543	1,847	16.5
Urban Road				
Slight	1,580	4,803	6,383	24.8
Severe	153	600	753	20.3
Fatal	12	50	62	19.4
Total[b]	1,745	5,453	7,198	24.2

[a] $\chi^2_2 = 8.342$; $p = 0.02$.
[b] $\chi^2_2 = 8.0$; $p = 0.02$.

TABLE 6 DISTRIBUTION OF DRIVERS INVOLVED IN ACCIDENTS BY DAY OF WEEK AND SEX OF DRIVERS, 1982–1986

	Day of the Week						
	Sun.	Mon.	Tues.	Wed.	Thurs.	Fri.	Sat.
Rural Road							
Total no. of drivers	79	70	54	48	67	42	35
No. of female drivers	11	10	11	2	9	7	1
Percentage of female drivers	13.9	14.2	20.3	4.2	13.4	16.6	2.9
Percentage of female drivers in traffic	16.4	15.4	15.4	15.2	14.8	13.7	13.3
Urban Road							
Total no. of drivers	47	46	37	29	46	35	20
No. of female drivers	10	12	9	6	17	4	1
Percentage of female drivers	21.3	26.0	24.3	20.7	37.0	11.4	5.0
Percentage of female drivers in traffic	24.5	24.6	25.7	25.5	23.9	17.6	15.1

DISCUSSION

The main finding of this study was that, overall, no significant difference exists in accident involvement rate between male and female drivers in Israel. This finding is in accordance with most previous studies dealing with this subject. Several differences between male and female drivers are suggested in the literature, and they can be adopted to explain variances in accident rates. Yet the rates for each sex are found to be similar and therefore the hypothesis of "balance of impacts" is proposed.

According to this hypothesis, involvement of female drivers in accidents is the result of two major, contradictory parameters. First, the behavior of female drivers is more cautious: they drive slower than do men, they take fewer risks, and they use seat belts more often (*2, 6, 7, 9, 10*). On the other hand, women have less experience in driving, they are more prone to distraction, and they make more errors of perception (*2, 4, 5*).

The result of these contradictory impacts is the creation of a general balance or equilibrium in the occurrence of accidents. On the average, therefore, the accident-involvement rate of women is not different from that of men. Weber also found that "men and women seem to be involved in accidents in about the same ratio as the amount they drive" (*10*).

The behavioral differences just described may account for the differences experienced by the two sexes in the distribution of accident types on the rural road studied as well as on the national highway system. The relatively higher involvement of men in collision accidents may be associated with risk-taking behavior, whereas the higher involvement of women in single-vehicle accidents may be related to lack of experience.

Further evidence for the cautious driving of women in Israel was provided in this study. Analysis of 2,232 speed observations—22 percent of which involved women—on the rural road showed that the average speed of women was 64 km/hr (with a standard deviation of 11.5 km/hr), whereas the average speed of men was 76 km/hr (with a standard deviation of 9.6 km/hr). This difference is significant at the 0.01 level.

As a further check on the balance of impacts hypothesis, the relative experience of Israeli women was also investigated. It was found that 32 percent of all driver's license holders in 1987 were women. In 1975, only 22 percent were women (*11*). This means that more newer drivers are women. Additionally, the distribution of years of driving experience by age group shows that for the older section of the population, the proportion of newer drivers is higher among women. As an example, Table 7 presents this distribution for the age group 35–44. Finally, although women constitute about one third (~33 percent) of the total driver population, their presence on the road was proportionally less: 23 percent for the urban road and only 15 percent for the rural site. This furnishes additional evidence of their lesser driving experience.

TABLE 7 DISTRIBUTION OF DRIVERS AGED 35–44 BY AGE OF FIRST DRIVER'S LICENSE AND SEX

	Age				
	≤24	25–29	30–34	35–39	40+
Men (%)	51.7	27.0	13.7	7.5	0.75
Women (%)	17.8	27.4	31.5	17.7	2.0

These findings of higher speeds for men on the one hand and less experience for women on the other hand provide additional support for the proposed hypothesis of a balance of impacts. The trend of less experience among women drivers may be diminishing gradually with time, as more women obtain driver's licenses earlier. This newer tendency may lead to changes that cannot be foreseen at this time in both driving patterns and accident rates. The resulting balance of impacts may also change with time.

CONCLUSIONS

The goal of this study was to conduct a comparative evaluation of accident rates and patterns for male and female passenger car drivers in Israel. It was found that, on the average, accident rates are similar for men and women, for both urban and rural driving conditions. No significant differences in the severity of accidents were found. Women were found to be more involved in single-vehicle accidents, whereas men were more involved in collision accidents.

When the distribution of accidents by day of the week was considered, it was found that women have a particularly low accident rate on Saturdays. In addition, it was found that their involvement rate on urban roads was highest on Thursdays, which is the main shopping day in many Israeli households.

Evidence for the lack of driving experience for women was found both in the literature and in the study data. In contrast, women were found to drive considerably more slowly on rural roads. It is suggested that the lesser experience and greater caution of women may counterbalance each other. This was therefore proposed as the reason for the similarity found in the accident rates of men and women.

Further research to sustain this evaluation may be conducted on (a) other types of road, such as secondary rural roads, (b) behavioral differences in driving patterns of men and women and their relationship to the accident patterns of the two sexes, and (c) the relationship between driving experience and both the behavior and accident rates of women as compared to those of men.

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Highway Accidents and the Older Driver

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The research reported in this paper examined the relationship between driver age and highway accidents by using a variation of the induced exposure method. In this method the exposure of drivers to highway accidents is represented by their relative probability of being the driver of the vehicle that was not cited for a contributing hazardous action in an accident, that is, involvement as an "innocent victim" in highway accidents. The measure of induced exposure used in this research is based on the information contained in the computerized accident records prepared by the Michigan Department of State Police and maintained by the Michigan Department of Transportation. These records contain the information reported by the investigating officer at the scene of the accident. The results indicate that the accident involvement of elderly drivers is higher than that of other drivers.

The research project described in this paper had two objectives, namely, determination of the most feasible approach to the study of the relationship between driver age and highway accidents and, by using this approach, identification of those factors or conditions that tend to indicate greater accident involvement for older drivers.

In recent decades, medical advances have greatly increased the average life expectancy. Elderly people are a greater percentage of the total population today than ever before (*1, 2*). Additionally, the automobile has become an integral part of society, particularly in the United States. Therefore a greater percentage of drivers today are elderly people (*3*), and these percentages are expected to continue increasing during the next few decades.

If driver safety is measured by the number of accidents per licensed driver, elderly people appear to have a good safety record. Typically, researchers base accident rates on the number of drivers in the various age groups. A representation of this type for single- and double-vehicle accidents in Michigan over the period 1983–1985 is given in Figure 1. If this approach is taken, the results indicate that driver safety increases with age. However, most elderly drivers do not drive as often or as far as other drivers. Traditionally, a more accurate measure of traffic safety has been the number of accidents per mile driven. If this method is used, it appears that elderly drivers have a below-average safety record (*4*). Given the projected increase in the number of elderly drivers and the corresponding increase in the percentage of total miles driven by elderly drivers, this is cause for concern.

Studies have been conducted by others who sought to determine where and why elderly drivers are a high-risk group. In one study it was found that the young driver, defined as a driver between ages 17 and 19, is more than 3 times as likely to have an accident as the average driver, and that the elderly driver, defined as a driver of age 65 or greater, is slightly less likely to have an accident than the average driver. However, it was noted that the relative accident rates for drivers between 30 and 64 were substantially lower than the average, which indicated a decrease in safety for elderly drivers (*5*). In another study the risks of being in an injury or fatal accident were compared for the 25–64 age group and the over-65 age group. The over-65 age group was found to be 2.5 to 5 times as likely to be involved in a fatal auto accident and 1 to 2 times as likely to be involved in an injury accident (*6*). Finally, it has been reported that older drivers tended to be responsible for accidents more often than younger drivers (*7*). The accident exposure of drivers in various age groups has often been determined through surveys or has been assumed to be proportional to the number of licensed drivers in each age group. An alternative method for analyzing accident frequency is the induced exposure method, which has been used by others (*8–14*).

The variation of the induced exposure method used in this research is based upon the assumption that the accident exposure by any class of driver or vehicle is directly proportional to the number of "innocent victim" involvements in multi-automobile accidents by that class of driver or vehicle (*15*). Innocent victim involvement is defined as involvement in an accident in which the driver (the "innocent victim") was not responsible.

RESEARCH DATA BASE

Data Sources

To examine the relationship between driver age and accidents, use was made of the accident records mentioned previously for 1983–1985, as well as three other highway data files maintained by the Division of Traffic and Safety at the Michigan Department of Transportation. These were the Michigan Dimensional Accident Surveillance (MIDAS) Geometric Segment File, the MIDAS Traffic Volume File, and the Traffic Signal Inventory File.

The geometric file contains information about the geometric characteristics of each unlimited access highway segment on the state trunkline system, such as number of lanes, lane width, roadside development, posted speed limit, curvature, and other elements of the horizontal alignment of the segment. The file also contains information about the existence of an intersection, the type of intersection, and its characteristics.

The traffic volume file contains information about the capacity, average daily traffic, and hourly traffic volume distribution for the counting stations for the state trunkline system.

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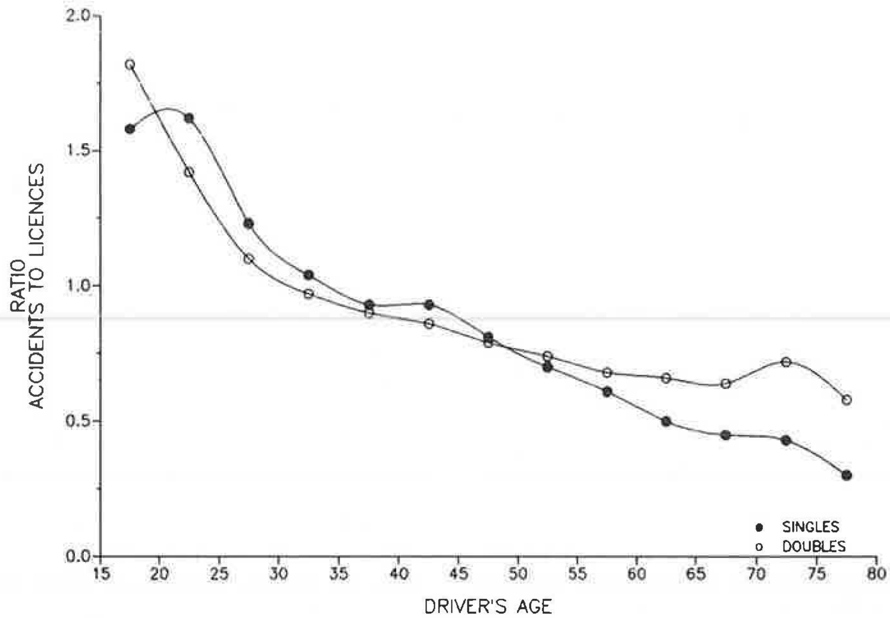

FIGURE 1 Comparison of the percentage of accidents per licensed driver for single- and two-vehicle accidents.

The traffic signal file contains information about the existence of traffic signals or other traffic control devices at intersections. The records contain information about the type of signal, phases, prohibitions of left or right turns, and other characteristics of the signal lantern, such as the manufacturer, material, lens size, and year of installation.

Each of the four data files uses a common system for identifying of highway segments. This factor allowed the research team to merge the accident records with the files describing the characteristics of the locations where the accidents occurred.

Subdivision of the Data for Analysis

After the project data file had been created, 28 separate subsets of data were created for analysis. These subsets were created by subdividing both intersection and nonintersection accidents in the project file into accidents that occurred on the Interstate highway system and accidents that occurred on federal, state, and other highway systems in the state. Those accidents that occurred on the Interstate highway system were then subdivided into accidents that occurred during the day and those that occurred at night. The accidents that occurred on the non-Interstate system were subdivided on the basis of the time of the accident, type of route (i.e., federal, state, or other) on which the accident occurred, and overall surrounding development of the area (i.e., urban or rural area) in which the accident occurred. The partitioning of the project data file is presented in Table 1.

The data were subdivided in this manner because the subdivisions allowed differences in the distribution of the accidents between two given subsets in which all the factors were the same except for one to be explained and attributed to the presence of the noncommon factor. Analyses showed that when the distributions of two subsets were compared and found statistically different, the difference could only be explained by the factor that was not the same in these two subsets. This type of approach thus leads to an identification of the factor that is affecting the accident distributions.

Driver 1–Driver 2 Validation

The basic hypothesis of this research is that a better representation of the driving frequency of drivers of each age group is their exposure to accidents, as represented by their involvement as innocent victims in an accident. Data that show the accident frequency distribution of Driver 2 (the innocent victim) as a function of age are considered in this research to be a measure of accident exposure and therefore also of driving frequency.

The basic interpretation of the meaning of Driver 1 and Driver 2 in the accident data base is fundamental to this hyphothesis. In Michigan, instructions on the accident reporting form indicate that Driver 1 is the driver of the vehicle that is responsible for the accident and Driver 2 is the driver of the vehicle that is not responsible for the accident. Additional data included in the accident record indicate any hazardous actions in which either driver may have been involved that were deemed by the investigating officer to contribute to the accident. In a discussion of a paper by Carr (*9*), in which a similar methodology was used, the impact of bias introduced by a tendency to assign responsibility to certain driver age groups or in situations in which both drivers or neither driver was responsible was addressed. Attempts were therefore made in the current research to minimize these types of bias by classifying a driver as responsible only when that driver was also cited for a hazardous action.

In an earlier study that used these accident records for the relative involvement of vehicles of different characteristics in accidents (*15*), it was found that when the vehicle identification number (VIN) shown on the accident record was used to derive vehicle characteristics from the VNDCTR program, the VINs of up to 42 percent of the vehicles involved in accidents did not

TABLE 1 SUBSETS OF PROJECT DATA FILE

Data Set	Contents
Interstate Accident Data Subsets	
Interchange	
1	Day
2	Night
Noninterchange	
3	Day
4	Night
Non-Interstate Accident Data Subsets	
Intersection	
5	Day, rural, federal
6	Day, urban, federal
7	Night, rural, federal
8	Night, urban, federal
9	Day, rural, state
10	Day, urban, state
11	Night, rural, state
12	Night, urban, state
13	Day, rural, other
14	Day, urban, other
15	Night, rural, other
16	Night, urban, other
Nonintersection	
17	Day, rural, federal
18	Day, urban, federal
19	Night, rural, federal
20	Night, urban, federal
21	Day, rural, state
22	Day, urban, state
23	Night, rural, state
24	Night, urban, state
25	Day, rural, other
26	Day, urban, other
27	Night, rural, other
28	Night, urban, other

decode properly. This problem occurred either because of errors made in recording this information by the investigating officer at the scene of the accident or errors made in transcribing this information from the original accident form to the accident data base.

Errors of this magnitude could dramatically affect the size of the accident samples used in this study and the statistical reliability of the results, particularly for age groups in which smaller samples exist (such as older drivers). Therefore an analysis was undertaken to verify the accuracy of the information recorded on the accident data base for Driver 1 and Driver 2, as indicated in the accident data base. If such errors did exist, it was important to determine if these errors were randomly distributed with respect to the age of the driver. Because the accident records contain both an indication of driver responsibility through the Driver 1 or Driver 2 notation and an indication of any hazardous action performed by each driver, the original data records were examined for a correspondence between the Driver 1 and Driver 2 notations and the indication of which driver committed a hazardous action.

The results for the case of all multivehicle accidents on non-Interstate highways, shown in Figure 2, are typical of the findings from this analysis. In this figure, three curves are plotted. One curve shows the Driver 1 distribution, with all accidents (a total of 193,102) included as indicated in the original data base. The second curve shows the Driver 1 distribution, with only those accidents (154,854) for the original data base in which the indicated Driver 1 was cited as having performed a contributing hazardous action and the indicated Driver 2 was not cited as having performed such an action. This process eliminated 19.8 percent of the accidents from the original data base (plotted in the first curve). The accidents that were eliminated were those in which neither driver was cited for a hazardous action, both drivers were cited for a hazardous action, or the wrong driver was cited for a hazardous action. Finally, the third curve shows the Driver 1 distribution based on the second curve, with the addition of the Driver 1 accidents in which the original data base had the indicated Driver 1 and Driver 2 reversed on the basis of the hazardous action notation. This final curve is based on 190,922 accidents. Therefore this process resulted in the elimination of only 2,180 accidents from the original data base, or roughly a little over 1 percent of the accidents.

A visual examination of this figure indicates that the curves are virtually the same, and statistical analyses verified that the errors were randomly distributed across all age groups. This latter result is significant because it indicates that through the process of examining a hazardous action citation for the drivers involved in an accident, any bias related to age in the designation of the driver responsible on the original accident record can be virtually eliminated. As a result of this analysis, it was apparent that accuracy of the original data base could be enhanced through the comparison and deletion process without the elimination of a significant number of accident records. In the worst case, this process eliminated about 19 percent of the accidents, but it also improved the accuracy of the remaining 81 percent of the accidents in the data base. In this case, it was found that the errors were statistically random at the 95 percent confidence level.

RESEARCH METHODOLOGY

This research is based on the assumption that the probability that a driver in a given age group will be involved as the driver responsible in an accident is represented by the percentage of accidents in which the drivers in that age group were involved and were the driver responsible for the accident. As stated earlier, this driver is defined as Driver 1. Furthermore, the probability that a driver in a given age group will be exposed to an accident in which the driver was not responsible is represented by the percentage of accidents in which the drivers in that age group were involved but were not the driver responsible for the accident. This driver is defined as Driver 2, or the so-called "innocent victim."

This interpretation of Driver 2 is meant to be a measure for accident exposure in this study. This induced exposure measure provides a mechanism by which accident frequency for drivers of different age groups may be studied under a variety of driving conditions, such as on different type of roads, during day and night conditions, under urban and rural conditions, and so on. If it is true that accident involvement as an innocent victim increases in direct proportion to accident exposure, then this surrogate measure should be a reliable indicator of such a phenomenon.

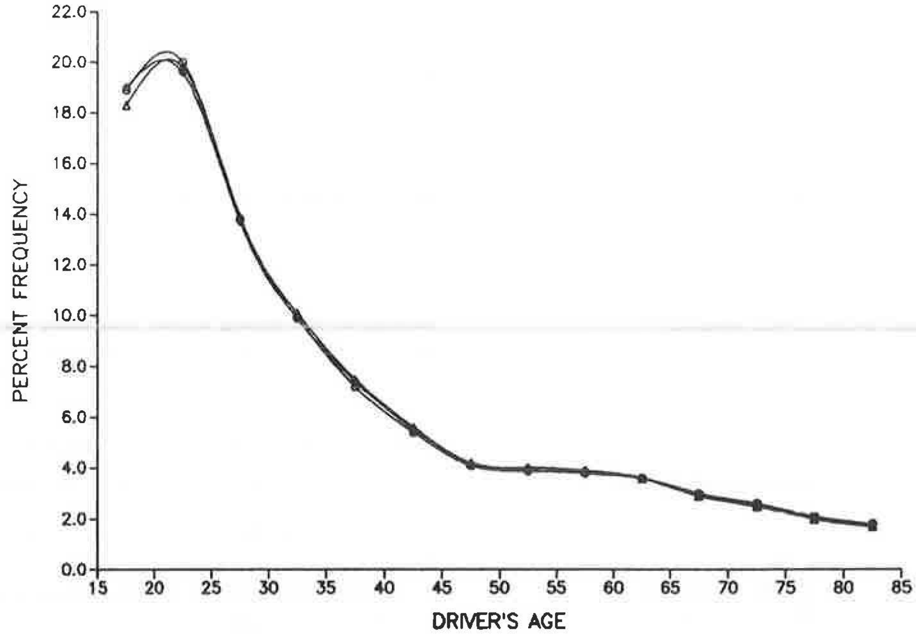

FIGURE 2 Driver 1 distributions for original and modified data sets for all multivehicle accidents on non-Interstate highways.

It should be noted that the interpretations of Driver 1 and Driver 2 in this research are similar to the interpretations made in research performed by others, in which involvement in single-vehicle accidents and involvement in multiple-vehicle accidents were used to derive both driver responsibility and driver exposure. A comparison of various formulations of induced exposure as represented by Thorpe (*8*) and Haight (*10, 11*) with the exposure measure used in this research is given in Figure 3. It can be observed that when the Driver 2 distribution is compared with Thorpe's formulation (*8*), the same exposure trend with driver age occurs, but there are large differences between the two formulations. These differences average about 27 percent across all age groups but are considerably higher for both the younger and the older age groups. Similarly, a comparison with Haight's original formulation (*10*) and his modified formulation (*11*) yields average errors of about 26 percent and 9 percent, respectively. Again, the greatest differences occur with the younger and older age groups.

The assumptions made by Haight and Thorpe in their formulation of induced exposure from the double accident data are considerably relaxed in our formulation of exposure. That is, for each pair of drivers involved in an accident, the designation of the responsible driver and innocent driver is made from the original accident report. The assumption is that the Driver 1

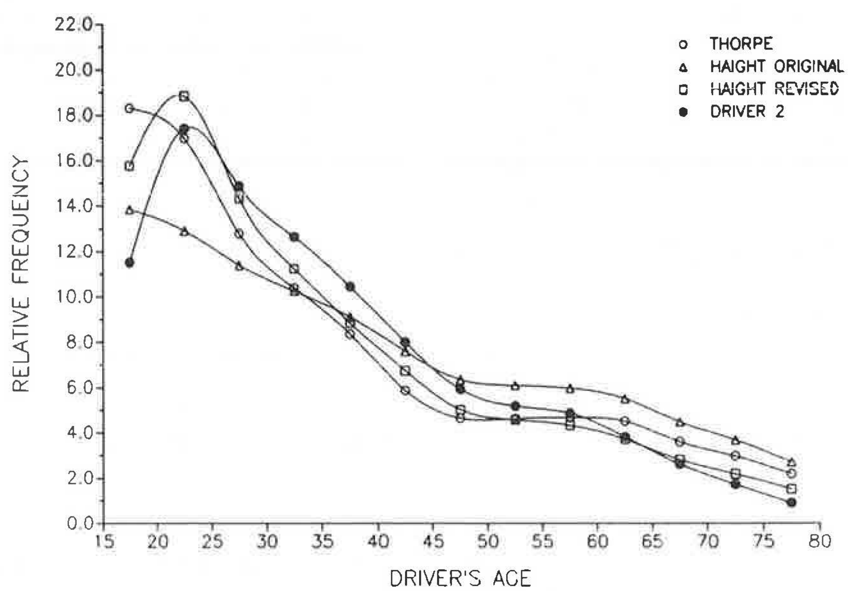

FIGURE 3 Comparisons of various induced exposure means.

distribution in multiple-vehicle accidents is a measure of responsibility and that the Driver 2 distribution in multiple-vehicle accidents is a measure of exposure. For this reason it is expected that the Driver 2 formulation of exposure is considerably more reliable than the formulations of Thorpe and Haight.

Preliminary tests have also been conducted on the accident data base, using the induced responsibility model (*16*) proposed by Wasielewski and Evans. Their formulation seeks to determine the total number of accidents that occurred in which both drivers were responsible on the basis of the distribution of single and double accidents and licensed drivers. These distributions are shown in Figure 4 for all accidents occurring in Michigan during 1983–1985. By applying the induced responsibility model, it was found that 10.3 percent of the accidents were the responsibility of both drivers. The results of comparing the actual double accidents with those derived from the Wasielewski and Evans formulation with a responsibility factor of 10.3 percent are shown in Figure 5. Because only about 10 percent of the accidents involve shared responsibility, the Driver 2 measure of exposure should be reliable.

It should be emphasized at the outset that the measure of exposure that is used here is in fact deduced from accident data and is therefore subject to some degree of error. However, in the absence of a definitive measure of actual exposure that would normally be obtainable only through extensive driver surveys, it may only be possible to deduce the validity of this research methodology by similar comparisons. The results of research undertaken using this measure of induced exposure should therefore be carefully interpreted as indications of likely trends rather than absolute facts about accident involvement.

The probability distributions for both Driver 1 and Driver 2 as a function of age group for all non-interchange multivehicle accidents on Interstate highways in Michigan are shown in Figure 6. If the interpretation of Driver 1 and Driver 2 noted previously is used, the Driver 1 curve on this figure shows the percentage of drivers in different age groups that were the driver responsible in these accidents. The curve indicates that drivers in the 20–25 age group are responsible more often than drivers in any other age group. This curve also indicates a decreasing probability of being involved in an accident and being responsible for that accident with increasing age. This curve is typical of the results of research that bases accident rates on other bases, such as the number of licensed drivers in each age group.

However, it is the interpretation and use of the data plotted in the second curve on Figure 6 that are unique in this research. This curve indicates the percentage of drivers in different age groups that were involved as innocent victims in these accidents. This curve is similar to those plotted by other researchers who used the induced responsibility model (*16*). The curve indicates that drivers in the 20–25 age group have the highest probability of being involved in such accidents as innocent victims and also that the probability of being involved in such accidents as an innocent victim decreases with age. If the results of the Driver 2 curve are an accurate measure of driver exposure to accidents, however, then the accident involvement as a responsible driver is higher than the accident exposure when the Driver 1 curve is above the Driver 2 curve. Conversely, when the Driver 1 curve is below the Driver 2 curve, accident involvement as a responsible driver is less than the accident exposure.

A better indication of the meaning of these statements may be found by examining the results presented in Figure 7. In this figure, the Driver 1 percentage is divided by the Driver 2 percentage. The result, called the relative accident involvement ratio, is plotted for each age group. As can be seen, this ratio is highest for the younger age groups, decreases until about age 40, remains relatively constant from age 45 to about age 60, and then begins to increase again. These results indicate that there is an overinvolvement in such highway accidents by drivers who are less than 30 years old and drivers who are more

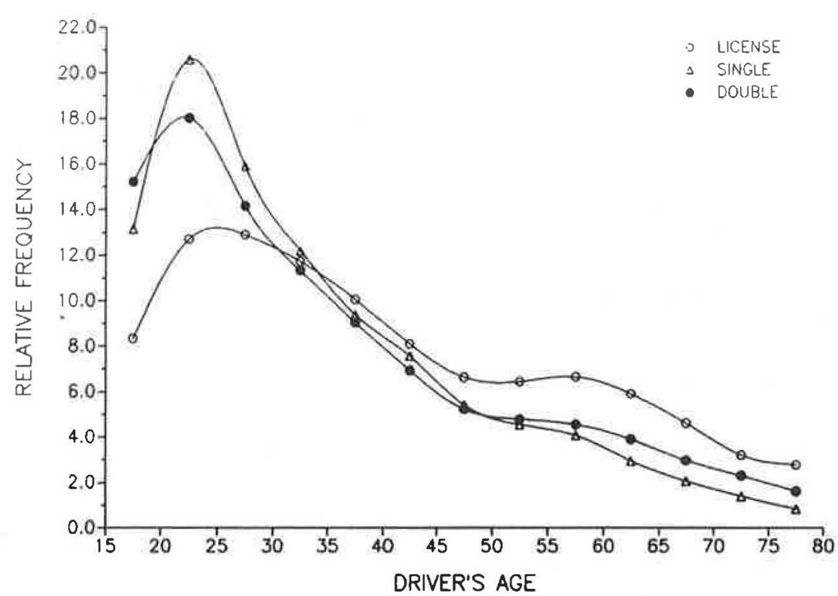

FIGURE 4 Distribution of single- and two-vehicle accidents and licensed drivers for 1983–1985 in Michigan.

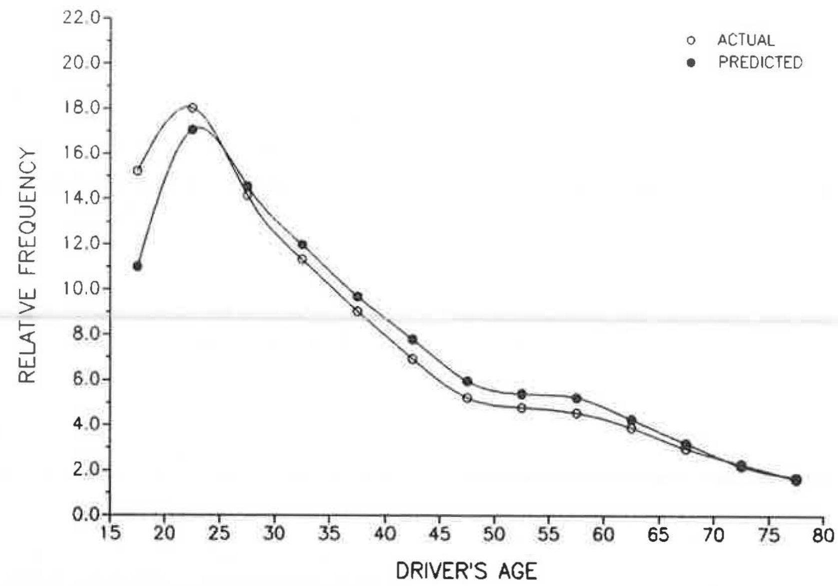

FIGURE 5 Comparison of actual and predicted two-vehicle accidents based on Wasielewski and Evans' formulation with dual responsibility of 10.3 percent.

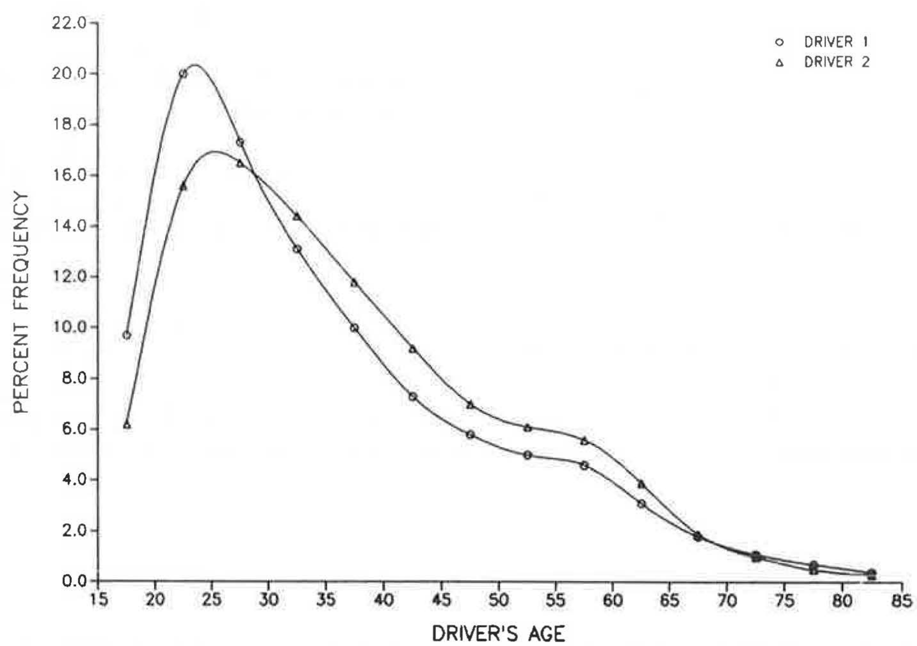

FIGURE 6 Driver distributions for noninterchange multivehicle accidents on Interstate highways.

than 70, because the relative accident involvement ratio is greater than 1.0 for such drivers.

These data give an indication of the relative frequency of accident involvement for drivers in the different age groups and are therefore useful for comparative purposes. When the relative accident involvement ratio is equal to 1.0, the accident involvement for the driver responsible is equal to the accident exposure for drivers in the same age bracket. If this ratio is greater than 1.0, the driver is more likely to be responsible when involved in an accident; that is, the driver is overinvolved in accidents. If the ratio is less than 1.0, the driver is less likely to be responsible when involved in an accident; that is, the driver is underinvolved in accidents.

The results displayed in Figure 7 indicate that young drivers (less than age 30) and older drivers (more than age 70) are more likely to be involved as responsible drivers in a noninterchange, multivehicle accident on an Interstate highway. The results also indicate that accident involvement decreases with increasing age up to about age 40, remains relatively constant from age 40 to age 60, and begins to increase with increasing age thereafter.

RESEARCH RESULTS

Several of the findings in this research study will now be presented for comparative purposes. An examination of the

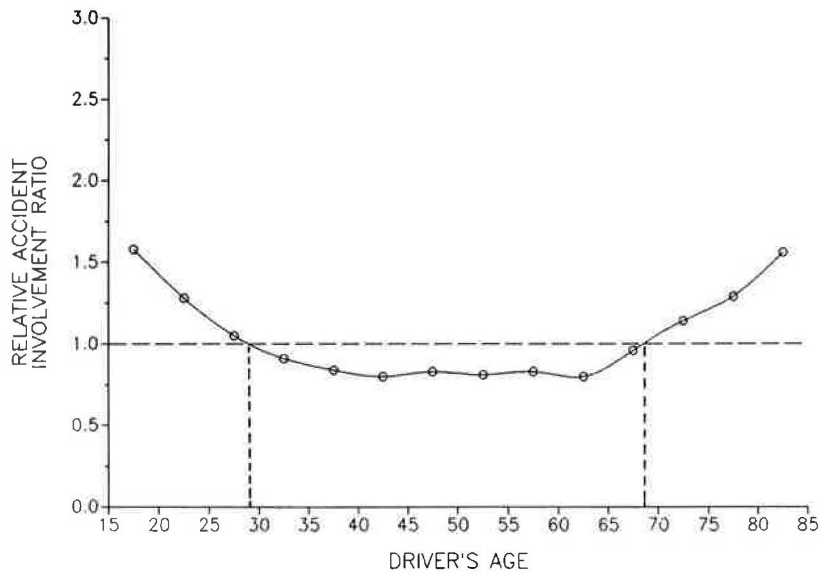

FIGURE 7 Relative accident involvement ratios for noninterchange multivehicle accidents on Interstate highways.

relative accident involvement of both male and female drivers for all multivehicle accidents in calendar years 1983 through 1985 was undertaken. Overall, men are involved in about 2.5 to 3 times as many accidents as women. However, as can be observed in Figure 8, there is a fairly good correspondence between the relative accident involvement of men and women up to about age 50. Beyond age 50, however, the relative accident involvement of female drivers is considerably higher than that of male drivers. These results would seem to suggest that both older men and older women are more likely to be involved in accidents after age 50 than they were in earlier years and that this increase in relative involvement for females is greater for women than for men.

A comparison was made for all multivehicle accidents on Interstate and non-Interstate routes in Michigan. One reason for making such a comparison is to consider the possible impact of the difference in highway design features of the two different classes of highway facilities. As shown in Figure 9, there is a higher accident involvement for older drivers on non-Interstate route than Interstate routes. The reasons for this are not clear at this point, but it is possible that the influence of highway design features is being observed. Additional research is being

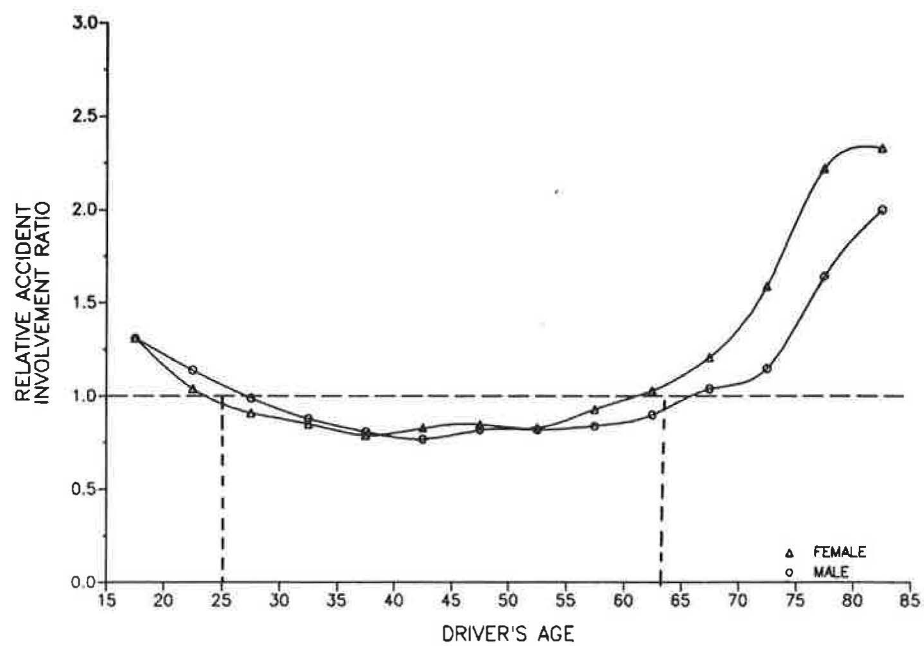

FIGURE 8 Relative accident involvement ratios for male and female drivers for all multivehicle accidents, 1983–1985.

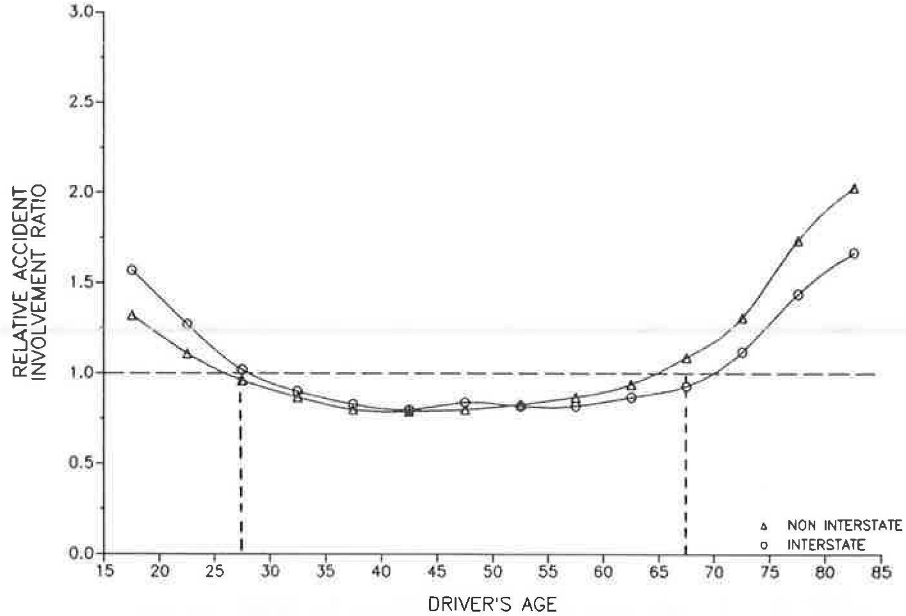

FIGURE 9 Relative accident involvement ratios for multivehicle accidents on Interstate and non-Interstate highways.

performed to identify the apparent increase in accident involvement for older drivers on non-Interstate highways.

Further studies of specific highway design features, such as horizontal curvature, grades, and number of lanes, are being tested in subsequent research. However, at this point it is clear that after age 55 the relative accident involvement of drivers increases on both types of facilities.

An examination of all nonintersection multivehicle accidents on non-Interstate highways in Michigan was also performed. As can be observed in Figure 10, the relative accident involvement on each of these facilities increased dramatically after beyond age 55, and this exposure was significantly higher than that for younger drivers. It may also be seen that the relative accident involvement for older drivers on Michigan state trunkline highways is lower than on U.S. routes and on other highways. The relative accident involvement for older drivers on highways other than Michigan state trunkline and U.S. routes is the highest. These highways include, for example, county roads, which typically have design standards less stringent than either Michigan state trunkline or U.S. routes. This suggests

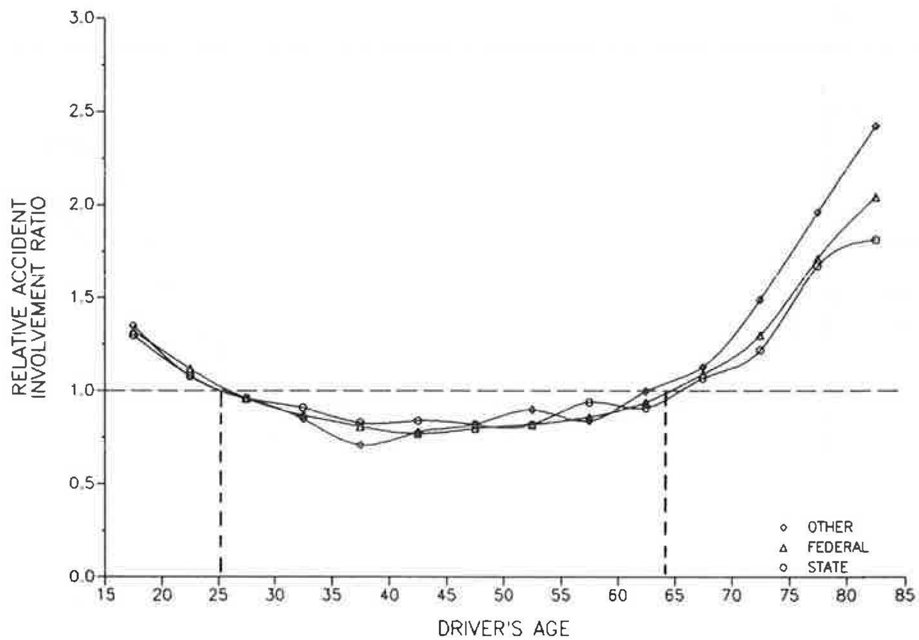

FIGURE 10 Relative accident involvement ratios for multivehicle accidents on non-Interstate highways.

that certain highway design features—perhaps alignment or lane width—may have a more significantly adverse effect on the accident potential for older drivers in comparison to other drivers.

The impact of light conditions, as indicated by whether the accident occurred during day or night, is shown in Figure 11. Although some instability exists toward the higher-age driver range, it appears that the relative accident involvement of older drivers at night is somewhat higher than during the day. The differences between day and night conditions for other drivers is not as significant. This may suggest that the reduced lighting conditions inherent in night driving require better signing or marking of roadways on non-Interstate highways to reduce the accident involvement of older drivers.

The accident involvement of drivers at intersections was also studied and the results are displayed in Figure 12. As can be observed, older drivers tend to be more heavily involved in these types of accidents than other drivers, and this involvement is markedly higher than for young drivers. This could indicate that there may be some diminished physical or mental capability, perhaps reaction time, that affects the abilities of older drivers in such locations. Furthermore, the data on this

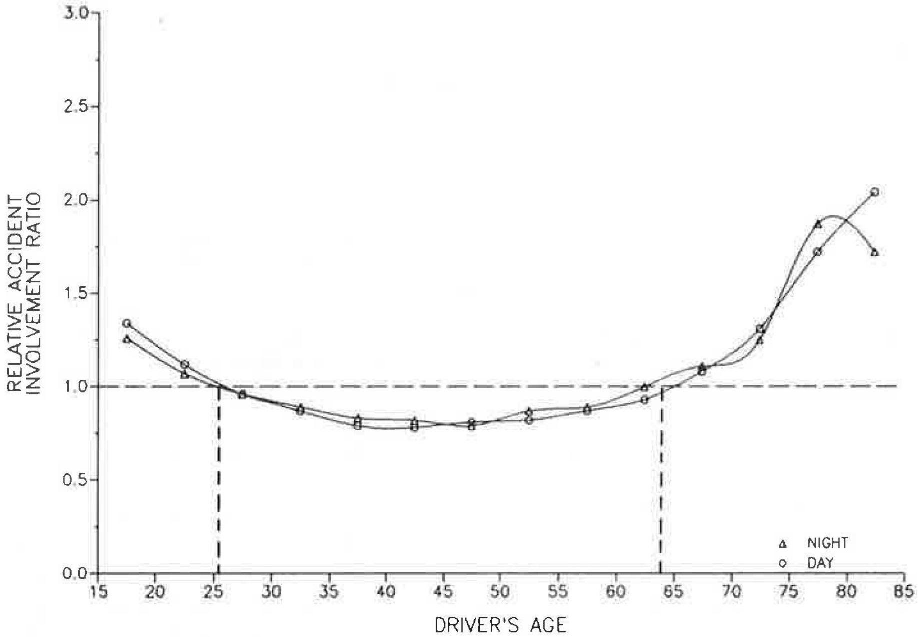

FIGURE 11 Relative accident involvement ratios for multivehicle accidents on non-Interstate routes during day and night.

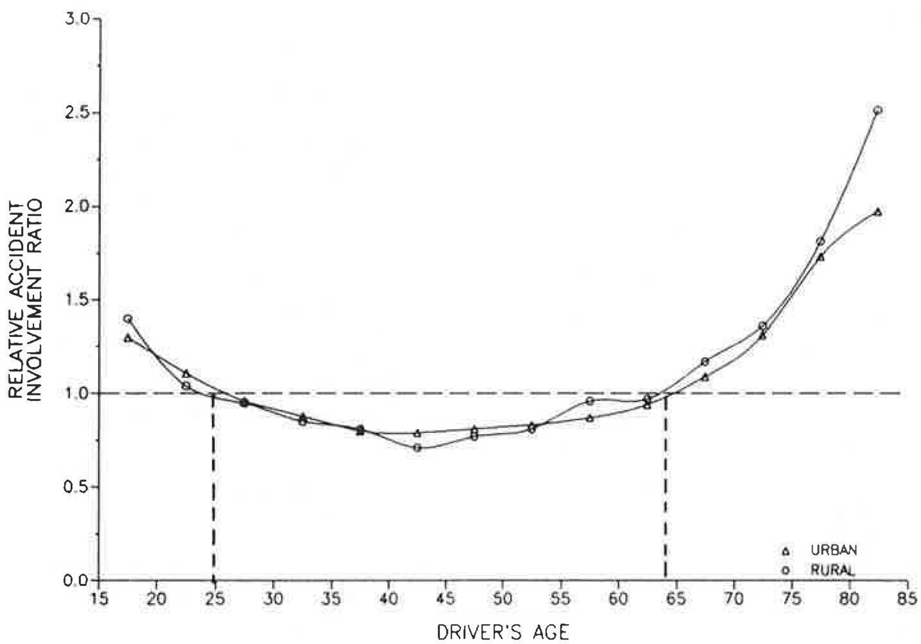

FIGURE 12 Relative accident involvement ratios for multivehicle accidents at intersections on non-Interstate routes in urban and rural areas.

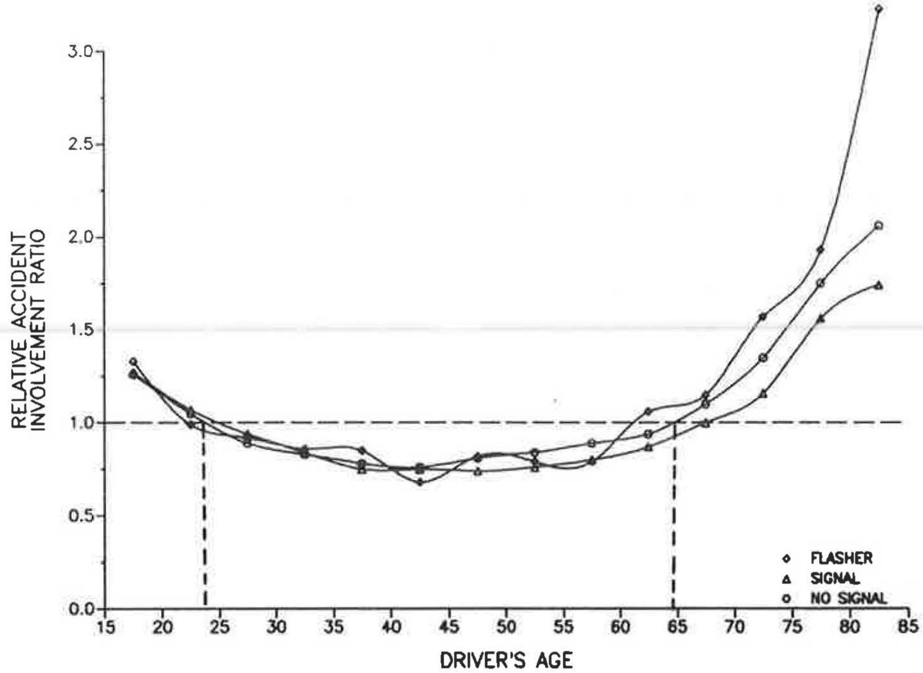

FIGURE 13 Relative accident involvement ratios for multivehicle accidents at signalized and nonsignalized intersections on non-Interstate routes.

figure indicate that older drivers appear to have a greater accident risk at intersections located in rural areas. A more detailed analysis of the accident history of drivers at intersections was also undertaken to examine relative exposure at nonsignalized and signalized intersections. The signalized intersections were classified into two groups, those operating on signal cycles and those operating with a flashing indicator. The results shown in Figure 13 indicate that flashing signals present considerable difficulty for older drivers, whereas signals operating on cycles are the least hazardous for these drivers. These data seem to suggest that there may be a particular problem with older drivers in terms of either the interpretation of or the reaction to flashing signals.

SUMMARY

The research has shown that because of Michigan's unique ability to combine accident records with the physical characteristics of the highway locations where these accidents occurred, indications of the relative involvement of drivers of different ages in accidents may be determined. Furthermore, by configuring the accident data into subsets on the basis of the highway facility characteristics, time of the accident, and measures of roadside development, isolation of the factors that seem to cause the variations in accident involvement by drivers of different ages can be obtained. This research has shown that older drivers have higher accident involvement rates than other drivers and that in many cases their accident involvement rate approaches or exceeds that of young drivers. Subsequent research is planned to identify measures that might be undertaken to improve the observed adverse involvement trends for older drivers.

ACKNOWLEDGMENTS

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Injury Accident Prediction Models for Signalized Intersections

MICHAEL YIU-KUEN LAU AND ADOLF D. MAY, JR.

An intuitive methodology for developing accident prediction models for signalized intersections based on the Traffic Accident Surveillance and Analysis System (TASAS) in California is illustrated. A fairly new grouping and classifying technique called Classification and Regression Trees (CART) was used as a building block for developing injury accident models. The proposed methodology is a three-level procedure with a "tree" structure for easy interpretation and application. Macroscopic-type models for injury accidents per year are derived, and the following factors have been found to be significant: traffic intensity, proportion of cross street traffic, intersection type, signal type, number of lanes on cross streets and main streets, and left turn arrangements. On the basis of the results, it is also apparent that the models derived from the proposed methodology and TASAS provide more intuition and flexibility than the existing models used in California and other models derived from both site observations and accident record systems.

The purpose of this paper, which is based on a project funded by the California Department of Transportation (Caltrans) and the FHWA, is to develop accident prediction models for signalized intersections on the basis of intersection characteristics such as geometric design elements, traffic control measures, traffic demand patterns, environmental factors, and accident history. The data base for the study was derived from the Traffic Accident Surveillance and Analysis System (TASAS) maintained by Caltrans and the California State Highway Patrol (*1*). One of the themes of study is the investigation of whether prediction models can be developed on the basis of existing accident record systems. Major investments of time and effort have been made in these systems, and they also require continual efforts by state and federal agencies for their maintenance.

Because it was not the aim of the study to collect additional data for model development in addition to the information contained in TASAS, it was assumed that macroscopic models that use existing data would suffice. The term "macroscopic models" refers to models that are not derived on the basis of detailed information and do not require such information, including turning movement counts, headway distributions, and so forth—details that are not usually available in accident record systems. Furthermore, classification of accidents for this kind of model is confined to injury, fatal, and property damage only (PDO) accidents. The advantages include easy comprehension and simple translation into monetary terms, which are required by most economic and feasibility analyses. Disadvantages include inadequacy in reflecting the process of collisions and the concept of conflicts.

The issue of systematically grouping entities (such as intersections) with similar accident patterns on the basis of different control, design, and demand features of the entities has not been examined critically in previous studies. A powerful and systematic tool for grouping or classifying entities can be a very important building block in developing accident prediction models because lack of homogeneity within subgroups classified by nonsystematic methods could introduce biases in prediction. A fairly new grouping and classifying technique called Classification and Regression Trees (CART), developed by Breiman at the University of California, Berkeley (*2*), was used in the study for this purpose.

This paper describes a methodology and specific techniques for developing prediction models for injury accidents. The proposed methodology is based on the results and comments of the pilot study, working paper, and review process, as described elsewhere (*3*). Injury accidents here include accidents in which the people involved have injuries ranging from very slight to serious. It is believed that the reporting level for injury accidents is about 80–90 percent, whereas the reporting level for PDO accidents is believed to be about 50 percent or less. There is also a difference in reporting levels among different jurisdictions. In contrast, fatal accidents are rare events, and only 0.5 percent of all intersection accidents are fatal. In view of these facts, models for injury accidents were developed as a first step for the project. The application of our three-level prediction procedure to some 2,500 signalized intersections that are under the control of the state of California and contained in TASAS is illustrated.

DATA BASE: TASAS

The data base used in the study is basically a simplified version of the TASAS system, containing information on about 2,500 signalized intersections and 122,000 accidents that occurred at these intersections from 1979 through 1985. Some of the intersections have shorter reporting periods. This is a reflection of changes in design, control, and so on.

Intersection-Related Characteristics

There are 2,498 signalized intersections in the data base, 95 percent of which are located in urban areas. Some of their characteristics are briefly discussed in the following paragraphs. Detailed information may be found in Table 1 and elsewhere (*3*).

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TABLE 1 FACTORS, LEVELS, AND PERCENTAGES OF INTERSECTION CHARACTERISTICS

Factor	Percentage	Levels (Notes)
Terrain	0.9	1 (mountainous)
	17.7	2 (rolling)
	81.4	3 (flat)
Design speed (mph) (SPEED)	5.0	1 (less than 30)
	1.4	2 (30–34)
	8.9	3 (35–39)
	5.3	4 (40–44)
	13.1	5 (45–49)
	12.0	6 (50–54)
	5.6	7 (55–59)
	26.8	8 (60–64)
	21.8	9 (greater than 65)
Rural/urban (RORU)	4.4	1 (rural)
	95.6	2 (urban)
Inside/outside city (IORO)	87.0	1 (inside)
	13.0	2 (outside)
Intersection type (ITYPE)	72.5	1 (four-legged)
	2.8	2 [multilegged (>4)]
	3.9	3 (offset)
	18.2	4 ("T")
	1.0	5 ("Y")
	1.5	6 (other)
Control type (CTYPE)	22.7	1 (pretimed two-phase)
	4.2	2 (pretimed multiphase)
	10.2	3 (semi–traffic actuated two-phase)
	7.2	4 (semi–traffic actuated multiphase)
	11.1	5 (full traffic actuated two-phase)
	44.6	6 (full traffic actuated multiphase)
Lighting type (LIGHT)	0.8	1 (no lighting)
	98.8	2 (lighted)
Main-line signal mast arm (MSM)	8.3	1 (no mast arm)
	91.2	2 (signal on mast arm)
	0.5	3 (missing)
Main-line left turn channelization (MLT)	42.3	1 (curbed median left turn channelization)
	20.2	2 (no left turn channelization)
	36.5	3 (painted left turn channelization)
	0.6	4 (raised bars)
	0.4	5 (missing)
Main-line right turn channelization (MRT)	80.1	1 (no free right turns)
	19.7	2 (provision for free right turns)
	0.2	3 (missing)
Main-line traffic flow (MTF)	4.0	1 (two-way traffic, no left turns permitted)
	89.0	2 (two-way traffic, left turns permitted)
	0.8	3 (two-way traffic, left turns restricted during peak hours)
	5.2	4 (one-way traffic)
	1.0	5 (other)
Main-line number of lanes (MNL)	9.2	2 (two lanes)
	3.9	3 (three lanes)
	66.9	4 (four lanes)
	2.2	5 (five lanes)
	16.5	6 (six lanes)
	0.2	7 (seven lanes)
	0.7	8 (eight lanes)
Main-line ADT (MADT)		(packed numeric 999999)
Cross street signal mast arm (XSM)	46.3	1 (no mast arm)
	53.2	2 (signal on mast arm)
	0.5	3 (missing)
Cross street left turn channelization (XLT)	13.8	1 (curbed median left turn channelization)
	58.7	2 (no left turn channelization)
	26.7	3 (painted left turn channelization)
	0.2	4 (raised bars)
	0.6	5 (missing)
Cross street right turn channelization (XRT)	78.0	1 (no free right turns)
	21.6	2 (provision for free right turns)
	0.4	3 (missing)
Cross street traffic flow (XTF)	2.2	1 (two-way traffic, no left turn permitted)
	90.5	2 (two-way traffic, left turn permitted)
	0.2	3 (two-way traffic, left turn restricted during peak hours)
	6.4	4 (one-way traffic)
	0.7	5 (other)
Cross street number of lanes (XNL)	57.4	2 (two lanes)
	6.0	3 (three lanes)
	32.1	4 (four lanes)
	0.7	5 (five lanes)
	2.3	6 (six lanes)
	0.0	7 (seven lanes)
	0.1	8 (eight lanes)
Cross street ADT (XADT)		(packed numeric 999999)
Median indicator	23.4	1 (undivided)
	76.2	2 (divided or independent) (alignment)
IADT		[= XADT/(XADT + MADT)]

There are ~72 percent four-legged and ~18 percent "T" intersections. The remainder are multilegged, offset, or "Y" intersections. Two thirds (~66 percent) of the intersections have four lanes on main streets, and ~57 percent have only two lanes on side streets. For left turn arrangements on main streets, 42 percent have curb median left turn channelization, and 36 have painted left turn channelization. On side streets, 14 percent have curb median left turn channelization, and 26 percent have painted left turn channelization. About half have design speeds of less than 55 mph. Some 44 percent of the intersections are controlled by multiphase, fully actuated traffic controllers, and 22 percent are controlled by two-phase, pretimed traffic controllers. About 10 percent have two-phase, semi-traffic-actuated controllers, and 11 percent have two-phase, fully actuated controllers. On main streets, 89 percent have two-way traffic with left turn permitted, and 4 percent have two-way traffic with no left turns permitted. About 5 percent have one-way traffic. The percentages for side streets are 90 percent two-way with left turn, 2 percent two-way without left turn, and 6 percent one-way. The median average daily traffic (ADT) on main streets is ~27,000, and the maximum is ~72,000. The median ADT on side streets is ~17,500, and the highest ADT is ~45,000.

Accident-Related Characteristics

There are ~122,000 accident records in the data base. These are accidents that occurred within 250 ft of the 2,498 intersections, from 1979 to 1985.

The majority of the accidents (76 percent) occurred in clear weather conditions, and only 12 percent of them occurred in cloudy conditions and 8 percent of them in rainy conditions. It was also recorded that 35 percent of the accidents were rear end collisions and 31 percent were broadside collisions. Only 2.4 percent of them were automobile-pedestrian collisions. The percentage of fatal accidents with one or more people killed was only ~0.5, and the percentage of injury accidents was ~38. The remaining were PDO accidents. About 75 percent of the accident reports noted that the parties involved had not been drinking, and only ~5 percent of them reported that those who were involved were under the influence of alcohol.

METHODOLOGY

Methodology is a systematic study of the subject in question, including such tasks as definition of objectives, selection of measures of effectiveness, generation of solutions, refinement of solutions, selection of models, and so on. A flowchart showing the proposed methodology used here, based on this principle, is shown in Figure 1, and the details of the procedure are discussed in the sections that follow.

The study of accidents has been regarded as a controversial and indefinite field by many researchers because occurrences of accidents are highly stochastic in nature. Because of this, a fairly long record of accident history would be required for any meaningful study. A long accident history also has its disadvantages. The question of changes in both basic and operating characteristics of entities and in methods of collecting and recording accidents during the period must be addressed. Furthermore, accident data and records are usually regarded as "incomplete" because useful information for model building, such as vehicle condition and design, driver's characteristics and behavior, and so on, is usually not contained in such systems.

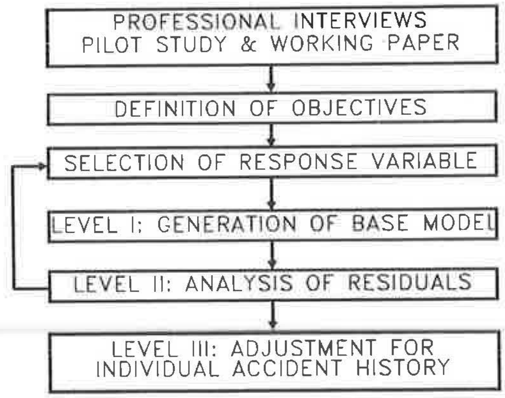


FIGURE 1 The proposed methodology.

Approach Objectives

Because of the problems mentioned previously, the proposed three-level estimating procedure has been derived with the following objectives in mind:

- To develop an approach that is intuitive and yet is reflective of the stochastic nature of accident occurrences;
- To develop a staged approach that can allow users to appreciate the importance and consequences of the process, in contrast to some "black box" approaches; and
- To develop a systematic approach that is capable of extracting some patterns from the fairly "incomplete" data sets in most accident record systems.

Selection of Response Variable

Selection of a response variable is a very important step in the process because this variable determines, to a large extent, the final model. The selection of the preferred model may hinge on the choice of the response variable. This variable, which is also commonly known as the dependent variable, is a measure of performance of the system, for example, the risk level of an intersection.

In this paper, only injury accidents are addressed because of the different reporting levels of PDO accidents and rarity of fatal accidents. The first task is to find an appropriate derivative of injury accidents for comparison and evaluation purposes because common sense indicates that it is not reasonable to compare an intersection with 1 accident in 1 year with another intersection with 1 accident in 10 years. For this purpose, normalization by time is a logical step. The next matter is a further normalization by exposure measures such as traffic intensity or traffic intensity-distance. This is a common practice; however, simple reasoning indicates that the attitude of "why bother" should prevail, if possible. As a result, the number of injury accidents per year was initially selected as our response variable. It was found that no further transformation was required because this response variable appeared to be adequate.

Generation of the Base Model

Instead of putting some form of the traffic intensity variable in the denominator of the response variable, a base model was built, with injury accidents per year as the response variable and traffic intensity as the predictor variable. One of the advantages of this approach is that it allows researchers to see the relationship between the two variables in an undistorted manner, such as a scatter plot. On the basis of the untransformed information in a graph, different functional forms to model the relationship between the two variables can be tried. Estimates of the parameters can be obtained by techniques such as least squares, maximum likelihood, and so on. The base model so obtained is called a Level I prediction in this paper.

Analysis of Residuals and Grouping of Intersections by CART

Further details, such as design, control, degree of conflicts, and environmental features of the intersections, are also considered to be major factors affecting safety at intersections. The importance of these factors can sometimes be reflected in the large variations between injury accidents and traffic intensity found in most scatter plots. One of the approaches that can be used is to analyze the residuals of the base model on the basis of other characteristics of the intersections. In other words, those intersections with similar characteristics that have higher or lower accident records than other intersections in general would be grouped together. The residual is defined as the difference between the observed value and the value predicted by the base model.

The next question is how many groups should be selected to represent high– or low–accident risk intersections. Extreme solutions include one and n groups, where n is the number of intersections in the data base. With a single group, the model is equivalent to the base model and is therefore not interesting, because some understanding of the design factors that tend to affect the safety of intersections should be provided. If there are n groups, it might be possible that the given characteristics of the intersections cannot be used to produce a grouping that reflects similar accident patterns. Also, if n groups are used, there is no way to identify those intersections that are "out of line" for accident surveillance purposes. Engineering judgment and a technique to group intersections with error measures would therefore be important to the process of getting an optimal group size.

The CART (Classification and Regression Trees) program can be used to analyze the residuals of the base model and then group intersections with similar accident patterns. The refinement of estimates by grouping intersections on the basis of other intersection characteristics is called the Level II prediction in this paper.

Adjustment by Accident History

The materials just covered refer to estimates of accident prediction for groups of intersections. In other words, all intersections within a certain group will have an equal estimate. It can be argued that the grouping made or models derived were only based on information that is available in the list of predictor variables, which may not contain all the factors involved. The kind of information that cannot be captured by the models or variations within groups can be found easily in the accident history of individual intersections. As a result, the accident history of individual intersections could become a very valuable source of information that reflects the safety level of individual intersections.

Level I and Level II predictions refer to group estimations or predictions, whereas the Level III prediction is aimed at the concept of safety estimation at individual intersections. The idea of linear combination of group estimate and individual accident history, as proposed by Hauer and Persaud (*4*), was used in the study. A detailed discussion on Level III prediction and its application to the proposed methodology is presented later in this paper. This staged procedure could allow flexibility to users that have different input requirements. At the same time, it gives them an opportunity to observe the evolution of their estimates.

CLASSIFICATION AND REGRESSION TREES (CART)

CART is a computer program and technique developed by Breiman et al. to classify and group entities on the basis of a set of measurements or characteristics, using tree methodology (*2*). CART is particularly useful to this study because the data set has high dimensionality, a mixture of data types (continuous and categorical), and (perhaps) lack of homogeneity. Lack of homogeneity refers to different relationships that hold between variables in different parts of the measurement space. Most important of all, the tree structure output provides information regarding the main factors and interactions between factors that are important in predicting accidents in a form that is easily interpreted and understood. CART differs from other tree structured programs in the pruning and estimation process, that is, in the process of "growing an honest tree" (*2*, Chapter 8).

Concept and Theory

The concept involves partitioning the intersections by a sequence of binary splits into terminal nodes, as shown in Figure 2. In each terminal node t, the predicted response value $y(t)$ is a constant. Because the predictor $d(X)$ is constant over each terminal node, the tree can be thought of as a histogram estimate of the regression surface, as shown in Figure 3. In developing predictive models by a tree technique with a learning sample, L, the following three issues for determining a tree predictor should be addressed:

- A way to select a split at every intermediate node,
- A rule for determining when a node is terminal, and
- A rule for assigning a value $y(t)$ to every terminal node t.

A detailed discussion on these subjects can be found in Breiman's work (*2*). To put it in simpler terms, the issue of the node assignment rule is the easiest to resolve, and it can be shown that the average of y_n, the observed value of the response variable, in terminal node t could be used to represent the group.

A regression tree is formed by iteratively splitting nodes, and one way to select a split at every intermediate node is to

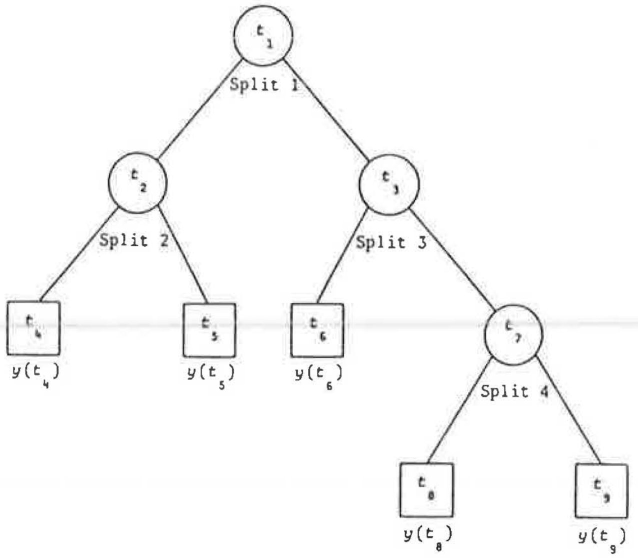


FIGURE 2 Partitioning of intersections.

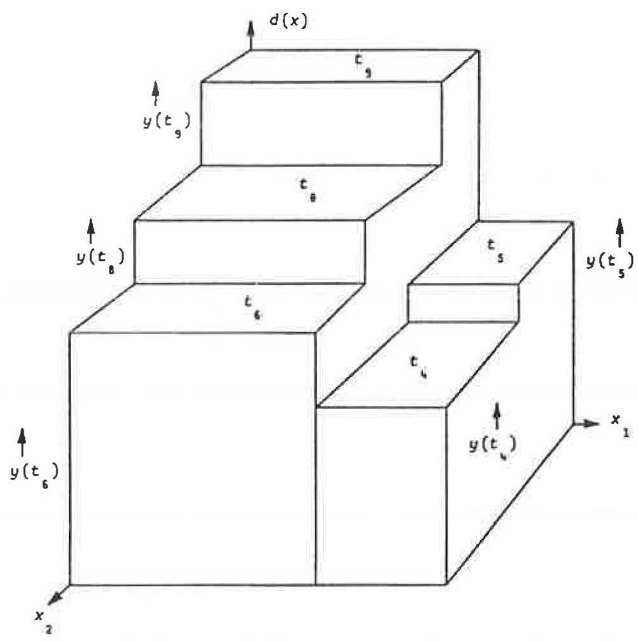


FIGURE 3 Histogram estimate at nodes.

maximize the decrease in the resubstitution error measure. The best split of t is the split that minimizes the weighted variance

$$P_L S^2(t_L) + P_R S^2(t_R) \tag{1}$$

P_L and P_R are the proportions and $S^2(t_L)$ and $S^2(t_R)$ are sample standard deviations of the cases in t that go left and right, respectively. Furthermore, this value is also smaller than $S^2(t)$, which is the sample variance of the y_n values in node t. In essence, the best split of t here corresponds to the most important factor that should be selected at t for splitting purposes.

The rule for determining whether a node is a terminal node is related to the issue of pruning of trees and can also be influenced by some options specified by users, such as minimum node size, tree selection rule, and so on.

Notes on the CART Program and Its Applications

The CART program is written in standard Fortran and can be run on most computers in both interactive and batch modes. One of the most important options that is available is the estimation method. This is the method specified by users to calculate the accuracy of models. There are three methods available in the CART program: resubstitution, test sample, and cross-validation.

Resubstitution uses all the cases in the learning sample, L, to construct predictor $d(X)$ and to estimate its error $R^*(d)$. For the resubstitution estimate, the following equation could be used:

$$R(d) = (1/N) \sum_n [y_n - d(X_n)]^2 \tag{2}$$

The problem with the resubstitution estimate is that it is computed with the same data used to construct the predictor $d(X)$ instead of an independent sample. This estimate therefore tends to give an overly optimistic picture of the accuracy of the predictor. Trees built by this method are sometimes called "exploratory trees." This is a rapid method of growing exploratory trees, and it can be used to explore a range of parameters, the effects of adding or deleting variables during preliminary stages, or both.

The second method is test sample estimation. This method randomly divides the cases, L, into two sets, L_1 and L_2. Only the cases in L_1 are used to construct predictor $d(X)$. The cases in L_2 are used to estimate $R^*(d)$ by the following equation:

$$R^{ts}(d) = (1/N_2) \sum_{(X_n, y_n) \in L_2} [y_n - d(X_n)]^2 \tag{3}$$

Care should be taken so that cases in L_1 are independent of cases in L_2. The drawback of this method is the reduction in effective sample size; thus the test sample estimation method is only recommended for problems with a large sample size.

For smaller sample sizes, the V-fold cross-validation method is recommended. The cases in L are randomly divided into V subsets of as nearly equal size as possible. For each v, $v = 1, \ldots, V$, apply the same construction procedure to the learning sample $L - L_v$, so that the predictor $d^{(v)}(X)$ is obtained. Then the estimate can be computed as follows:

$$R^{cv}(d) = (1/N) \sum_v \sum_{(X_n, y_n) \in L_v} [y_n - d^{(v)}(X_n)]^2 \tag{4}$$

Cross-validation is the recommended procedure because it provides "honest" trees. It is preferred over test sample procedure because every case is used in constructing the predictor and calculating error estimates.

Options are also available for selecting different sizes of optimal trees. These options are explained in detail in the work of Breiman et al. (2).

ACCIDENT PREDICTION MODELS

It is generally true that the accuracy of prediction models depends on the details of the information base on which the models are built. This relationship should not depend on the type and complexity of the models or techniques used. In other words, it should be possible to improve the accuracy of a model by feeding more information into the process. This pattern was also found in the current study. Evidence is presented in the examples given at the end of this section. The initial parts of the section describe the application of the proposed procedure.

Level I: Model Based on Traffic Intensity

It has been demonstrated in this data set that traffic intensity was indeed the most important single factor in predicting injury accidents. A base model was built on this fact. In this paper, traffic intensity is expressed as the total number of vehicles (in millions) entering an intersection per year, calculated from average daily traffic on cross and main streets. A scatter plot with the average number of injury accidents per year (IACCYR) on the vertical axis and millions of vehicles entering an intersection from all legs per year (MVYR) on the horizontal axis is shown in Figure 4. There are only 2,488 intersections or points in the figure because 10 intersections had incomplete information.

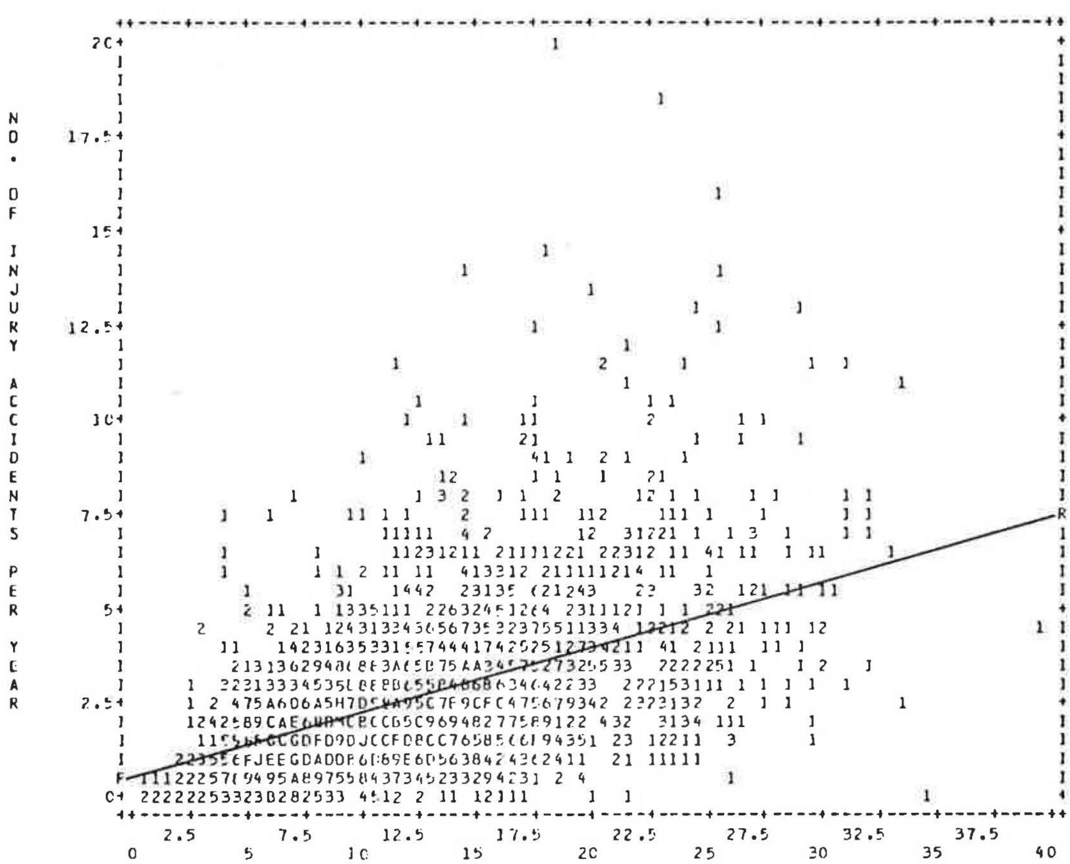
Different functional forms, including power and logarithm transformations, have been used to fit the data, and a straight line relationship was finally selected because it was as good as any other form. Estimates of slope and intercept were then obtained by regression analysis, as shown in Figure 4. Other estimation methods could have been used but were considered unnecessary. The following equation was selected as the base model for injury accidents per year:

$$\text{FIACCYR} = 0.61856 + 0.16911 * \text{MVYR} \qquad (5)$$

The question of nonzero intercept was also discussed, and it was decided that no adjustment should be made, simply because such an adjustment would introduce biases in the estimation process and the difference would be minimal. However, when MVYR is less than 3.0, a warning to users should probably be made. On the other hand, no consistent biases (overestimation or underestimation) were found in the estimation process.

Level II: Models Based on Intersection Characteristics

The residuals of the base model, as shown in Equation 5, are analyzed by CART, and the factors and levels that are available and used in the analyses are shown in Table 1. These factors were selected from the TASAS system after consultation with some of the Caltrans practicing engineers who will be using the models. Fortunately, missing values represent only a very small proportion of the data set and are treated as separate levels in the analysis for simplicity reasons. An additional factor (IADT), called the index of conflict, was also created for the



No. of Vehicles Entering Intersection Per Year in Million, MVYR
Coefficient of Correlation=0.47498
Slope=0.1691 (t value=26.9), Intercept=0.6186 (t value=6.9)

FIGURE 4 Scatter plot and regression result of base model.

study because turning movement counts were not available in TASAS. This index of conflict is calculated as the proportion of cross street traffic to total traffic. IADT turned out to be a very important factor in predicting intersection accidents, as shown in Figure 5, which shows a ninefold cross-validation tree. The tree, with nine terminal nodes and a relative error of 0.90, was selected by the 0.3 standard error rule. A relative error of 0.90 implies that CART is able to provide a further reduction in error of about 10 percent. This also shows that the 2,488 intersections can be divided into nine groups with similar accident characteristics on the basis of the tree structure, as shown in the same figure.

Group 1 is characterized by intersections that have an index of conflict of less than or equal to 0.065, that is, 6.5 percent of all entering traffic is on the cross street. A simple classification rule in this situation appears to indicate that when the level of conflicts is low, no other factor except traffic intensity is significant in affecting the safety of intersections. There are 492 intersections in this group, with an average residual of about −0.73 and a standard deviation of about 1.5. The average negative residual of −0.73 here means that 0.73 injury accidents should be deducted from the base model derived from Equation 5. In simple terms, intersections in this category generally have a lower risk.

Group 2 is characterized by intersections that have an index of conflict larger than 0.065 and are either "T," "Y," or multilegged intersections. Because less than 3 percent of the intersections in the data set are multilegged, it can be assumed that this group represents those intersections that have fewer legs. This could be why they have lower accident statistics in comparison with intersections with more legs, such as four-legged and offset intersections. As shown in the intermediate node, these latter intersections have an average residual of 0.36, compared with −0.62 in Group 2.

Group 3 has an average residual of 0.05. Intersections in Group 3 have narrow cross streets (less than or equal to a total of three lanes), and they tend to have smaller residuals than the remaining groups (4 to 9).

In comparison with the intermediate node corresponding to intersections with pretimed controllers and two-phase fully actuated controllers, Group 4 has a smaller average residual than the other groups. It could be argued from a safety viewpoint that it might be worthwhile to spend additional money on installing actuated controllers.

Group 6 is similar to Group 5, with the exception that the index of conflict is greater than 0.475. Both groups have pretimed signal controllers. It is obvious that Group 6 has a much higher accident potential than Group 5. This difference is due to a higher level of conflict.

Groups 7 and 8 are similar to groups 5 and 6, with the exception that all of them have two-phase fully actuated controllers. Group 7 includes intersections that have 5 lanes or fewer on main streets, but Group 8 includes only those intersections that have 6 or more lanes on main streets. Group 8 has a higher residual (3.56) than Group 7, and this appears to indicate that very wide intersections might require more than two phases to accommodate the turning movements that occur in these intersections.

Group 9 has an average residual of 4.82, which appears to indicate that left turn prohibitions during peak hours may not increase risk.

From the results, it can be seen that the groupings and their estimates are not unreasonable and that their characteristics could provide additional insights to engineers who are designing or redesigning signalized intersections. These models contain as many reasonable factors as other models obtained by regression analysis for similar projects in which field observations and accident record systems were used (*5*).

Level III: Models Based on Information that Includes Individual Accident History

It is obvious that no matter how complicated the models are, there are likely to be some unique intersection features that cannot be captured. One method of compensating for this is to employ a concept of combining estimates on the basis of the accident history of the intersection and estimates for the group in which the intersection belongs. Hauer and Persaud (*4*) have derived an estimate, Z, on the basis of a linear combination of the two results of two approaches for predicting the safety of an individual intersection. The estimate is obtained from the following equation:

$$Z = aE\{m\} + (1 - a)x \tag{6}$$

where

$$a = (1 + \mathrm{Var}\{m\}/E\{m\})^{-1}$$

and where m = expected accident statistics and x = accident count. Hauer and Persaud also suggested that the sample mean $\bar{x}$ could be used to estimate $E\{m\}$ and that sample standard deviation(s) could be used to estimate $\mathrm{Var}\{m\}$ by the following equations:

$$E\{m\} = E\{x\} \tag{7}$$

$$\mathrm{Var}\{m\} = (s^2 - \bar{x}) \tag{8}$$

This combination technique does not appear to be very powerful for predicting future accidents if there is a long accident history with many accidents. However, the technique is particularly useful in predicting accidents at intersections with few events and short history. This combination technique would not be applicable to new intersections because a new intersection would not have any accident record or history.

Examples Illustrating the Procedures and Overall Significance of the Models

The procedure to be illustrated has a three-level structure, and users can terminate the analysis at the end of any level to suit their input requirements. Five intersections in the data set were arbitrarily selected for illustration purposes. The results, based on the procedures described in earlier sections, are tabulated in Table 2. The first eight rows, from MADT to MNL, are the characteristics of the intersections, described in Table 1. MVYR is the independent variable of Equation 5, and FIACCYR is the dependent variable of the same equation. FIACCYR is also the estimate for Level I prediction. IACCYR is the observed number of injury accidents per year that occurred at the intersection. $E\{x\}$ is the average or sample mean of the residuals of the subgroups based on grouping by CART.

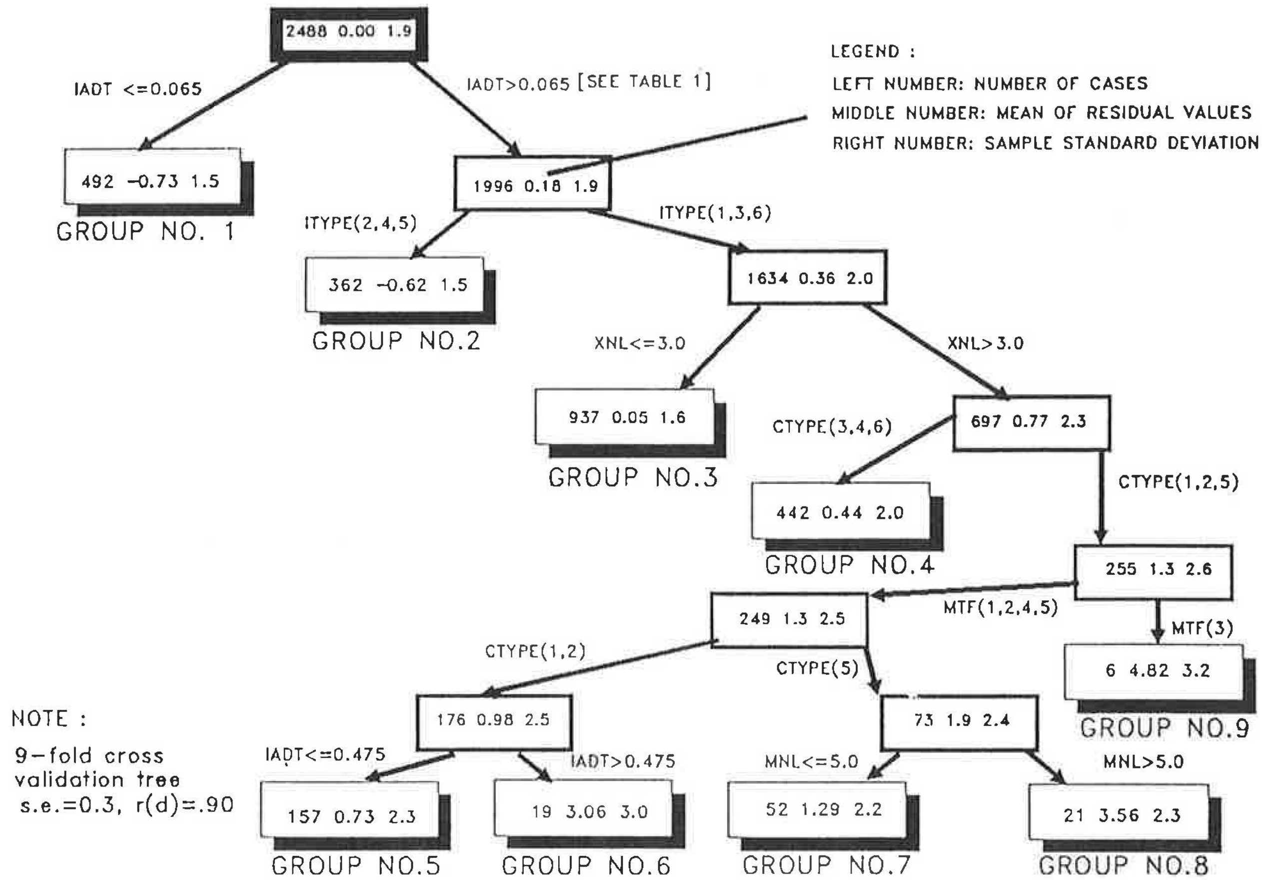


FIGURE 5 Grouping of intersections by CART.

CIACCYR is the estimate for Level II prediction, and it is calculated as the algebraic sum of $E\{x\}$ and FIACCYR. In other words, $E\{x\}$ by CART is the adjustment factor to account for different design or control features of a particular intersection.

For Level III prediction, the standard deviation(s) of the subgroups from CART and individual accident history would be required. The proportion or weight (a) to be used can be derived from Equation 6. The x in Equation 6 is calculated as the difference between the observed number of injury accidents (IACCYR) and FIACCYR. Finally, the new adjustment factor (Z) can be calculated with Equation 6, and the estimate for Level III prediction (HIACCYR) would be the sum of Z and FIACCYR.

It can be seen that the differences between the predicted and observed values generally improve from Level I to Level III. This improvement is not surprising because input requirements increase from Level I to Level III. It can be argued that examining the differences between the observed and predicted values for Level III predictions is not fruitful because they are adjusted according to the actual differences between observed and calculated values. Furthermore, it is not meaningful to compare the predicted values (especially Level III) with the observed values because the predicted values represent future accident potential. Out-of-sample testing would be more meaningful in this regard; however, this type of test might not be feasible due to the length of the record systems. A check on the accuracy of the models predicted by Levels I and II was also performed. This check could be considered redundant because the accuracy of these models was assured by the large t values of the estimates, as shown in Figure 4 for Level I, and by the cross-validated tree for Level II, as shown in Figure 5. A correlation of 0.475 between the actual and predicted values was found for Level I, and a correlation of 0.580 was found for Level II. A similar analysis for Level III was not carried out because Level III predictions represent *future* accident potential. The improvement from 0.475 to 0.580 might not appear to be large; however, it should be recalled that only existing information on road designs and traffic conditions found in TASAS was used, and factors such as driver's characteristics, vehicle design, weather, and so on were not employed in the analysis.

CONCLUSIONS AND LIMITATIONS

The models derived here for injury accidents include factors such as traffic intensity, percentage of cross street traffic, intersection type, signal type, number of lanes on main and side streets, and left turn arrangements. However, only intersection type and traffic intensity are included in the current models used in California. The other factors used in the present models, such as rural or urban conditions and being inside or outside city limits, have not been found significant in this study. The models proposed in this paper have higher predictive power, and they also provide more insight for engineers who are designing or redesigning signalized intersections and evaluating alternative strategies.

TABLE 2 EXAMPLES ILLUSTRATING THE PROCEDURE

ID#	2574	2541	1794	934	290
MADT*	49000	37000	18100	20998	16000
XADT*	10000	301	3000	17111	1501
IADT*	0.169	0.008	0.142	0.449	0.086
ITYPE*	1	4	1	4	1
XNL*	4	2	2	4	2
CTYPE*	2	5	1	6	5
MTF*	2	2	2	2	2
MNL*	4	4	4	4	2
MVYR	21.53	13.61	7.70	13.91	6.39
IACCYR	5.86	1.33	3.71	0.00	2.00
LEVEL I(Equation 5.1)					
FIACCYR	4.26	2.92	1.92	2.97	1.70
LEVEL II(Figure 5.3)					
E(x)	0.73	-0.73	0.05	-0.62	0.05
Group no.	5	1	3	2	3
CIACCYR	4.99	2.19	1.97	2.35	1.75
LEVEL III(Equations 5.2-5.4)					
S	2.3	1.6	1.6	1.5	1.6
a	0.14	0.32	0.02	0.28	0.02
x	1.60	-1.59	1.79	-2.97	0.30
Z	1.48	-1.31	1.76	-2.32	0.30
HIACCYR	5.74	1.61	3.68	0.65	2.0

* Please refer to Table 1 for more details

LEGEND

ID#: Intersection Identity Number
MADT: Main Street ADT
XADT: Cross Street ADT
ITYPE: Intersection Type
XNL: Total Number of Lanes on Cross Street
CTYPE: Control(Signal) Type
MTF: Main Street Traffic Flow
MNL: Total Number of Lanes on Main Streets
MVYR: Total Number of Vehicles Entering Intersection per Year in Millions
IACCYR: Actual Number of Injury Accidents per Year
FIACCYR: Level I Forecasted Number of Injury Accidents per Year
CIACCYR: Level II Forecasted Number of Injury Accidents per Year
HIACCYR: Level III Forecasted Number of Injury Accidents per Year

As far as development of macroscopic models for injury accidents is concerned, it is concluded that the proposed methodology and TASAS are very suitable for this kind of study. The results obtained are not unreasonable and correspond with conventional wisdom. These factors would normally be expected to have a high degree of association with accident patterns. However, it could be argued that factors such as phases for left turn vehicles, provisions of left turn pockets, number of conflict points, and so on should also play an important role in the prediction process. Unfortunately, models with this kind of detail would require specific information on turning movement counts, conflict analyses, and other factors. Such information is not available in most accident record systems, including TASAS.

The proposed methodology follows the general pattern used in system analysis, thus allowing room for refinements and changes, if necessary. The three-level prediction procedure allows flexibility by having different levels of inputs. At the same time, this procedure gives users an opportunity to appreciate the evolution of their estimates. The selection of the response variable could also play an important role in the model development process. The use of number of injury accidents per year as the response variable has kept this analysis simple throughout. The use of traffic intensity as an exposure measure (similar to accident rate) can cause some problems, and this factor should be used as a predictor variable instead. For example, a reduction in accident rate with an increase in traffic flow has been found in some studies by regression analysis with accident rate (accidents per vehicle) as a dependent variable (*5*). Of course, accident rates could be used to summarize results at the end of the analysis, if this is found to be convenient.

The analysis of residuals of the base model by Classification and Regression Trees (CART) has proved to be a viable technique for building models based on data sets that have high dimensionality, a mixture of data types, and lack of homogeneity. Its tree structure output makes the interpretation of results easy. On the other hand, incorporating and interpreting interaction terms between factors can be quite time consuming in a regression analysis.

For future research, the following tasks would be useful in overcoming some limitations of the work completed to date. The models derived in this paper can be called macroscopic models, and there could be further attempts to analyze the accident aspects of the data base, for example, types of collision, time of occurrence, road condition, types of vehicle, driver's condition, and so on. On the other hand, the issues of underreporting of PDO accidents and the slight difference between serious injury accidents and fatal accidents should be considered and examined in future studies.

ACKNOWLEDGMENTS

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A Mobile Illumination Evaluation System

RICHARD A. ZIMMER

State and local transportation agencies have responsibility for the design, construction, and inspection of high-mast roadway lighting projects, using prescribed guidelines. These guidelines require many measurements of the illuminance levels of highway lighting installations to determine whether the installations achieve the desired design goals, to determine whether manufacture specifications are met, and to document long-term aging effects. The state of the art in photometry, or the measurement of light visible to the human eye, has been developed to a very high degree of accuracy and reliability by commercial manufacturers of photometric instruments. One large drawback to these instruments in highway work is that the readings are normally taken by an operator carrying the instrument to the location to be measured and hand-recording the value. This approach is very time consuming and dangerous as well, if readings are required from a busy, in-service roadway. This study has resulted in the development of a simple, cost-effective, and easily assembled illuminance measurement system to evaluate high-mast (80–180-ft) roadway lighting systems from a passenger vehicle traveling at traffic speeds. The system provides readings, in footcandles, at fixed distance intervals as short as 15 ft. The measurements are then recorded on a computer disk for later analysis. The printed analysis provides a footcandle value for each traffic lane at each distance interval and a calculation of the maximum, minimum, average, and uniformity ratio. The system is assembled from commercially available units with a minimal amount of construction, resulting in easy implementation.

State transportation agencies have responsibility for the design, construction, and inspection of roadway lighting projects, using prescribed guidelines (*1*, *2*). In the course of carrying out this responsibility, it is periodically necessary to measure illuminance levels of highway lighting installations to determine whether they achieve the desired design goals, to determine whether manufacture specifications are met, and to document long-term aging effects. The development of equipment, techniques, and methods for these measurements is not a new requirement, to judge from publications such as *Measurements Carried Out on Road Lighting Systems Already Installed* written by P. J. Bouma in 1939 (*3*).

The state of the art in photometry, or the measurement of light visible to the human eye, has been developed to a very high degree of accuracy and reliability by commercial manufacturers of photometric instruments. One such instrument, used by the Texas State Department of Highways and Public Transportation (SDHPT), is the Tektronix J16 Digital Photometer. This highly accurate instrument (2 percent) has been used to obtain illuminance readings both at controlled research facilities such as Texas Transportation Institute (TTI) and on highways throughout the state of Texas. To obtain the data, the operator is required to carry the portable photometer to each location, place the sensor on the pavement and write down the digital footcandle meter reading. Such a method produces accurate results but is very time consuming and dangerous as well, if readings are required from a busy, in-service roadway. Vehicle-mounted measurement systems are not new either. One example is the measuring van used to measure illuminance and luminance on road surfaces, developed by Baba in 1969 (*3*). Even though this system was quite functional, it had a drawback in that a large van was needed for the large amount of electronic equipment and power supply, producing a high cost. As technology has progressed the microprocessor has made the mobile measurement system approach more manageable, as exemplified by the "Dynamic Roadway Lighting Measuring System with Split Type Photocells" developed by Y. Ohno (*4*). This system appears to be effective in measurement work, but it uses two CIE-corrected sensors (CIE = International Commission on Illumination) and two signal conditioners, a complex position correction algorithm, and noncommercial processor equipment. These characteristics limit availability and restrict service and calibration support, as well as increase the cost. Because the primary requirement given by SDHPT was to evaluate high mast-lighting systems, it was determined that a simpler system could be implemented. This is because of the small difference between the lamp height to the road and the lamp height to the sensor mounted on the roof of an automobile.

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SYSTEM REQUIREMENTS

So that a large number of illuminance readings could be obtained to provide a record of average, minimum, and maximum footcandle levels along heavily traveled highways, the measurement system would have to be automated and operated in a vehicle traveling at traffic speeds. Footcandle readings could then be taken at reasonable speeds (up to 50 mph) every 20–30 ft on roadway test sections and stored. The stored data could then be analyzed by computer later to determine photometric performance of the roadway lighting system.

Specific requirements of the system were as follows:

- Equipment (photometer, distance measurement device, data storage element, and necessary cabling) should be portable, that is, easily attached and removed from an automobile.
- The system should provide footcandle readings to be taken at 20-foot intervals at speeds up to 50 mph. The computer interface and software program should be compatible with IBM-PC™ computer. The system should be designed for entering identification data (ID) for each test run (i.e., highway and lane, start and stop points, etc.).
- The computer program should provide calculations for each test section, including average footcandles, minimum

footcandles, maximum footcandles, and average to minimum ratio.

- The system should exhibit a high degree of accuracy; it should at least be comparable with current practice.
- The system should be cost effective (approximately $5,000 hardware cost) and should not use higher orders of sophistication than those necessary to accomplish the required tasks.

SYSTEM DESIGN APPROACH

In an effort to provide a working system that met the system requirements, four tasks were undertaken:

1. Determine system hardware and software requirements;
2. Identify, acquire, and assemble required hardware subsystems;
3. Develop and test the required software; and
4. Test and finalize the system design.

SYSTEM HARDWARE

Photometer

Because highly accurate electronic digital photometers are commercially available, it was decided that improvement of the current technology should not be pursued. Instead, a unit that met the system requirements was purchased. Because SDHPT has successfully used the Tektronix J16 in the past (Figure 1), it was considered highly suitable for the task. The requirement for interfacing the unit with a digital data acquisition system without degrading its accuracy was obtainable by requesting option 07. This option is a BCD (binary coded decimal) output that is simply an electrical representation of the digital display. This BCD output, which is available at a connector on top of the unit, is updated at a rate of six readings per second. This rate exceeds the system requirement of a reading every 20 ft at 50 mph, or 3.66 readings per second. The J16 uses a separate probe at the end of a 25-ft cable that is magnetically mounted on the roof of the vehicle. The probe is very accurately cosine-corrected; thus oncoming traffic headlights have no effect because the cosine of 90 degrees is zero. In addition to the 13 data lines from the J16, there are two control lines. One indicates when the data are valid, and the other is an input to hold the current reading.

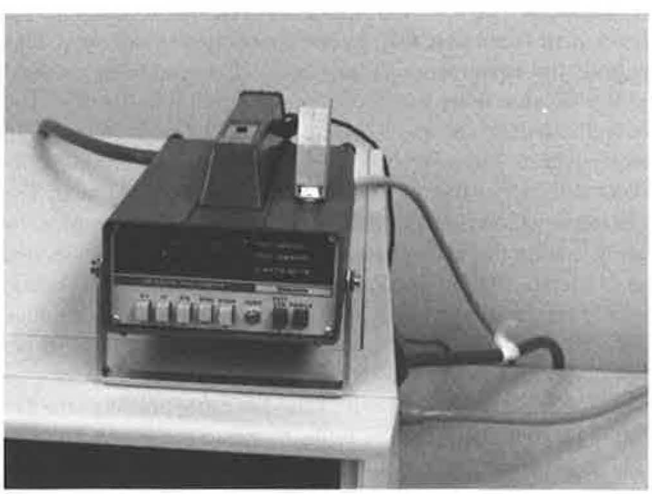

FIGURE 1 Digital photometer.

Computer

The requirements for the computer were that it must be portable; must possess parallel input capability as well as a counter, or have card slots available; be IBM-PC™ compatible; and have adequate storage capability. After a survey of current laptop and portable computers was made, a Compaq portable with dual floppy disk drives was selected for use. This unit provided the required expansion slots needed for the parallel I/O card and counter/timer card. Because technology is rapidly changing, a smaller unit may be found in the near future. The current Compaq, however, already fits nicely in a passenger vehicle (Figure 2).

Distance Sensor

Originally, a "fifth wheel" was considered for obtaining distance information for the computer. The device would be attached to the bumper of the test vehicle. This highly accurate unit is very expensive and easily damaged if the vehicle is backed up over it, so alternate sensors were considered. A low-cost sensor that attached in line with the vehicle speedometer cable was located (Figure 3). The attachment point is quite simple if the vehicle has a "cruise control" under the hood with removable speedometer cables. Because typical speedometer cables turn 1,000 revolutions per mile, a sensor that produces 20 pulses per revolution will provide 20,000 pulses in a mile. This equals a pulse every 3.169 in. of forward travel, which provides adequate resolution for the system. The sensor used in the

FIGURE 2 System computer located in rear seat of an automobile.

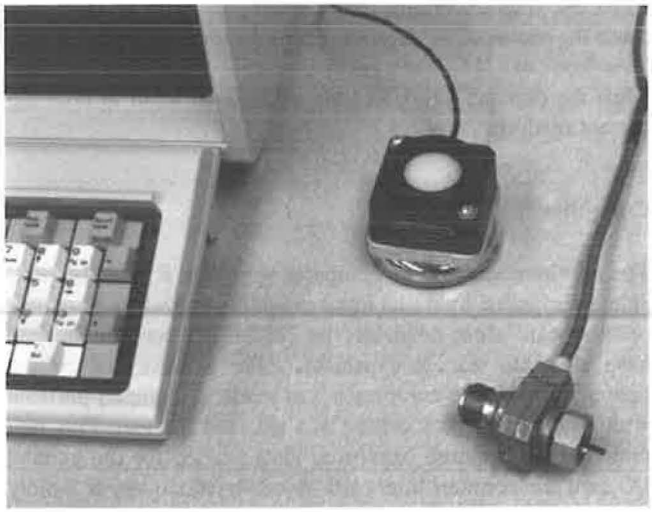

FIGURE 3 Illuminance probe and magnet (center) and distance sensor (bottom).

FIGURE 4 System power inverter.

prototype is a Hall Effect unit (Model AA-1422-20, Arthur Allen Manufacturing Corporation), which requires 5 volts power and produces a 5-volt (TTL) clean square wave output.

Counter Interface Board

So that illumination readings could be taken at regular intervals along the roadway it was necessary to have the computer system be capable of counting distance pulses from the sensor. This was done by using a CTM-5 plug-in circuit board manufactured by Metrabyte Corporation. This board provided five counter/timers that were readable from the main program by means of a binary subroutine. The distance sensor obtains its operating power through a 37-pin connector on the card and connects to two concatenated counters to provide a maximum count of 4×10^9.

Parallel Interface Board

A MetraByte PIO 24 parallel I/O board was used to interface the J16 photometer with the computer. A 37-pin connector located on the end of the board and accessible at the side of the computer provides for the 13 data lines and 2 control lines. The card is initialized and configured for input or output from software.

Power Supply

Because the system was to operate in a mobile environment in a standard passenger vehicle, the only source of power to be considered was the vehicle electrical system or auxiliary batteries. It was decided to use the 12-volt automotive system. Because the photometer and computer operate on 120 V ac, a power inverter was required. A frequency-stabilized 250-watt Tripp Lite unit was obtained and was found to supply an adequate amount of power (Figure 4).

THEORY OF OPERATION (HARDWARE)

Many of the critical design aspects in a well-established photometer such as the Tektronix J16 are taken care of by the manufacturer. The J6511 probe is used with the J16 to provide illuminance readings in footcandles (Figure 3). The J6511 is a multielement glass filter and silicone photodiode that ensures a close match to the CIE photopic curve (color corrected). The silicon sensor recovery time is virtually instantaneous, so low light levels can be measured immediately after exposure to bright light. The angular response is accurately cosine corrected, simulating an ideal 180-degree field-of-view detector. Because the unit is cosine corrected and located on the roof of the vehicle, oncoming traffic headlights have no effect on the readings. The sensor is attached to the roof of the vehicle by a doughnut ceramic magnet, similar to those found on mobile antennas. A 25-ft cable connects the sensor to the J16, which operates inside the vehicle near the computer. The J16 was ordered with options 03 and 07. Option 03 provides for 115 V ac operation, and option 07 provides the BCD output, as noted previously. Because The BCD output to the computer is a direct representation of the LED digital display, the accuracy of the instrument is not degraded by the connection in any way. This permits the equipment warranty and calibration to be retained in full because there has been no modification to the unit. The overall accuracy of the J16 and J6511 probe is very good: linear within 2 percent over the entire range, enabling single-point calibration and a stability within 2 percent per year. The electrical calibration of the J16 mainframe is performed by using a calibrated voltage source or digital volt meter calibrated to National Bureau of Standards specifications. Calibrated probes can be used with any J16 without additional calibration. BCD data are transferred from the $3^1/_2$ digit display (4 bits per full digit and 1 bit for the $^1/_2$ digit) from a 25-pin connector on top of the unit. A 4-ft, 25-wire shielded cable connects the J16 to the parallel interface card in the Compaq computer. Thirteen TTL compatible lines are used to provide BCD data to the card, one line is common, and one line is used to hold the J16 reading.

The other source of information to the computer is the forward travel distance of the vehicle, originating with the speedometer line pulser. The Hall Effect unit used in this study produces 20 pulses per revolution and attaches easily at a General Motors cruise control unit. Arthur Allen Corporation provides other fittings, extensions, and adapters to fit many types of vehicles. The sensor requires 5 volts at about 10 milliamps and produces a TTL-compatible output if it is ordered with a pullup resistor. The pulse or square wave output is connected to the CTM-5 card by means of a three-wire shielded cable. The pulser output is connected directly to counter 2, and counter 2 output is connected to counter 3 input. This way, if counter 2 should reach an overflow condition of 65,537, counter 3 will be incremented by 1, and counter 2 will be reset to zero and continue counting. Thus a total count of $65{,}536^2$ is permissible for distance calibration or data collection.

Before the interface cards are installed in the Compaq computer, they must be properly configured for the hardware used in the system. This is simply done by setting their "base address" switches to the correct I/O address location. In this system the CTM-5 card is set to 300 hex, and the PIO 24 card is set to 200 hex. These locations will be referenced by the software to perform specific initialization or I/O functions. The interrupt control on the CTM-5 card is not used and should be left in the off or (x) position. Installation of the cards in the Compaq computer should be left to a qualified technician because opening first the plastic and then the metal case requires special tools, as outlined in the Compaq user's guide.

Power for the system is provided by the square wave, frequency stabilized power inverter (12 V dc-115 V ac) by Tripp Lite, mentioned earlier. The unit chosen for the system was model PV-250FC, which is frequency controlled to 60 Hz and has a maximum load rating of 250 watts. The computer and photometer draw 140 watts maximum, which results in a dc input to the inverter of about 10 amps. For best results the inverter should be connected directly to the vehicle battery through a 20 amp fuse and 10- or 12-gauge wire.

THEORY OF OPERATION (SOFTWARE)

The heart of the measurement system is the software programs that control the data gathering process. The main program is written in interpretive BASIC. Even though this language does not have the speed of compiled BASIC or other compiled languages, it is easily modified by the user to suit current or future requirements. The CTM-5 counter/timer card uses a special binary program, CTM5.BIN, to set the many mode registers and counter control registers and to handle initialization. The CTM5.BIN program must be on the same disk as the BASIC program but is user transparent because the BASIC program handles the loading and calling routines. The disk operating system (MS DOS™, Version 2), BASICA (Version 2), and AUTOEXEC.BAT must also be on the working disk. The AUTOEXEC program is set up to automatically start the main program running when the computer powers up if the working disk is in drive A.

The program (MILES1.BAS) consists of two sections. The first contains the measurement routines and the second is the analysis portion.

Measurement

When the program is started, an identification page appears on the screen with a two-choice menu. The first choice is to take measurements, an operation that will be discussed now, and the other to analyze the results of any recorded test. As the measurement program is started, the CTM-5 card is initialized by loading the binary driver by contracting BASIC's workspace to 48 kilobytes. The master mode registers are set, and the counter mode registers are initialized by call routines. Because the distance pulser may be changed from vehicle to vehicle, it is always necessary to calibrate this unit in a new installation by selecting "calibrate" from the menu, setting up the CTM-5 card as a simple counter. The space bar is used to start the counter totaling over a measured distance. The F10 key terminates the count at the end of the measurement run. While the counter is totaling, the program is looping and taking a "snapshot" of the count, which it displays on the screen for operator verification of proper operation several times per second. Once the count is terminated by the operator, the program requests the distance traveled. The total count is then divided by the distance in feet to provide a constant of pulses per foot to be used in later calculations. If the same vehicle has been used before, the program allows the operator to input the constant directly.

The next step is to ask whether the data are to be saved to disk. If permanent storage of data is not needed, the program will show photometer readings on the screen but will not save them to disk. If disk storage is chosen, the program will request that a formatted disk be placed in drive B and the space bar pressed. Once this is done, a certain amount of error checking is performed to see that the disk is formatted, that the "write protect" tab is off, and that the disk is in the drive correctly. If all is correct, a request for a file name will be made. This file name will be the name of the data disk file, with no extensions. Again, the error checking function will check whether the file name already exists and if so, request that it should be replaced.

Once the data disk is in place and initialized, the program will ask for the header information (ID). This information, which is pertinent to the test time and location, is entered on the keyboard by the operator and saved to disk in the same order in string variable form. Next, the operator is requested to input the J16 range setting used for the test. This is necessary because there is no way for the computer to know the distance of the meter from the BCD output connector. The program then requests information on how far apart the readings are to be taken. This number, in feet, is subsequently multiplied by the pulses per foot constant to arrive at a pulses per reading value. A limit of 15 ft has been placed in the selection to provide enough time for the J16 to integrate between readings at 50 mph (approximately six readings per second maximum). A request is then made for the distance from the luminaire to the center of the first measurement lane. This value is used in the analysis phase to determine lateral displacement. The lane width, which is used to provide for additional lateral displacement, is also requested. Once the constants have been determined, the program is ready to measure light values.

The light data taking routine is one of the most complex and demanding parts of the program. The CTM-5 card is reconfigured to count down repetitively from a value placed into the "load register." This value is the pulses per foot times the feet between readings. As the terminal count is reached (0), the output of the counter is set to toggle or flipflop, and a reading is processed. When initialization is complete, the screen will switch to a large print mode and request that the space bar be pressed to start the measurements. Once the operator presses the space bar at the appropriate roadway location, the screen will display photometer readings in the center and the measurement lane at the upper left corner. To do this, the program pauses until the terminal distance count is reached and the "status bit" of counter 2 changes (high to low or low to high). At that time, a "hold" level is sent to J16 via the PIO 24 card to freeze the display reading. The BCD data are read at that time by the PIO 24 card in 2 bytes, or 16 bits, and then the hold line is released. The two data bytes are mathematically operated on to rebuild the decimal representation of the J16 readout, between 0 and 1999. Each reading is tested for an over-range condition before it is stored. If a over-range condition from the J16 is detected (>2,000), an alarm beep will be sounded, and a value of 9999 will be substituted for the erroneous reading.

The photometer values are displayed on the screen and recorded in an array that was previously dimensioned to 1,000. A short beep is sounded each reading to confirm that measurements are being taken. Once a reading is saved in memory, the program stops and waits for another status change from the distance counter to take another reading. If the F10 key is pressed by the operator, indicating the end of the test section, the program stops taking data and proceeds to restore the counter to a reset and resting state. All data in the array are then written to the data disk in floating point form, using 4 bytes per data point with the number 999 at the end of the record. Storage efficiency could be increased by a factor of two by storing the data points as integers with one scale factor, but the method used allows the ASCII disk files to be examined directly in footcandles.

After the data from a test run are saved to disk, the program displays a menu with choices to Measure another lane, Measure another site, or Quit the program. If another lane is selected, the program will increment the lane number and loop back to start measurements again. Each additional lane will be saved sequentially under the same file name with a 999 at the end of each lane record. If a new site is selected, the program will close the disk file and loop back requesting a new disk file name and new header information. If Quit is selected, the disk file is closed, and the program returns to the initial menu page.

Analysis

The data analysis portion of the program is entered by selecting the second option on the main menu screen. After initialization, the program requests whether the output is to be directed to the screen or the printer. The file name to be analyzed is then requested, and the current data files on drive B are displayed. Once a name is entered, a test for errors is made. If none are found, the header information is printed on the selected device. If the screen has been selected, the output will stop at the end of the header information for viewing. Then the footcandle information will be displayed, with each lane as a separate column at the various lateral distances from the luminaire. Each row represents the forward distance from the starting point, and the cumulative measurement in feet is indicated. At the end of the

```
              MOBILE ILLUMINATION EVALUATION SYSTEM
     Texas State Department of Highways and Public Transportation

   FILE NAME                  test137
   HWY. NO.                   IH35
   CONTROL-SECTION-JOB        1807-6-9
   COUNTY                     TRAVIS
   DIRECTION                  NORTH
   START STATIONING           234+50
   STOP STATIONING            678+50
   TESTING DATE               07/20/87

                      ILLUMINATION SUMMARY
                          (Footcandles)

   Travel *      Lane Distance From Luminaire (ft.)
   Dist.  *
   (ft.)  *     0          12          24
   -------*----------------------------------------
    0     *   0.40        0.45        0.43
    30    *   0.40        0.53        0.56
    60    *   0.68        0.68        0.63
    90    *   0.88        0.81        0.71
    120   *   1.15        0.97        0.83
    150   *   1.73        1.24        0.87
    180   *   1.48        0.89        0.67
    210   *   0.94        0.66        0.59
    240   *   0.82        0.59        0.42
    270   *   0.64        0.42        0.29
    300   *

   Average=   .7453334
   Maximum=  1.73
   Minimum=   .29
   Average/Min Ratio=   2.570115
```

FIGURE 5 Typical data printout.

footcandle readings are calculations of average, maximum, minimum, and the average to minimum ratio. A typical data printout is shown in Figure 5.

AVAILABILITY OF PROTOTYPE SYSTEM SOFTWARE

A copy of the BASIC program MILES1 and specific wiring information may be obtained by contacting the Texas State Department of Highways and Public Transportation D-8 or the Texas Transportation Institute.

The CTM5.BIN program is available under license from MetraByte Corporation and is supplied with a CTM-5 board.

MS DOS™ and MS BASIC™ are available under license from Microsoft Corporation.

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Fog Mitigation Update: Fog Mitigation Measures as Applied to Highway Bridge Structures

CORY B. POTASH AND JAMES R. BROWN

In response to a court action, a plan was developed to mitigate potentially hazardous effects of fog on a proposed highway bridge. The proposed site is subject to naturally occurring fog in addition to fog consisting of large quantities of water vapor emitted by a paper mill. A study was conducted to evaluate available mitigation measures. The measures evaluated include dispersion systems, guidance systems, design alternatives, transportation management, and incident detection. The purpose of this paper is to present the conclusions of the study. On the basis of the study results, a fog mitigation plan was recommended. The plan consists primarily of guidance and surveillance measures. The major components of the recommended plan include fixed message signs, raised reflective pavement markers, lighted pavement markers, highway surveillance, and variable message signs. The recommended plan was accepted by the governing agency and will be implemented. In addition, other considerations, which are presented in this paper, will be explored while the proposed bridge is under construction.

The Recommended Transportation Plan for the Charleston, South Carolina area, completed in 1968 and updated in 1975 and 1976, includes the proposed construction of the Mark Clark Expressway. The western segment of the expressway was approved for funding by FHWA in 1970, and a large portion of this segment has been constructed.

The eastern segment of the expressway would include a bridge spanning the Cooper River near a paper mill owned and operated by Westvaco, Inc. The mill emits over 6 million gallons of water vapor per day into the atmosphere. The eastern segment of the Mark Clark Expressway would be partially funded by federal aid highway funds and, as such, must meet all U.S. Department of Transportation (DOT), FHWA, and National Environmental Policy Act (NEPA) requirements. In conformance with these requirements, the South Carolina Department of Highways and Public Transportation (SCDHPT) completed a Final Environmental Impact Statement (FEIS) for the project, which was approved by FHWA. Subsequent to completion of the FEIS, Westvaco filed a complaint in the U.S. District Court for the District of South Carolina Charleston Division, alleging, among other things, violation of NEPA because SCDHPT and FHWA failed to address in the FEIS the threat to highway safety presented by fog.

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The complaint asked that the SCDHPT and FHWA be enjoined from engaging in any further activities directed toward building the bridge until they had complied with the NEPA. In conjunction with this action, Westvaco had prepared a series of studies evaluating the potential impact of naturally occurring fog and fogging caused by the large quantities of steam released from the Westvaco paper mill (mill-induced fog) on traffic and safety conditions at the proposed location of the Cooper River Bridge. The company also prepared studies evaluating alternative bridge locations and the feasibility of constructing a tunnel, rather than the proposed bridge.

SCDHPT reevaluated the proposed bridge in consideration of Westvaco's concern over the potential fog problem. As part of the reevaluation, SCDHPT prepared an independent evaluation of the potential for fog occurrences on the proposed Cooper River crossing. Subsequently, the SCDHPT issued an environmental assessment, based on their reevaluation, which concluded that fog would have no significant impact on the safety of motorists using the Cooper River Bridge at its proposed location. FHWA concurred with the conclusions of the environmental assessment and decided that it was unnecessary to supplement the FEIS.

After consideration of the additional studies completed by Westvaco and SCDHPT, and additional expert testimony, the court ruled not to enjoin the project but required SCDHPT and FHWA to file a Supplemental Environmental Impact Statement (SEIS) to consider the impact of fog on the proposed bridge and to prescribe the specific actions to be taken to mitigate any potential hazards due to fog.

In conformance with the court's order, the SCDHPT requested Parsons Brinckerhoff Quade & Douglas, Inc. (PBQD) to evaluate and propose measures to mitigate the potential fog hazards on the proposed Cooper River Bridge. In conducting the study, it was understood that there is no reasonable measure to completely eliminate the potential for accidents on the proposed Cooper River Bridge, with or without fog. The exploration of available measures sought to identify measures that would reduce the probability and severity of accidents occurring on the proposed Cooper River Bridge to levels approaching or equal to those without fog, although it was recognized that such levels may not be achievable. Recommended measures, however, are considered to be the most effective generally available measures to improve safety conditions on the proposed Cooper River Bridge under fog conditions. It is believed that the recommended measures, when applied as a comprehensive mitigation system, would significantly reduce the frequency and severity of accidents on the proposed structure during fog.

NATURE AND EXTENT OF THE FOGGING PROBLEM

Fog decreases visibility and makes driving more difficult and dangerous. Traffic studies indicate that although driving speed is reduced in fog, the probability of overdriving the safe stopping distance is greatly increased. Accident statistics indicate that there is a greater likelihood of multivehicle accidents and fatal accidents in fog. Dangers due to fog are accentuated on Interstate highways, as compared to arterial roadways, because of the increased speeds and greater traffic levels found on Interstates.

Mill-induced and natural fog occurs in the Charleston area throughout the year, particularly during the nominal "fog season," November through March. On a daily basis, natural fog is more likely to be present during the cooler nighttime and early morning hours, which are characterized by relatively stable atmospheric conditions, than during the warmer daytime and early evening hours, when there is greater vertical mixing of air in the atmosphere. The paper mill emits over 6 million gallons of water vapor per day into the atmosphere from some 50 different sources.

By using computerized analytical dispersion models, it was predicted that the combination of natural and mill-induced fog would result in a reduction in visibility to less than 660 ft perhaps 12–15 times per year. That fog is more dense in the vicinity of the Westvaco paper mill was confirmed by the testimony of two Charleston harbor pilots, with over 40 years of combined experience in traversing the branch of the Cooper River near the mill and bridge site. They stated that they had personally experienced denser fog near the paper mill about 12–15 times per year (*1*). Although visibility within either type of fog can be significantly reduced, observations of the water vapor plumes emitted from the paper mill indicate that depending on wind speed and direction and the vertical temperature profile of the atmosphere, mill-induced fog can appear and disappear very rapidly at a particular location as the wind changes speed and direction.

Measures are required to mitigate the potential impact of fogging conditions on the 30,000 vehicles per day that are expected to use the Cooper River Bridge by the year 2000 and, especially, to improve safety conditions for the approximately 1,800 vehicles per hour that will traverse the bridge in the relatively fog-prone early morning peak traffic hour, from 7:00 a.m. to 8:00 a.m. Because of the mild climate in the Charleston area, ice formation was not considered to be a significant problem and therefore was not a subject of this mitigation effort.

RECOMMENDED MITIGATION MEASURES

Available mitigation measures were identified and evaluated on the basis of effectiveness, availability and reliability, cost, and general practicability. Measures evaluated include dispersion systems, design alternatives, guidance systems, and transportation management alternatives. None of the dispersion methods were determined to be effective in a highway bridge application and therefore are not included in the recommended fog mitigation plan. Similarly, bridge design alternatives would have required both significant project delays and potential redesign of the bridge, and therefore these alternatives were not considered to be feasible mitigation measures.

The principal criteria used in the selection of mitigation measures was whether the proposed method has been shown to increase safety in similar highway applications during fog events. The proven ability to reduce impacts, rather than the ability of a measure to completely eliminate fog or its impacts, was the realistic goal of this study because there are no methods that can eliminate the chance of an accident occurring during fog. The goal of the selected fog mitigation plan was to reduce the probability and severity of accidents in fog to levels approaching or equal to those without fog.

The recommended mitigation plan does not include measures to reduce the frequency, duration, and intensity of the fogging events, such as applying controls to limit vapor released at the Westvaco paper mill. Instead, the measures identified were selected to provide effective assistance to drivers encountering intense fog conditions on the Cooper River Bridge.

The major elements of the recommended fog mitigation plan included the following:

- *Fixed permanent single message signs indicating that the bridge is prone to fog.*
- *Raised reflective pavement markers to delineate roadway edgelines and lane delineation lines.* National guidelines for the placement and spacing of raised reflective pavement markers have not been formally adopted. The suggested arrangement of raised reflective pavement markers shown in the FHWA *Traffic Control Devices Handbook* (2) indicates that raised reflective pavement markers can be spaced between 20 ft and 40 ft apart when used to supplement pavement striping. During wet or fog conditions, pavement is not always visible, and this loss of visibility occurs when pavement striping is most needed to delineate lanes. Therefore the spacing of raised reflective pavement markers (yellow on the left, white on the right) should be spaced on 20-ft (minimum standard) to 10-ft (desirable standard) centers. The raised reflective pavement markers indicating the lane delineation line should be arranged to simulate standard broken line marking (10-ft marking segments interspersed with 30-ft gaps). Three white raised reflective pavement markers should be installed at the beginning, middle, and end of the 10-ft marking segments. Transverse marking of the shoulder area is recommended to reinforce delineation of the right edge of the travelway and to define the shoulder refuge area. The shoulder marking should be spaced on 40-ft intervals with four to six white raised reflective pavement markers.
- *Lighted pavement markers on ~200-ft centers along roadway edgelines to provide long-range delineation of the roadway beyond the reach of vehicle head lamps.* Lighted pavement markers are recommended to provide daytime guidance during fog conditions and to provide long-range guidance when fog reduces vehicle headlight range so that only nearby raised reflective pavement markers are illuminated. Standard high-intensity unidirectional lighted pavement marker units have been used in roadway application. The unit is 12 in. in diameter

and is available with 3½-in. or 9-in. base, which can be inserted into the bridge deck and the pavement of the approach roadway.

A driver normally views the roadway up to 1,000 ft in front of the vehicle. As driving and visibility conditions worsen, the driver's visibility range will be reduced to 400 ft or less. At a minimum, a driver should be able to see two sets of lighted pavement markers to maintain proper roadway orientation. Placing lighted markers ~200 ft apart will provide the minimum number of markers required to maintain proper roadway orientation (2). A review of the engneering plans for the proposed bridge indicates that a spacing of ~200 ft can be maintained throughout the length of the bridge, including the 1,600-ft vertical curve main spans of the bridge.

- *Increased highway surveillance by highway troopers and the installation of a closed circuit television system to provide timely detection of fog conditions.* Television cameras permanently mounted on the bridge truss would be directed toward a series of targets (e.g., simulations of tail lights) placed at predetermined incremental distances along the portion of the bridge prone to mill-induced fog. The signals from the cameras would be transmitted to dedicated television monitors at the local Highway Trooper District office. The number of targets visible on the monitor would provide an estimate of the degree of visibility on the bridge directly proportional to site distance. State highway troopers will provide increased on-site surveillance during the nominal November through March fog period and, more particularly, during periods when meteorological conditions are expected to cause fumigation of the proposed bridge by mill-generated vapor. Such a determination could be made by a qualified, certified meteorologist experienced in weather forcasting.

On the basis of the results of increased surveillance by state highway troopers or estimates of sight distance determined from the dedicated television monitors, additional response procedures would be implemented. These measured include activation of lighted pavement markers, activation of illuminated variable message display units to provide advance warning and instructions to drivers concerning upcoming fog conditions, and the implementation of transportation management measures. The specific techniques to be applied would depend on the severity of the observed fog conditions. Such techniques would be applied on the basis of a predetermined response agenda.

- *Overhead sign bridges with internally illuminated variable message display units to provide specific fog incident information to the driver.* The overhead sign bridges should be located outside of each end of the fog-prone area to warn motorists of existing or potentially hazardous conditions. The location of these bridges should be such that traffic could divert to alternative roadways. The location of these bridges in relation to the roadway alignment should be tested with procedures recommended in Section 2C-3, Placement of Warning Signs, of the *Manual on Uniform Traffic Control Devices* (3). These variable message warning signs would be placed on the approaches to the bridge to provide advance warning and instruction to drivers about potential fog conditions on the bridge, specifying lower speed limits during fog conditions and providing motorists with additional directives and information on driving conditions and required actions.

The measures recommended in this study make up a system of interrelated guidance and transportation techniques that function both independently and in association to improve safety conditions on the Cooper River Bridge during fog. The basic level of protection in the system consists of two permanent passive measures: permanent single-message warning signs and raised reflective pavement markers. These are the simplest and least specialized components of the system and would function regardless of the state of any other system component.

The second area of protection within the system is surveillance. Both manned (highway trooper) and remote (television monitor) elements are recommended for detection of hazardous conditions and accidents on the bridge. This redundancy is intended to increase the reliability of this portion of the fog protection system, which is used to activate other key system components.

The third level of the system is the enhanced guidance component. This consists of the lighted pavement markers, which will be used to supplement the raised reflective pavement markers during fog events.

The fourth element of the system, the variable message signs, would provide motorists with warnings and information relevant to the specific fog event. The messages given by such a system could include a broad range of information, including advanced warning of fog conditions, speed regulations, required diversions, and information on other required traffic management controls. Such traffic management controls could include total diversion of traffic from the bridge.

The interrelationship and redundancy of the different elements of the system give the system a built-in resiliency. Failure of individual components of the system (because of electric power interruption, for instance) would still leave basic components of the system intact (raised reflective markers, fixed panel signs, and increased surveillance by highway troopers). An emergency diversion plan would remain an option under these circumstances, should conditions warrant it.

OTHER CONSIDERATIONS

The 4-year bridge construction period provides an opportunity for SCDHPT to refine its selected mitigation measures and study the fog situation further. Given the time available, the following measures were proposed for consideration by SCDHPT.

Monitoring

While the bridge is under construction, there is an opportunity to collect additional data along the actual elevated roadway segment on the frequency, duration, and extent of fogging. Suitable meteorological and automatic fog-detecting devices would be used. This information can be used to better define the geographic and temporal extent of the fogging problem, allowing for better definition and specification of the mitigation program. Monitoring of fog conditions during the construction phase could be used to identify the specific locations where proposed mitigation measures are to be applied, including the location of closed circuit television cameras, lighted pavement markers, and variable message signs.

Lighting Study

Fixed area lighting is not currently planned for the Cooper River Bridge, although the bridge design would not prohibit the implementation of such lighting. A detailed study of lighting on the bridge and approach sections of the highway may ultimately yield a cost-effective lighting system for the bridge segment under fog conditions. There are myriad lighting systems and system design variations. Recommendations as to the most appropriate lighting plan cannot be provided without careful and systematic study of various lighting programs available for the bridge. Such an evaluation is beyond the scope of this study but could be initiated during the 4-year bridge construction period to identify an effective system for use during fog conditions.

Fog Mitigation Update

The identification of measures to safeguard motorists during fog conditions is an ongoing process of research and development. A continuing program of literature review and research on fog mitigation would allow SCDHPT to remain current on fog-related safety programs for roadways. A periodic survey of the literature and communication with other state highway officials may identify additional measures to minimize the effects of fog on safety conditions.

Advanced Detection Techniques

The use of automatic fog detectors (e.g., visiometers and back-scatter equipment), high-resolution closed circuit television cameras, or other electronic devices currently under development could potentially prove to be an effective complement to the proposed mitigation measures. To be of value, such equipment must be reliable and proven in similar applications. Further detailed study of these devices could potentially identify reliable and proven equipment that could be used to augment the proposed mitigation measures, particularly in the areas of system activation and incident detection.

Response Agenda and Protocol

The individual measures to mitigate the impact of fog on safety identified in this study must be applied in a systematic, predetermined, and coordinated fog response system. Elements of such a system include identification of the specific responsibilities, protocols, and agenda for activating and implementing the various mitigation measures, as well as for informing the public as appropriate. This detailed response agenda and protocol must be documented, and responsible individuals must be trained in the various elements of the mitigation system.

REFERENCES

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Optimization of Post Delineator Placement from a Visibility Point of View

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An analytical computer optimization of the height, spacing, and lateral offset of post delineators for tangent sections and horizontal curves on two-lane and four-lane highways was developed in this study, and a small-scale field demonstration and evaluation were performed. The analytical optimization was based solely on visibility considerations, or more precisely, on a driver's reception of 60 multiples of threshold illumination from the fourth post delineator reflector ahead of the automobile. It was concluded that post delineators with 18 in.^{2} of encapsulated lens sheeting material with a specific intensity of 309 cd/fc/ft^2 (−4-degree entrance, 0.2-degree observation angle) should be placed every 275 ft along tangent sections of four-lane divided highways, whereas post delineators with prismatic sheeting material with specific intensities of 825 and 1,483 cd/fc/ft^2 (−4-degree entrance, 0.2-degree observation angle) should be placed every 350 and 400 ft, respectively. Mathematical relationships are presented from which optimal spacings can be calculated for curves of various radii on two-lane and four-lane highways. Height and lateral offset effects on visual detection for the placements investigated are negligible. The use of silver prismatic retroreflectors measuring 18 × 1 in. on the front, a red prismatic retroreflector of the same size on the back side of post delineators near intersections on four-lane divided highways (as a wrong way indicator), and two black diagonal bands for contrast enhancement in snow is recommended.

The *Manual on Uniform Traffic Control Devices* (*1*) defines road delineators as retroreflecting devices mounted at the side of the roadway to indicate road alignment. These delineators have an advantage over many other forms of delineation, such as pavement markings, in that they remain visible when the roadway is wet or snow covered. Guidelines for the installation of these delineators are given in sections 3D-2 through 3D-5 of the manual (*1*). However, many of these guidelines are very vague. For example, according to Section 3D-5, the delineators should be spaced such that the lateral distance between each post is from 200 to 528 ft on tangent sections of highway, and Table III-1 gives spacings for curve sections of highway with radii from 50 to 1,000 ft. There is no discussion of when specific spacings between 200 and 528 ft should be used along tangent sections of highways, and the manual does not address curves with radii greater than 1,000 ft.

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In an attempt to make the placement of post delineators consistent within the state of Ohio, the Ohio Department of Transportation (ODOT) clarifies a few of these federal guidelines in sections AS 4C-5, AS 4C-7, and AS 4C-8 of the Ohio Department of Transportation's *Traffic Control Application Standards Manual* (*2*) and in section 4B-5 and figures CD-5, CD-6, CD-8, CD-9, and CD-11 of the *Ohio Manual of Uniform Traffic Control Devices* (*3*). According to these manuals, the top of the retroreflective sheeting patch, with dimensions of 3 × 6 in., should be 48 in. plus or minus 1 in. above the pavement surface. For tangent sections of four-lane divided highways, these post delineators are to be spaced 400 ft apart at a maximum of 12 ft, 6 in. from the edge of the pavement such that their placement is uniform over the entire section of the highway. These manuals present tables of recommended post delineator spacings for curves with radii from 50 to 1,000 ft that are identical to the one shown in the *Manual on Uniform Traffic Control Devices* (*1*, Table III-1). Once again, there is no mention of curves with radii of greater than 1,000 ft.

A literature review failed to reveal any studies that have addressed, either on an experimental or analytical level, the effects from a visibility or driver performance point of view of the spacing of post delineators. In addition, there appears to be no technical information available that would enable justification of the 400-ft spacing for tangent sections or trade-offs to be made between post delineator height, lateral offset, spacing, retroreflector dimensions, and retroreflector photometric performance. There is therefore a need for studies such as the one reported by Zwahlen (*4*), in which the spacing and placement of snowplowable raised reflective pavement markers (RRPMs) are recommended with respect to a driver's visual information needs, capabilities, and limitations. Such an optimal placement of post delineators not only may lead to a placement scheme that would result in adequate driver performance and safety but also might minimize the relatively high life cycle cost of the post delineators.

The objectives of this study were

- To use a computer model to evaluate analytically, from a visual detection point of view, the reflective performance of post delineators as a function of height, retroreflector dimensions, photometric performance, lateral offset, and spacing;
- To conduct a small-scale field demonstration with ODOT and FHWA personnel to evaluate various retroreflector patch configurations; and

• To recommend to ODOT a set of specifications for post delineator height, retroreflector dimensions, lateral offset, and spacing.

GENERAL OPTIMIZATION APPROACH

The analytical optimization approach was aided by the use of a program developed for an Apple Macintosh microcomputer. This program allows the user to vary the headlamp beam type, atmospheric transmissivity, vehicle-driver dimensions, lateral offset of the post, and post height, as well as the type and dimensions of the retroreflector. Once the levels for each of the variables have been chosen, the program calculates the amount of light that is reflected back to the driver's eyes from each of the vehicle's headlamps for various retroreflector spacings. This program is based on Allard's law (*5*), which is used to calculate the illuminance of an object as long as the background luminance is small compared to the average luminance of the light source.

To provide a framework in which the very small illumination values in footcandles (fc) at a driver's eyes can be compared on a one-to-one basis with visual backgrounds that have different luminance levels and to obtain numbers tied to human detection performance, the final results of the photometric calculations are expressed as multiples of threshold (number of times above the illumination threshold for 98 percent plus detection of a white point source against a uniform background in the laboratory). To relate the illumination at a driver's eyes to multiples of threshold (MOT), a luminance level of the dark road background must be assumed. This level can then be related to a 98 percent threshold detection illumination value for a white point source. The multiple of threshold concept has been discussed by Zwahlen (*6*).

A MOT value that will be acceptable in the detection of post delineators must be selected. Because a driver sees the post delineator that is closest to the vehicle rather clearly most of the time and has a good indication of where the second, third, and possibly fourth post delineator will be in the visual field, the minimum values for the multiples of threshold need not be as high as those that a driver needs to detect a single unexpected point source. This situation could require a multiples of threshold value of up to 1,000 to assure timely detection [a MOT of 1,000 corresponds to a human brilliancy rating between satisfactory and bright, according to Breckenridge (*7*)]. The study dealing with the optimal placement of snowplowable raised reflective pavement markers, referred to earlier (*4*), used a MOT value of 30 as an acceptable value for the detection of the fourth raised reflective pavement marker ahead of the automobile. However, for post delineators a higher MOT value should be used for the following reasons:

• Within a driver's visibility range, post delineators are located farther away from the driver in comparison with RRPMs, and because the highway geometry is more likely to change over the longer distance, the driver is less able to predict the location of the next post delineator.

• The post delineator may be located in the driver's peripheral visual field, where the detection sensitivity is reduced.

• The retroreflectors on post delineators are ~3–4 ft above the pavement surface, where they may blend in with other point sources in the background near the horizon; they are therefore not as conspicuous as RRPMs.

• Post delineators must be able to provide drivers with guidance in snow conditions, where most other delineation elements are no longer visible. The post delineators therefore constitute somewhat of a last defense.

• On the basis of these reasons, a MOT value of 60 (17.1×10^{-8} fc or 1.84 km candles), which, according to Breckenridge (*7*), corresponds to a human brilliancy rating between faint (0.9 km candles) and weak (4 km candles), was used as the minimum level for detection of the fourth post delineator. According to Kaufmann (*5*, p. 3-25) the retroreflective sheeting patches can be assumed to be point sources beyond distances of 500 ft for a 3 × 6–in. patch, beyond 1,000 ft for a 12 × 1.5–in. patch, and beyond 1,500 ft for an 18 × 1–in. patch. If the MOT level for shorter distances is to be calculated, a correction factor must be incorporated into the calculations. As an example of the effect of this correction factor, a distance of 250 ft for a 3 × 6–in. patch, 500 ft for a 12 × 1.5-in. patch, or 750 ft for an 18 × 1–in. patch would require about a 5 percent reduction in the MOT value. The fourth post delineator is usually located at a calculated distance of 1,100 to 1,650 ft.

In optimizing the spacings of the post delineators, it was necessary to consider the following restrictions:

• On a straight and level highway the minimum number of post delineators visible to a driver should be at least four to ensure that the driver has a comfortable preview time and that a change in direction of the road (left or right curve ahead) will be detected in a timely manner. The post delineators should also provide some visual lateral position control cues. In case an occasional post delineator is missing or has lost nearly all of its reflectivity, a driver would still see three post delineators, which should be enough for perception of the approximate course of the road ahead.

• A straight and level Interstate highway with a lane width of 12 ft is assumed. It is further assumed that an automobile would be driven exactly in the middle of the right-hand lane.

• A uniform dark background with a luminance value of 0.01 foot-Lambert (fL) has been assumed (clear, moonlight, lower end of night driving range), which corresponds to an illumination threshold value (for 98 percent detection of a white point source in the laboratory) of 0.28493×10^{-8} fc.

• The headlamps and the silver retroreflectors on the post delineators are clean and operating at the prescribed output (100 percent), and the windshield is also assumed to be clean, with a transmission factor of 1. It was decided that instead of using arbitrarily degraded transmission and efficiency factors for the windshield, the retroreflectors, and the beams, the selection of a minimum acceptable MOT value of 60 would take some of the possible headlamp, windshield, and retroreflector deficiencies, as well as some background variations and driver deficiencies due to information processing load, age, and so on, into account.

• The headlamps of the vehicle are assumed to be properly aimed (i.e., approximately 2 degrees to the right and approximately 2 degrees down for the low beams).

• The vehicle operator is assumed to be seated fairly erectly in the driver's seat, which is assumed to be on the left side of the vehicle.

The independent variables, which were investigated for tangent sections of four-lane highways, left and right curve sections of four-lane highways, and left and right curve sections of two-lane highways, include

- The type of retroreflector,
- The height of the retroreflective patch (measured from the surface of the road to the top of the retroreflective patch),
- The lateral offset (measured from the edge of the highway to the center of the retroreflective patch),
- The dimensions of the retroreflective patch, and
- The longitudinal spacing of the post delineators.

On the curve sections of four-lane divided highways it is assumed that the post delineators are always on the right side of the two lanes regardless of whether the highway curves to the left or right. In this case, as for the tangent sections of four-lane highway, the post delineators are offset 12 ft from the right-hand edge of the highway. The spacing of post delineators along curved sections of four-lane highways is not explicitly addressed by the *Ohio Manual of Uniform Traffic Control Devices* (*3*). On curve sections of two-lane rural highways it is assumed that bidirectional post delineators are placed on the outside edge of the curve only. In this situation, the post delineators (set 2 ft from the edge of the roadway) would be 8 ft to the right of the longitudinal center of the vehicle for a left curve and 20 ft to the left of the longitudinal center of the vehicle for a right curve on a highway with an assumed lane width of 12 ft.

Because the low beams of the vehicle are aimed approximately 2 degrees to the right horizontally, the post delineator spacings on right- and left-hand curves must be analyzed separately. Therefore the computer optimization calculations were carried out for four different curve situations: first for left- and right-hand curves on four-lane divided highways and second for left- and right-hand curves on two-lane rural roads.

COMPUTER OPTIMIZATION RESULTS

During the initial stages of this study the effect of variables that were not directly related to the post delineators was studied, including the vehicle-driver dimensions, the type of headlamp used, and the transmissivity of the atmosphere. A more thorough discussion of the entire optimization study has been presented by Zwahlen (*8*).

Three sets of vehicle-driver dimensions were investigated: those of a 95th percentile man in a typical semitractor, a 50th percentile person in a typical large automobile, and a 5th percentile woman in a typical small automobile. The results indicate that on tangent sections of highways when post delineators are installed according to the guidelines established by ODOT, the delineation conditions favor the 5th percentile woman in a typical small automobile and are somewhat less favorable for the 95th percentile man in a semitractor.

Three common types of headlamps were also investigated, the halogen 6054 high beam, the halogen 6054 low beam, and the 6052 low beam. Of these three headlamp beam types the 6052 low beam provided the least favorable illumination conditions. Therefore, to make the results applicable to a high percentage of the driving population, the results presented in this paper will apply to a 95th percentile man in a typical semitractor equipped with 6052 low beams. The study also included two levels of transmissivity, 0.99 per 100 ft (relatively clear conditions) and 0.8946 per 100 ft (1 in. of rainfall per hour or light fog). Although lower levels of illumination were present for the 0.8946 level of transmissivity, illumination levels for the 0.99 level of transmissivity are presented throughout the results because this is the more common condition, and it is felt that the drivers will adjust their driving strategies and speed during conditions of heavy rain, snow, or fog.

During the computer optimization, two types of retroreflective materials were studied. These include encapsulated lens sheeting material (such as 3M high intensity) with a specific intensity of 309 candela per footcandle per square foot (cd/fc/ft^2) and prismatic sheeting material (such as Reflexite A/C 1000) with a specific intensity of 825 cd/fc/ft^2 at a 0.2-degree observation angle and a −4-degree entrance angle. During the study, actual measurements of the prismatic sheeting materials in the field indicated that they had an average specific intensity of 1,483 cd/fc/ft^2 at a 0.2-degree observation angle and a −4-degree entrance angle; therefore many of the theoretical results also include numbers for prismatic materials with this specific intensity.

As shown in Figure 1, which shows MOT values at various distances for prismatic sheeting with a specific intensity of 825 cd/fc/ft^2 and encapsulated lens sheeting with a specific intensity of 309 cd/fc/ft^2, the minimum selected 60 MOT value for the prismatic sheeting material (825 cd/fc/ft^2) corresponds to a detection distance of ~1,390 ft or a spacing of ~348 ft for the 100 percent (ideal) efficiencies and windshield transmittance stated in the assumptions. For the encapsulated lens sheeting material the 60 MOT value is reached at ~1,090 ft, which would correspond to a spacing of ~273 ft for the 100 percent (ideal) efficiencies and windshield transmittance stated in the assumptions. If the spacing for encapsulated and prismatic sheeting materials were calculated with the assumption of a windshield transmittance of 0.7 and a retroreflector efficiency of 90 percent, then these distances would be about 86 to 88 percent of the distances calculated under the ideal conditions. Because it would be possible to place post delineators with prismatic sheeting material further apart than post delineators with encapsulated lens sheeting material while still fulfilling the minimum selected MOT value, it would seem that the prismatic sheeting is superior to the encapsulated lens sheeting material for post delineator applications.

Three retroreflector heights were evaluated (34, 40, and 46 in., measured from the surface of the road to the top of the retroreflective patch), and the MOT values were plotted against detection distance for both tangent (straight) and curve (2.6-degree curvature, radius of 2,200 ft) sections of highway. Figure 2 shows that at any given distance the MOT value is slightly higher for the 34-in. post delineator height than it is for the 40- or 46-in. post delineator height on both tangent and curved sections of highway. It should also be noted that as the retroreflective patches are mounted closer to the ground, shorter delineator posts, which require less material, may be used (a 40-in. reflector height with the reflector mounted 2 in. below the top of the delineator post will use about 8 percent less material than the 46-in. retroreflector height that is currently used); however, other considerations, such as guardrail height, grass growth, snow, dirt and spray accumulation, make it inadvisable to decrease the retroreflector height below ~40 in.

Lateral offset values of 10, 12, and 14 ft (measured from the edge of the road) were investigated. Figure 3 shows the MOT level at various distances for each of these three lateral offsets and for straight and curved sections (2.6-degree curvature, radius of 2,200 ft) highway. It can be seen that there is practically no difference between the three lateral offset distances until the distance to the fourth delineator post is less than 400 ft. However, the MOT values for distances less than 400 ft are

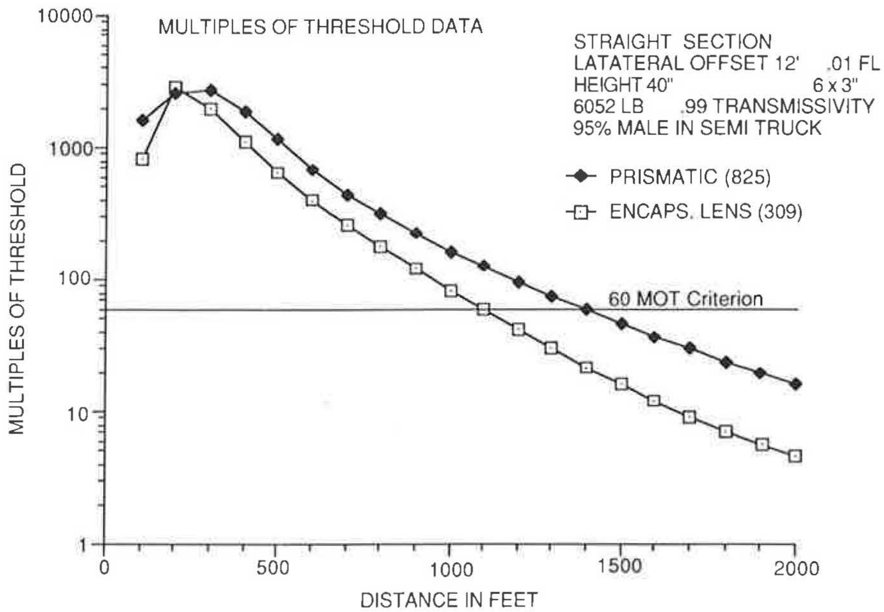

FIGURE 1 MOT versus distance for prismatic and encapsulated lens sheeting materials.

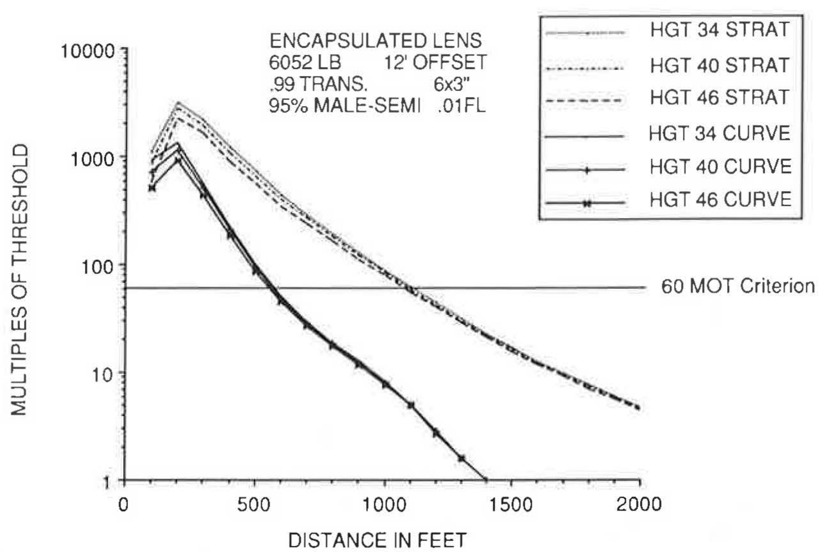

FIGURE 2 MOT versus distance for variable height and road geometry.

greater than 200, and therefore any differences are of no great practical significance because the retroreflectors on the post delineators should be clearly distinguishable under all conditions.

It might be noted that because lateral offset does not seem to influence the illuminance level at the driver's eyes near the 60 MOT line, the spacing optimization that is carried out for four-lane divided highways might be generalized to include tangent sections of two-lane highways as well because the only difference between the conditions for the two types of roadways would be the lateral offset.

The photometric effect of changing the current dimensions of the retroreflective patch (3 in. in width by 6 in. in length) to a patch 1.5 in. wide and 12 in. long was investigated. This patch size allowed the use of the same amount of reflective area (18 in.^{2}) as well as posts of equal height. The calculations were done by dividing each retroreflector into four equally wide, independent horizontal, rectangular patches and determining the illumination at a driver's eyes for each separate patch. Thesums of the illuminations from these four equally wide patches, each with a different vertical centroid height above the road surface, were then plotted in Figure 4. It should be noted that although there is no practical difference in the MOT values at corresponding detection distances for the two different retroreflector patches from an illumination or photometric point of view, there may well exist a difference from a driver's perceptual point of view (see the results of the field evaluation).

An evaluation of the spacings that have been or are presently being used by ODOT in conjunction with encapsulated lens sheeting material was made, with interesting results. Post delineator spacings of 400 and 528 ft were evaluated with a post delineator height of 40 in. (measured from the highway's surface to the top of the retroreflective patch), a 3 × 6–in. encapsulated lens retroreflective patch, 6052 low beams, lateral offset of 12 ft, and vehicle-driver dimensions for a 95th percentile man in a semitractor. The MOT for the first four retroreflectors

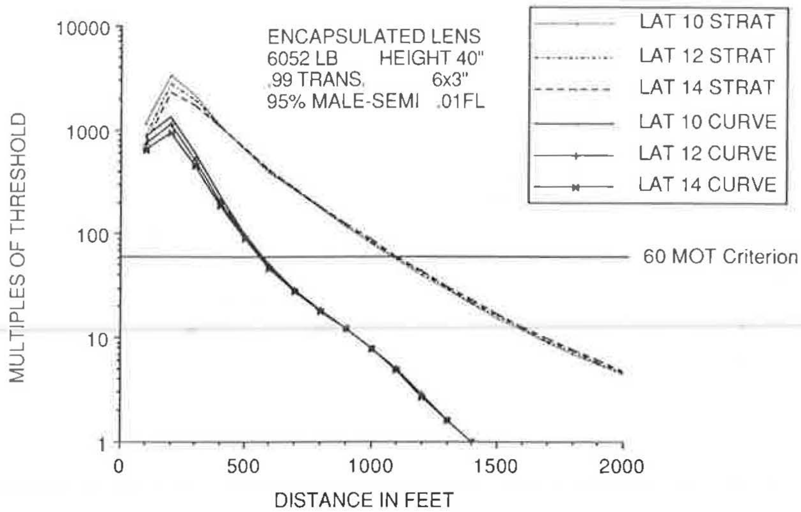

FIGURE 3 MOT versus distance for variable offset and road geometry.

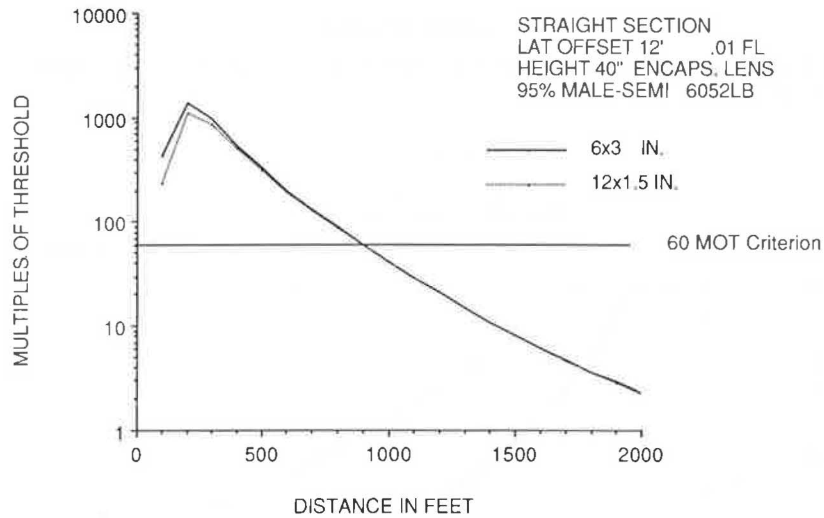

FIGURE 4 MOT versus distance for variable patch dimensions.

were then calculated for each of these two post delineator spacings. For a 0.99 transmissivity (clear conditions) and a post delineator spacing of 528 ft, the MOT values were 595 for the first post delineator, 69 for the second, 12.9 for the third, and only 3.6 for the fourth.

With somewhat degraded atmospheric conditions (0.8946 transmissivity), the MOT values show a further decline, with a MOT value of 203 for the first post delineator, 8.1 for the second, 0.5 (not visible under laboratory conditions) for the third, and 0 for the fourth post delineator. If the 400-ft spacing is evaluated in the same way, using a transmissivity of 0.99, the MOT value is 1,159 for the first post delineator, 185 for the second, 42.2 for the third, and 12.4 for the fourth. Once again, these values decline further for the degraded weather conditions (0.8946 transmissivity), with a MOT value of 511 for the first post delineator, 36.3 for the second, 3.7 for the third, and 0.5 (not visible under laboratory conditions) for the fourth. The use of this retroreflective encapsulated lens sheeting material for a post delineator spacing of 528 or 400 ft clearly does not fulfill the chosen minimum MOT value of 60 and therefore is probably unsatisfactory from a driver visibility or detection point of view.

If the post delineators equipped with encapsulated lens sheeting material were to fulfill the minimum MOT value of 60, the posts could be no farther than 273 ft apart along tangent sections of highway for a transmissivity of 0.99, 6502 low beams, a 95th percentile man in a typical semitractor, lateral offset of 12 ft, retroreflector height of 40 in., and a 3 × 6–in. patch. If the post delineators were equipped with prismatic sheeting material with a specific intensity of 825 cd/fc/ft^2, then the retroreflectors could be placed 348 ft apart, and if they were equipped with prismatic sheeting material with a specific intensity of 1,483 cd/fc/ft^2, then they could be placed 413 ft apart and still satisfy the minimum value of 60 MOT for a transmissivity of 0.99 per 100 ft.

Table 1 and Figure 5 show post delineator spacings at which a MOT value of 60 is obtained at the fourth post delineator for both left and right curves on four-lane highways for various curve radii and for three different retroreflective intensities (309, 825, and 1,483 cd/fc/ft^2 at a 0.2-degree observation angle and −4-degree entrance angle). Detection distance values could not be obtained for radii below 550 ft due to lack of beam pattern data for slightly positive vertical beam angles, combined with large horizontal beam angles, and collection of such

TABLE 1 SPACINGS FOR RIGHT AND LEFT HORIZONTAL CURVES

Curve Radius (ft)	Retroreflector[a]	Right Curve Spacing (ft)	Left Curve Spacing (ft)
3,000	EL	165	135
	P1	195	160
	P2	221	188
2,200	EL	145	125
	P1	170	155
	P2	199	173
1,800	EL	130	115
	P1	160	141
	P2	190	162
1,400	EL	120	110
	P1	149	128
	P2	172	150
1,000	EL	110	100
	P1	128	113
	P2	141	130
750	EL	95	90
	P1	108	100
550	EL	80	85

NOTE: Recommendations for four-lane divided highways, using encapsulated lens and prismatic sheeting materials as retroreflectors, vehicle-driven dimensions of a 95th percentile man driving a typical semitractor, and 0.99 per 100 ft transmissivity. No data are available for curve radii smaller than 550 ft due to lack of available candle power values for large horizontal angles and slightly positive vertical beam angles.

[a]EL = encapsulated lens sheeting (silver), assumed specific intensity 309 cd/fc/ft^2; P1 = prismatic sheeting (silver), assumed specific intensity 825 cd/fc/ft^2; and P2 = prismatic sheeting (silver), measured specific intensity in field 1,483 cd/fc/ft^2.

photometric data was beyond the scope of this study. In Table 1 and Figure 5 it can be observed that with the exception of radii below 550 ft, the post delineator spacings for left curves are significantly shorter than those for post delineators for right curves, due to the geometric interaction of the driver-vehicle-retroreflector dimensions and the aim of the low beam pattern.

Functional relationships for the spacing of post delineators on curves in four-lane highways with various radii were obtained for the three different retroreflective intensities. The main objective when formulating these relationships was to obtain a relatively good fit over the radius range from 550 to 2,200 ft by fitting the curve to the 60 MOT detection distances (for the fourth delineator post location) for radii of 550, 750, 1,000, 1,400, 1,800, 2,200, and 3,000 ft for left and right curves. However, no photometric data from the computer model exist to allow justification of the use of this relationship for spacings along curves with radii of less than 550 ft, therefore an arbitrary minimum spacing of 20 ft for a 50-ft-radius curve, which is presented in both the *Manual on Uniform Traffic Control Devices* (*1*) and the *Ohio Manual of Uniform Traffic Control Devices* (*3*), was used.

The relationships that exist between post delineator spacings and curve radii not only are different for different retroreflective materials but also are different for left and right curves. However, to eliminate confusion that may result from the application of the different relationships for post delineator spacings along left and right curves, the relationships given for four-lane highways have been fitted for each type of retroreflective material so that they are adequate regardless of the direction of the curve. These relationships are as follows: $9.8\ (R-40)^{1/3}$ for encapsulated lens sheeting (309 cd/fc/ft^2), $11.5\ (R-45)^{1/3}$ for prismatic sheeting material (825 cd/fc/ft^2), and $13.5\ (R-47)^{1/3}$ for prismatic sheeting (1,483 cd/fc/ft^2). These relationships can then be applied from a 50-ft radius (20-ft spacing) up to a radius value that provides spacings of 275 ft for encapsulated lens sheeting, 350 ft for prismatic sheeting at 825 cd/fc/ft^2, or 400 ft for prismatic sheeting at 1,483 cd/fc/ft^2. The derived constants are to be considered tentative at this point and will change somewhat if other specific intensity values are used. However, the basic functional relationship of using a multiplicative constant to multiply the cube root of a difference appears to be quite robust.

To test the accuracy of these relationships, the computer model was used to perform photometric calculations for a left curve with a radius of 10,000 ft. These calculations yield a post delineator spacing of 211 ft (201 ft using the appropriate formula given earlier) for the encapsulated lens sheeting, 227 ft (247 ft using the appropriate formula given earlier) for the prismatic sheeting at 825 cd/fc/ft^2, and 257 ft (290 ft using the appropriate formula) for the prismatic lens sheeting at 1,483 cd/fc/ft^2. When these calculated post delineator spacings for a radius of 10,000 ft (0.6-degree curve) are compared with the distances obtained through the mathematical relationships, an error of only 5 percent is observed for the encapsulated lens sheeting, 9 percent for the prismatic sheeting at 825 cd/fc/ft^2, and 13 percent for the prismatic sheeting at 1,483 cd/fc/ft^2. Thus for a 10,000-ft-radius left curve, the difference is fairly small and is acceptable when it is considered that in deriving the multiplicative constant, the exponent, and the amount to be subtracted from the radius, trade-offs were made that kept the functional relationship fairly simple and minimized the error for curve radii between 550 and 3,000 ft (10.4–1.9 degrees of curvature) for which calculated 60 MOT detection distances were available.

Table 2 shows post delineator spacings where 60 MOT detection distances for the fourth post delineator are obtained for radii ranging from 500 to 2,000 ft, two different transmissivities, and left and right curves on two-lane highways. These spacings are compared with recommended spacings from Figure CD-5 of the *Ohio Manual of Uniform Traffic Control Devices* (*3*) in Figures 6 and 7. It should be noted that the spacings recommended by ODOT for curve radii greater than 1,000 ft are calculated from a formula given in the footnote of Figure CD-5, which was to be used to calculate spacings of post delineators leading into and out of the curve. In Table 2 of this paper, it can be observed that in all cases the post delineator spacings for the left curves are significantly lower than those for the right curves. In fact, in Figure 6, which shows spacings for a right curve, it can be seen that the spacings recommended by ODOT are much shorter than the spacings required to obtain an illuminance level at a driver's eyes of 60 MOT for any of the three types of material. However, if the spacings for the left curve in Figure 7 are examined, it can be seen that the spacings recommended by ODOT would not satisfy the 60 MOT value for the encapsulated lens material on curves with radii larger than 1,300 ft.

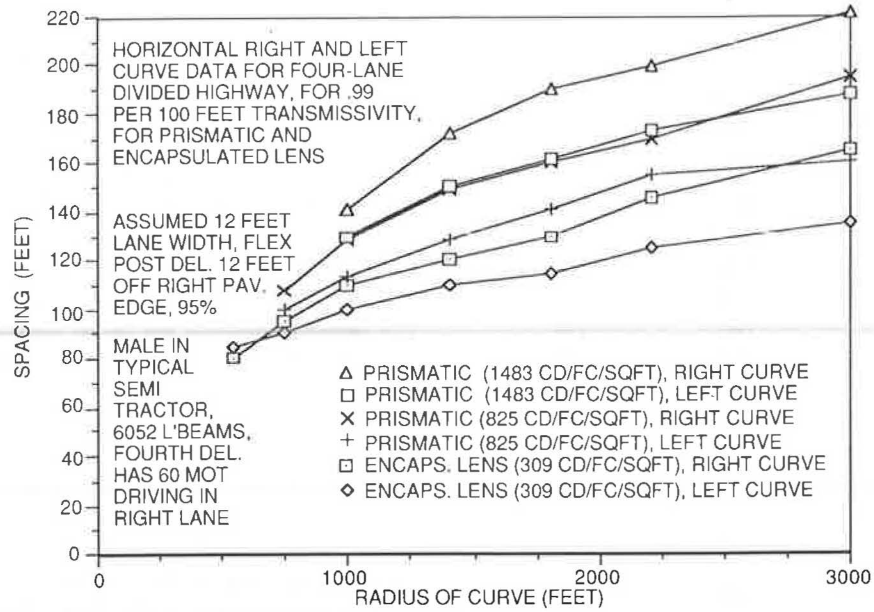

FIGURE 5 Computed optimal spacings for flexible post delineators for right and left horizontal curves on four-lane divided highways.

TABLE 2 FLEXIBLE POST DELINEATOR SPACINGS FOR LEFT AND RIGHT CURVES ON TWO-LANE RURAL ROADS

		Retroreflector Transmissivity[a]					
Radius (ft)	Curve	EL 0.99	EL 0.8946	P1 0.99	P1 0.8946	P2 0.99	P2 0.8946
500	Right	88		92			
	Left						
600	Right	99	85	103	92	112	100
	Left	81	76				
700	Right	107		113		123	
	Left	87					
800	Right	113		122		133	
	Left	91		101			
900	Right	116		129		143	
	Left	94		107		120	
1,000	Right	121		135		152	
	Left	97		112			
2,000	Right	168	141	185	155	205	165
	Left	116	93	153	108	166	124

[a]EL = encapsulated lens sheeting at 309 cd/fc/ft^2; P1 = prismatic sheeting at 825 cd/fc/ft^2; and P2 = prismatic sheeting at 1,483 cd/fc/ft^2. 0.99 = transmissivity of 0.99 per 100 ft (clear); 0.8946 = transmissivity of 0.8946 per 100 ft (1 in. of rain per hour or light fog).

Because the illumination values are dependent on the curve radius and the type of retroreflective material, it is once again necessary to present mathematical relationships from which post delineator spacings can be calculated for various curve radii. However, it should be noted that a curve on a two-lane highway that is approached as a right curve while traveling in one direction will be approached as a left curve when traveling in the opposing direction. If these post delineators are placed only on the outside edge of the curve, the same delineators (bidirectional) will be used by motorists approaching the curve from either direction. To provide a satisfactory visual stimulus to a driver who approaches the curve from either direction, it is necessary to use the mathematical relationships that yield the shortest spacings.

The functional relationships for the spacing of post delineators on curves along two-lane rural highways were obtained in the same manner as for the four-lane divided highways. These relationships were based on a minimum arbitrary spacing of 20 ft for a 50-ft-radius (114 degrees of curvature) curve and 60 MOT detection distances (at the fourth post delineator location) obtained for left curves for the radii shown in Table 2. For all curves on two-lane highways the mathematical relationships are $10\ (R\text{-}43)^{1/3}$ for encapsulated lens sheeting, $11.5\ (R\text{-}44)^{1/3}$ for prismatic sheeting at 825 cd/fc/ft^2, and $13\ (R\text{-}46)^{1/3}$ cd/fc/ft^2 for prismatic sheeting at 1,483 cd/fc/ft^2.

Photometric calculations that were carried out with the computer model for a curve with a 10,000-ft radius resulted in flexible post delineator spacings of 181 ft for encapsulated sheeting material, 208 ft for prismatic sheeting at 825 cd/fc/ft^2, and 241 ft for prismatic sheeting at 1,483 cd/fc/ft^2. These compare with spacings of 215 ft for encapsulated lens sheeting material, 247 ft for prismatic sheeting material at 825 cd/fc/ft^2, and 280 ft for prismatic sheeting material at 1,483 cd/fc/ft^2 calculated from the mathematical relationships. No photometric data from the computer model are available to justify the use of these mathematical relationships for curve radii of less than 500 ft. Although the differences of 19 percent, 19 percent, and 16 percent that exist between the results from the computer model and the mathematical relationships for a curve radius of 10,000 ft seem rather large, they may be acceptable given that most curves on two-lane rural highways fall within a range of radii from 200 to 2,000 ft [28.6 to 2.8 degrees of curvature; see Zwahlen (*9*)].

TANGENT SECTION FIELD DEMONSTRATION AND EVALUATION RESULTS

A small-scale field demonstration and evaluation was conducted with ODOT and FHWA personnel. The 10 test sections

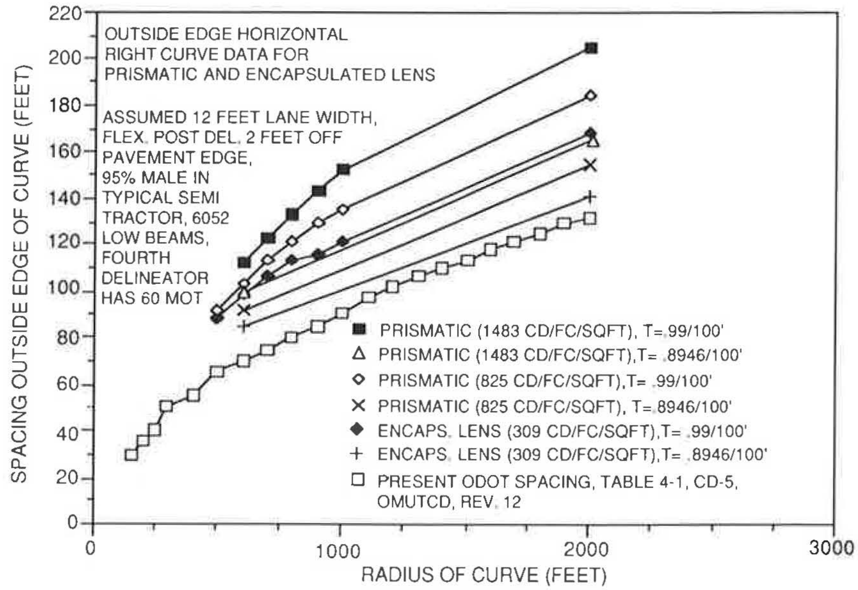

FIGURE 6 Comparison of computed optimal spacings for flexible post delineators for right horizontal curves (on outside edge only) on two-lane rural roads with present ODOT spacings.

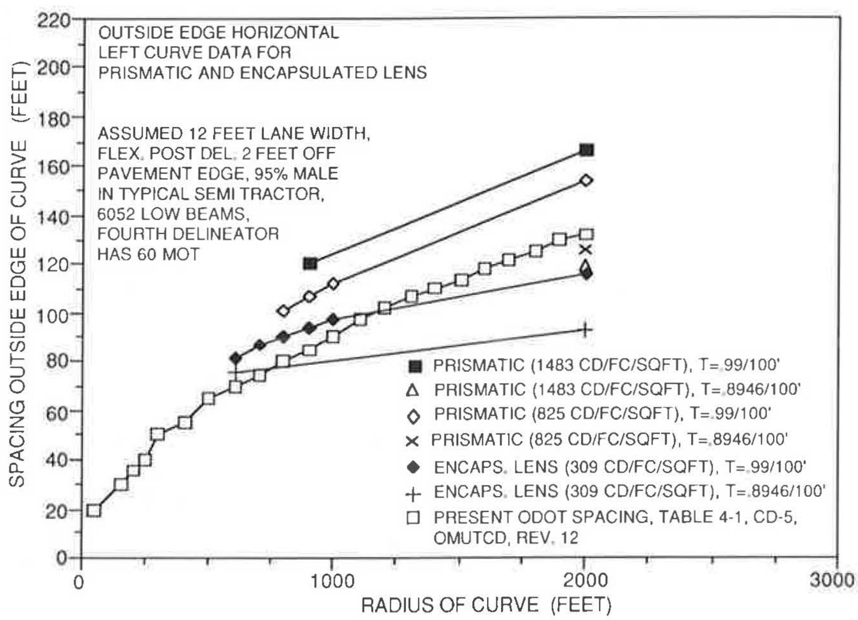

FIGURE 7 Comparison of computed optimal spacings for flexible post delineators for left horizontal curves (on outside edge only) on two-lane rural roads with present ODOT spacings.

were on highways 50 and 32 west of Athens, Ohio, in Athens and Meigs counties. Test section A used post delineators with 6 × 3–in. retroreflectors made of encapsulated lens sheeting material and a longitudinal spacing of 275 ft. Test section B was equipped with post delineators with 12 × 1.5–in. encapsulated lens sheeting material retroreflectors and a longitudinal spacing of 275 ft. Post delineators with 12 × 1.5–in. retroreflectors made of enclosed (embedded, engineering grade) lens sheeting and a spacing of 275 ft were used at test section C, while test section D had post delineators with 6 × 3–in. retroreflectors made of prismatic sheeting material and a spacing of 350 ft. Test sections E and F were equipped and post delineators with an 18 × 1–in. silver prismatic retroreflector and two black 6–in. strips near the top of the post slanted at a 30-degree angle on the front of each post and an 18 × 1–in. red retroreflector on the back. The two black 6–in. strips were slanted away from the road in test section E and toward the road in test section F. The post delineators on test sections E and F were spaced 350 ft apart. The post delineators on test section G had 12 × 1.5–in. retroreflectors made of prismatic sheeting material and were spaced 350 ft apart. Test section H was equipped with post delineators with 12 × 3–in. encapsulated lens retroreflectors and were spaced 350 ft apart. Test section I used post delineators with 24 × 3/4–in. silver prismatic retroreflectors and two black 6-in. strips near the top of the post. The strips were slanted 30 degrees toward the highway on the front. These post delineators were spaced 350 ft apart. Post delineators with 18 × 1–in. prismatic retroreflectors and two black 6-in. strips near the top of the post slanted 30 degrees toward the highway on the front were used on test section J and were spaced 350 ft apart. Each test section contained approximately 10 flexible delineator posts. All posts were 42 in. high (measured from the pavement edge to the top of the retroreflective delineator post) and 12 ft away from the right-hand pavement edge. On the basis of field measurements, the encapsulated lens sheeting had a specific intensity of about 296 cd/fc/ft^2 at a 0.2-degree observation angle and –4-degree entrance angle. The enclosed lens sheeting had a specific intensity of ~95 cd/fc/ft^2 at a 0.2-degree observation angle and –4-degree entrance angle. The prismatic sheeting had a specific intensity of about 1,483 cd/fc/ft^2 at a 0.2-degree observation angle and –4-degree entrance angle.

During the demonstration, 13 ODOT and FHWA employees evaluated the 10 test sections at night. The first evaluation run in a passenger automobile was with low beams, while the second evaluation run was with high beams. For each run, a two-page Post Delineator Rating Form was filled out. The results of this small evaluation indicated that sections E and F (prismatic sheeting, 18 × 1–in. vertical silver strip, 350-ft spacing) were subjectively judged to be the best from the appearance and guidance viewpoint. This type of retroreflector, with dimensions of 18 × 1 in., seems to provide drivers with an excellent shape cue, an excellent distance estimation cue, and an excellent guidance cue. The bottom of this patch is still at least 22 in. above the pavement surface when the top of the patch is 40 in. above the pavement surface and is therefore very functional in the road environment. A vertical strip of 18 × 1 in. would also allow the design of a narrower post that would be subject to less wind stress and might result in an additional post material savings of up to 12 percent.

The least satisfactory post delineator configuration was the enclosed lens material 12 × 1.5–in. vertical silver strip with a 275-ft spacing. The results were very similar for high beams and for low beams. Sections E and F were between two intersections, and the post delineators had a red 18 × 1–in. vertical prismatic sheeting strip on their backs as a means to warn motorists that they were driving in the wrong lane. On the basis of the evaluation results, such a red strip would help considerably, especially during a dark night or a wet dark night or on a snow-covered road, to inform drivers that they were in the wrong lane and going in the wrong direction. In addition, sections E, F, I, and J had post delineators with a black and white pattern (two black diagonal bands, each 6 in. wide, and 6 in. apart, starting 1 in. below the top, slanted at a 30-degree angle toward the road). On the basis of the evaluation results, such a contrast pattern was judged to be useful during the daytime and in snow.

CONCLUSIONS

Post delineator placement and spacing recommendations for tangent sections and left and right curve sections of two-lane and four-lane highways have been derived on the basis of explicit procedures, calculations, and visual detection assumptions. This analytical optimization was used to determine a preferred retroreflector height of 42 in. (measured from the pavement surface to the top of the post). No practical differences were found between lateral offsets of 10, 12, and 14 ft. Therefore, if it is beneficial to place the posts at any given distance between 10 and 14 ft from the pavement edge of the right lane (to minimize damage during repaving, shoulder rehabilitation operations, or snow removal), such a lateral offset is acceptable.

To minimize the number of post delineators per mile, retroreflective sheeting should be made of prismatic material and should measure 18 × 1 in. The top of the sheeting strip should be no more than 2 in. below the top of the post. An 18 × 1–in. red retroreflective prismatic sheeting strip should be placed on the back side of the five post delineators closest to an intersection on both sides of a four-lane highway as a wrong way indicator. If the post delineator is to be placed in a geographic region where snow is common, then a black and white pattern (two black diagonal bands, each 6 in. wide, and 6 in. apart, starting 1 in. below the top of the post, slanted at a 30-degree angle toward the road) should be used to enhance contrast and delineation against a white, snow-covered background.

Along tangent sections of highways the post delineators should have a longitudinal spacing of 275 ft for encapsulated sheeting material with a specific intensity of 309 cd/fc/ft^2, 350 ft for prismatic sheeting material with a specific intensity of 825 cd/fc/ft^2, and 400 ft for prismatic sheeting with a specific intensity of 1,483 cd/fc/ft^2. The recommended spacing of the post delineators along curve sections of a four-lane divided highway and the recommended spacing of the bidirectional post delineators (with 18 × 1–in. sheeting strips on both the front and the back of the delineator post) along curve sections of a two-lane highway should be calculated by using the mathematical relationships that were previously presented for each type of retroreflective sheeting material. These mathematical relationships have the general form of a constant multiplied by the cubed root of the curve radius minus some constant. Because the optimization results of this study are based primarily on visibility calculations and because the recommendations for the shape of the retroreflective strip, the red retroreflective sheeting strip on the back of posts near intersections, and the black diagonal bands for contrast enhancement in snow have not yet been fully evaluated from a driver performance and safety point of view, it is recommended that further, more comprehensive field validation studies be conducted before these recommendations are implemented.

ACKNOWLEDGMENT

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Effect of Bridge Lighting on Nighttime Traffic Safety

MICHAEL S. JANOFF

Extensive research has indicated the existence of adverse neurological effects of flickering or rhythmic lights, referred to as photic stimulation. Although regularly flickering light rarely exists for more than a few seconds in the natural environment, some types of artificial lighting, such as that found on long bridges or in long tunnels, can provide a photic stimulation for a sufficient time (i.e., minutes) to cause such adverse effects. These effects, theoretically, could result in an increase in nighttime traffic accidents. About 20 years ago, a few highway agencies began to design and install low-mounted, lineal-type lighting systems—almost all of which produce a flicker—on a number of bridges under the assumption that such lighting was an improvement over conventional lighting systems. This assumption was never validated. One such installation was the San Mateo Bridge over San Francisco Bay. Theoretically, the length of this bridge should provide sufficient exposure to this lighting to induce the adverse neurological effects. The availability of detailed accident records for the San Mateo Bridge provided a basis for determining whether the nighttime safety on this bridge was in any way related to the amount of time a motorist was exposed to this lighting system. The analyses of the accident data have shown that the nighttime safety on the bridge has been adversely affected by the lineal lighting, that the effect occurs after only about 3 min exposure to the lineal lighting, and that replacement of this type of lighting with conventional overhead lighting promotes a safer nighttime traffic environment.

Laboratory research as early as the 1930s revealed the existence of adverse neurological effects of flickering or rhythmic lights (photic stimulation). Such effects can include somatic, mental, and emotional changes ranging from mild physical discomfort or drowsiness up to complete loss of consciousness (*1–3*). Although regularly flickering light rarely exists for more than a few seconds in the natural environment, some types of artificial lighting, such as that found on long bridges or in long tunnels, can provide such a photic stimulation for a sufficient time (i.e., minutes) to cause these adverse effects. These effects, theoretically, could result in an increase in the frequency of nighttime traffic accidents.

The Illuminating Engineering Society (IES) has reported that in tunnels, the effects of flicker may produce undesirable behavioral sensations that in extreme cases include dizziness, drowsiness, queasiness, or seemingly hypnotic states in motorists and have caused seizures in people who have convulsive disorders (*4*). Lott has summarized the critical ranges of frequencies that produce these adverse effects and hence should be avoided in tunnel lighting designs: 5 to 10 cycles per second (*5*).

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On bridges, about 20 years ago, a few highway agencies began to design and install low-mounted, lineal-type lighting systems—almost all of which produced a flicker. The assumption was that such lighting provided better illumination and delineation than could be provided by conventional overhead lighting, was more glare-free than conventional lighting systems, provided superior visibility, and ultimately, should provide a safer nighttime environment (*6, 7*). However, such hypotheses were never validated.

One such installation was the San Mateo Bridge over San Francisco Bay. For over 20 years, this bridge has had a low-mounted, lineal lighting system that produces a noticeable flickering light (*6*). In theory, the length of this bridge (about 7 mi) provides sufficient exposure to this lighting to induce the adverse neurological effects described previously.

The availability of detailed accident records for this bridge, covering 7 years (1976 to 1982), provided a basis for determining whether the nighttime accident frequency on this bridge was in any way related to the amount of time that a motorist was subjected to the lighting system. If the nighttime safety on the bridge decreased in proportion to the time spent on the bridge, given that all other factors were controlled or accounted for, it would provide additional proof in an actual traffic environment of the original laboratory research.

In addition, because the lineal lighting on part of the bridge was later changed to a conventional pole-mounted system, an opportunity was available for a before-after accident analysis to determine the effect of this lighting change on nighttime safety.

DESCRIPTION OF SAN MATEO BRIDGE

The San Mateo Bridge is a multilane span, ~7 mi long, divided into two sections: a 5-mi level causeway section with two traffic lanes in each direction and a 2-mi raised section with three lanes in each direction. A medial barrier, 30 in. high, separates eastbound and westbound traffic. Average speeds on the bridge are 50–60 mph, similar to a limited access divided highway. The surface is portland cement.

DESCRIPTION OF LINEAL LIGHTING SYSTEM

The original lighting system, installed ~1966, consisted of 10-ft-long lineal fixtures containing 8-ft fluorescent aperture lamps. The remainder of each fixture contained the ballast. The fixtures were arranged end to end and mounted on the top of the medial barrier. The average horizontal illuminance on the causeway part of the bridge was ~1.25 fc with a uniformity (av/min) of about 1.7:1. On the raised part the values were about

1.0 fc and 5:1 uniformity. The lighting on this part of the bridge was changed in 1979 to a conventional pole-mounted high-pressure sodium (HPS) system, with average illumination over 2 fc.

The lineal lighting system used on the San Mateo Bridge generates a light pulse every 10 ft. At 55 mph (81 ft/sec), the pulse rate, or flicker, is 8 cycles per second. This flicker is very noticeable in the visual periphery, especially to drivers in the leftmost lane, closest to the lighting system. The flicker rate is in the center of the frequency range that should be avoided in lighting installations, according to the work of Lott (*5*).

FIRST ANALYSIS: EFFECT OF LINEAL LIGHTING ON SAFETY

Dependent Factors

The dependent factors chosen for the first analysis were the frequency of nighttime and daytime accidents, the night-to-day accident ratios, and the percentage of nighttime accidents. These factors were analyzed using accident data for the period 1976–1982 from the 5-mi causeway section of the bridge.

Objective

The overall goal of this first analysis was to determine the change, if any, in the preceding safety factors as a function of the length of time that a motorist drives under such lineal lighting. Because the volume on the bridge is constant over its entire length and the analysis of daytime accidents would account for any effects of the length of the bridge itself, excluding the lighting, it was felt that any change in safety at night would be attributed to the effects of the lighting system.

Analysis

To meet the stated goal, all accidents were classified by

- Light condition (day or night);
- Location on the causeway part of the bridge (mile marker); and
- Date (year).

The data for these three classifications were taken from accident report summaries obtained from the state of California. The statistic "accidents per mile" (APM) was computed for each of the five 1-mi sections of the causeway for each direction of traffic separately and for the entire 7-year period. The nighttime data are illustrated in Figures 1 and 2, and the daytime data are illustrated in Figure 3.

Results

Accidents per Mile

APM for the first and last miles of travel for both directions of travel and for both day and night were used for this analysis. The data are illustrated in Table 1.

It is quite evident that for both day and night, in both directions of travel, the APM are greater for the last mile than the first. However, the nighttime increases, as illustrated in Table 1 (+85 percent and +175 percent for eastbound and

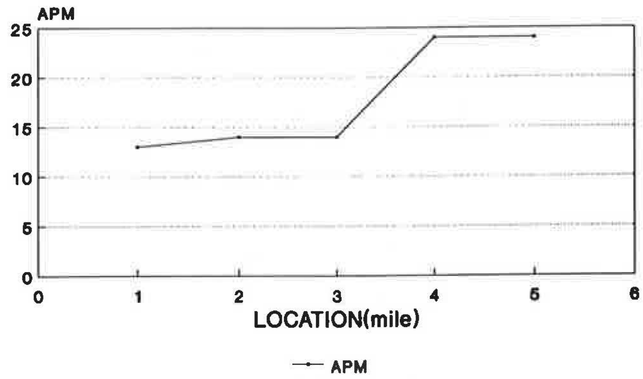

FIGURE 1 APM by location (nighttime, east).

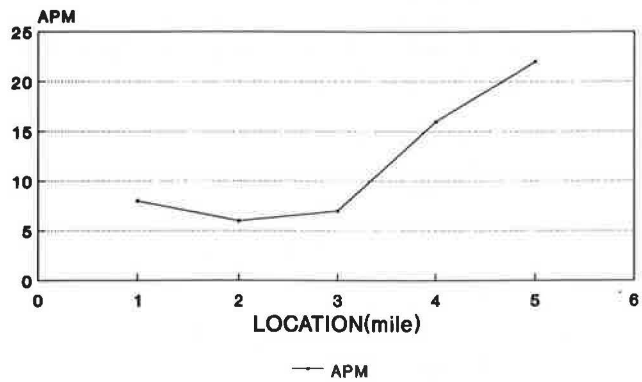

FIGURE 2 APM by location (nighttime, west).

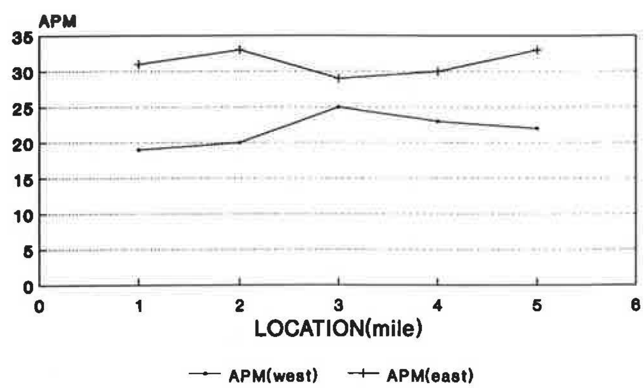

FIGURE 3 APM by location (daytime, east and west).

TABLE 1 ACCIDENTS PER MILE, DAY VERSUS NIGHT

Direction	Light Condition	Location: First Mile	Location: Last Mile	Change (%)
Eastbound	Day	31	33	+6
	Night	13	24	+85
Westbound	Day	15	22	+47
	Night	8	22	+175

westbound, respectively), are much greater than the daytime increases (+6 percent and +47 percent, respectively). This difference indicates a substantial decrease in safety at night between the first and last mile of travel.

Night-to-Day APM Ratios and Percentage of Night APM

Combining the night and day accidents per mile into night-to-day ratio (NDR) and percentage of night accidents (PNA),

$$\text{NDR} = \frac{\text{night accidents per mile}}{\text{day accidents per mile}} \tag{1}$$

$$\text{PNA} = \frac{\text{Night accidents per mile}}{\text{Day + night accidents per mile}} \tag{2}$$

which are considered to be better indicators of nighttime safety, yields the data in Table 2. It is again evident that both factors increase substantially (between 40 and 89 percent), indicating a decrease in nighttime safety between the first and last miles of travel for both directions of traffic.

TABLE 2 NIGHT-TO-DAY AND NIGHT PERCENTAGES

	Location		
Direction	First Mile	Last Mile	Change (%)
Eastbound			
Night-to-day	0.42	0.73	+72
Night percentage	0.30	0.42	+40
Westbound			
Night-to-day	0.53	1.00	+89
Night percentage	0.35	0.50	+43

Summary

The preceding analysis has shown that the nighttime safety on the San Mateo Bridge noticeably decreased, in both directions of travel, between the first and last 1-mi sections of the bridge. Because volume and environmental conditions such as weather are all constant over the entire length of the bridge and the increase in daytime accidents between first and last mile is far less than the increase at night (both directions), it is concluded that the lighting system, with its pronounced flicker, is the primary cause of this decrease in safety.

For both directions of travel, the nighttime safety does not decrease until the motorists have traveled at least 3 mi under such flickering lights. This is evident from the shape of the curves presented in Figures 1 and 2, which are flat until the fourth mile of travel in both directions. Hence the effect of the flicker is not evident until a driver has been exposed to it for about 3 min. The equivalent daytime results (Figure 3) show negligible change in APM over the 5-mi length in both directions.

The implication of the first result is that it is quite possible that the flicker produced by the low-mounted lineal lighting is inducing adverse neurological effects that cause a loss of driver control and result in this increase in nighttime accidents. The second result implies that the adverse effects will not occur instantaneously but require some minimum amount of driver exposure to such lighting before they become evident.

SECOND ANALYSIS: EFFECT OF LIGHTING CHANGE ON NIGHTTIME SAFETY

Dependent Factors

The dependent factors chosen for the second analysis were the frequency of nighttime and daytime accidents, the daytime and nighttime accident rates, the night-to-day accident ratios, and the percentage of nighttime accidents.

Objective

The overall goal of this analysis was to determine the effect of the lighting modification on the nighttime safety of motorists traveling that part of the bridge where the lighting was modified (referred to in the remainder of this paper as the changed section) and on the section where the lighting remained lineal (unchanged section).

Analysis

To meet the preceding goal, accident and volume data for 1976–1982 were obtained, and all accidents were classified by

- Time (date of occurrence, before or after),
- Location on bridge (changed or unchanged part), and
- Light condition (day or night).

Because the lighting modifications occurred in 1979, this year was excluded from the analysis. This was done because the exact completion date of the modification was unknown and because the effect of the actual lighting construction on traffic and safety could not be quantified. The remaining data (6 years, 1976–1978 and 1980–1982) were then classified as just described to yield eight distinct classifications or cells: two times (before or after); two locations (changed or unchanged); and two light conditions (day or night).

Accident frequencies and accident rates (frequency/volume) were computed for each of the eight cells. Additionally, night-to-day accident ratios and percentage of night accidents were computed for the four combinations of location and time. The results are discussed in three parts: effect on accident frequencies, effect on accident rates, and effect on night-to-day accident ratios and percentage of night accidents.

Results

Accident Frequencies

The accident frequencies for each of the eight cells are summarized in Table 3. From these data it can easily be seen that on the changed part of the bridge,

- Night accidents decreased 21 percent and
- Day accidents increased 43 percent;

and on the unchanged part of the bridge,

- Night accidents increased 74 percent and
- Day accidents decreased 3 percent.

These changes indicate an increase in nighttime safety on the changed portion while safety was decreasing at night on the

TABLE 3 ACCIDENT FREQUENCIES

	Light Condition	Location	
		Changed	Unchanged
Before	Day	41	106
	Night	28	39
After	Day	59	103
	Night	22	68
Change (%)	Day	+43	−3
	Night	−21	+74

unchanged part (the "control" section) and safety was decreasing during the daytime on the changed part.

Accident Rates

The average annual daily traffic (AADT) on the bridge rose by 18 percent between the before and after time periods (both day and night). Thus if after accident frequencies are divided by 1.18, the effect of the volume change is eliminated.

Adjusting the "after" data in Table 3 in the manner just described yields the data in Table 4. From these data it can be seen that on the changed part of the bridge,

- Night accident rates decreased 32 percent and
- Day accident rates increased 22 percent;

and on the unchanged part of the bridge,

- Night accident rates increased 49 percent and
- Day accident rates decreased 18 percent.

TABLE 4 VOLUME ADJUSTED ACCIDENT FREQUENCIES (ACCIDENT RATES)

	Light Condition	Location	
		Changed	Unchanged
Before	Day	41	106
	Night	28	39
After	Day	50	87
	Night	19	58
Change (%)	Day	+22	−18
	Night	−32	+49

Again, these changes indicate an improvement in nighttime safety on the changed part of the bridge while safety was decreasing on the unchanged part. During the daytime on the changed part, safety was also decreasing.

Night-to-Day Accident Ratios and Percentage of Night Accidents

Tables 5 and 6 summarize the NDR and PNA computations. It is obvious that on the changed part of the bridge,

- NDR decreased 46 percent and
- PNA decreased 34 percent;

TABLE 5 NIGHT-TO-DAY ACCIDENT RATES

	Location	
	Changed	Unchanged
Before	0.68	0.37
After	0.37	0.66
Change (%)	−46	+78

TABLE 6 PERCENTAGE OF NIGHT ACCIDENTS

	Location	
	Changed	Unchanged
Before	0.41	0.27
After	0.27	0.40
Change (%)	−34	+48

while on the unchanged part,

- NDR increased 78 percent and
- PNA increased 48 percent.

Both of these results indicate that the modification from lineal lighting to conventional overhead lighting improved the nighttime safety on the changed part of the bridge, whereas on the unchanged part a decrease in safety occurred.

Summary

This second analysis has shown that the nighttime safety on the changed part of the San Mateo Bridge improved when the lighting system was modified from rail-type lineal fluorescent fixtures to conventional pole-mounted, high-pressure sodium.

SUMMARY AND INTERPRETATION OF RESULTS

The analyses have provided two major results. First, the nighttime safety of motorists on the part of the San Mateo Bridge that retained the rail-type lineal lighting decreases after ~3 min exposure to such lighting. Second, when the lineal lighting was replaced by a conventional pole-mounted lighting system on part of the bridge, the nighttime safety improved on that part, whereas the nighttime safety decreased on the part that retained the lineal lighting.

The implication of the first result is that the lineal lighting, with its noticeable flicker, is causing this decrease in nighttime safety. The second result reinforces this interpretation because the nighttime safety improved substantially when the lineal lighting was replaced by a conventional pole-type system on part of the bridge.

Returning to the original hypothesis concerning the adverse neurological effects of such flickering light, it definitely appears possible that the decrease in safety results from neurological disorders caused by the flicker, compounded by the considerable light blockage and resulting shadow patterns caused by trucks and buses in the left lane. An alternative hypothesis is that the lineal lighting system itself, instead of the

TABLE 7 STATISTICAL ANALYSES (CHI SQUARE TESTS)

	Light Condition	Location	Factor	Significance Level
First Analysis				
Direction				
East or west	Night	First vs. last	APM	ns
East	Night	First vs. last	NTD	0.08
West	Night	First vs. last	NTD	0.02
Second Analysis				
Time				
Before and after	Night	Changed and unchanged	Accident frequency and rate	0.05
Before and after	Day and night	Changed and unchanged	Accident frequency and rate	0.05
Before and after	Day and night	Changed and unchanged	NTD	0.001

flicker, may be inducing the decrease in nighttime safety because of its design (i.e., light emanates from the side rather than above, as in all other driving situations, including daylight). Although this is an interesting hypothesis, there is no way of easily testing it. In any case, it is obvious that it is the lineal lighting that is causing the decrease in nighttime safety, for whatever reason.

A last suggested hypothesis is that the increase in illuminance provided by the newer HPS system is the sole cause of the increase in nighttime safety and that the lineal system was unsafe only because it did not provide sufficient light. If this hypothesis were true, then the graphs of nighttime accident frequency per mile illustrated in Figures 1 and 2 would be flat. Clearly, both of them increase at almost the same point (i.e., at 3 mi), indicating a time-dependent effect. This effect would be expected to result from a flicker but not from lighting alone. In addition, Box (*8*) has found that some increase in lighting (about 0.3 to 0.6 fc) increases nighttime traffic safety but that further increases do not. It should be noted that both the lineal and HPS systems have averages above Box's values. Finally, Janoff (*9*) found a direct relationship between average illuminance and nighttime safety: as the average illuminance increased, so did the nighttime accident rate. It is therefore felt that this last hypothesis is false.

IMPLICATIONS AND RECOMMENDATIONS

The implication of the preceding analyses and results is that bridge and tunnel lighting designers should avoid the lineal type of lighting system with luminaire configurations that produce such a noticeable flicker, especially for long bridges and tunnels. On the basis of the results presented by Lott, the critical range of flicker that should be avoided is 5–10 c/s.

STATISTICAL ANALYSES

The results of the statistical analyses for both analyses are presented here (Table 7). In general, for both analyses, the changes in simple accident frequencies, rates, and accidents per mile (APM) were often not significant, but the changes in night-to-day ratios, which are considered to be better indicators of nighttime safety, were always significant.

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Driver Decision Making at Traffic Signals

RICHARD VAN DER HORST

The driving task at signalized road intersections is simplified substantially over that at nonsignalized ones. At the guidance level, the decision-making process consists mainly of the stopping/nonstopping decision at the moment that the signal changes from green to yellow. Most red light running offenses appear to be related to this particular moment. The results of a 1-year before-and-after behavioral study in the northern part of the Netherlands demonstrate that a yellow interval of appropriate length (4 s for 50 km/hr and 5 s for 80 km/hr intersections) cuts the number of red light violations in half. Compared with fixed time control, vehicle-actuated signal control leads to a 1-s shift in the probability of stopping with respect to the potential time to the stopline at the onset of yellow, illustrating the role of driver expectancy in decision making. Through a comparison of these results with observed behavior at drawbridges and signalized railway grade crossings without gates, it is concluded that the absence of a separate yellow interval at railway grade crossings and drawbridges is disadvantageous. For all situations, a uniform stop signalization should be implemented. This signalization should consist of a steady red signal preceded by a yellow interval that is properly designed to serve the driver adequately in normal driving as well as to meet minimal driver needs in deteriorated circumstances.

For task analysis purposes, the driving task is generally divided into three performance levels: control, guidance, and navigation (*1*). In terms of the relationship with other traffic, the driving task at the guidance level for signalized road intersections is considerably simplified compared to that for nonsignalized intersections. However, at signalized intersections the road user has to deal with other tasks, such as processing information from traffic control signals. One of the important behavioral aspects of the driver's task is the decision whether to stop or to proceed when the signal changes from green to yellow and then to red. In an earlier study an optimal setting of the yellow interval was evaluated, and it was demonstrated that the type of signal control system (fixed time versus vehicle actuated) influences driver expectancy in this decision-making process (*2*).

The same kind of decision making is demanded when a driver approaches a drawbridge or a railway grade crossing at the onset of the red signal. The main difference in signalization in these cases, compared with roadway intersections, is the absence of a separate yellow interval. In this paper, by using the results of three studies dealing with driver decision making at road intersections, drawbridges, and railway grade crossings, it will be argued that at least part of the problem of running red lights can be solved by applying simple measures.

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STOPPING OR NONSTOPPING

The Traffic Rules and Traffic Signs Law in the Netherlands distinguishes among traffic control signals at road intersections, railway grade crossings, and drawbridges. In all these situations the required reaction to the red signal (either steady or flashing) is quite unambiguous, namely, "stop." At road intersections the red signal is always preceded by a yellow signal, which, as the law states, means "Stop; but those drivers who are close enough to the intersection that they are not reasonably able to stop may proceed."

The rationale for the yellow interval is, in principle, to prevent vehicles that are within a given (short) distance of the intersection from inevitably running through the red signal. At drawbridges the signal changes from off to steady or flashing red. This change is sometines preceded by prewarning flashing amber signals at a given distance. At railway grade crossings the signal changes either from off (at railway crossings with gates) or from flashing white (at railway crossings without gates) to two signal heads alternately flashing red. The absence of a yellow phase at these stops is questionable, and the precise meaning of the flashing of a red signal can be questioned as well.

When the signal changes from green to yellow at a road intersection, a decision is demanded. The criterion for this decision (the distance within which it is not reasonable to stop) is not very precise, and for that reason it is subject to differences in interpretation. Although the yellow phase is absent at railway grade crossings and drawbridges, the decision-making process itself is not very different. Factors that may influence this process involve the driver's motivation and attitude, the amount of predictability of the situation, the estimate of the consequences of not stopping (likelihood of running red and getting a fine or of getting involved in a conflict with intersecting traffic or a collision with the grade crossing gate), and the estimate of the consequences of stopping (discomfort, waiting time, likelihood of a read end collision or that braking will end in a stop within the intersection). Evaluating these factors also involves the driver's estimates of the required deceleration on the basis of the speed and distance to the stop line and the expectations of the duration of the yellow phase and the all-red period (or the time left until the gates close or the train arrives).

All of this discussion is based on the premise that the signals are implemented and positioned to be visible in due time for the approaching driver to make a decision. Also, the behavior of other road users may influence the decision making. An example is the so-called "mitschleppeffekt," in which a nonstopping leading automobile influences the following automobile to continue as well (3, 4).

SIGNALIZED ROAD INTERSECTIONS

So that the decision-making processes at road intersections, drawbridges, and railway grade crossings can be compared, the results of the road intersection study with respect to the timing of the yellow signal and the type of traffic control strategy will be discussed briefly. More detailed information can be found elsewhere (2, 5).

Duration of the Yellow Interval

Extensive literature is available regarding the timing of the yellow interval. From an explorative study on the extent of the problem of red light running (6) and from a review of the literature on measures related to traffic signal control (5), it was concluded that the optimum yellow timing was 4 s of yellow for 50 km/hr intersections and 5 s for 80 km/hr intersections. Compared with current values in the Netherlands, this means that the duration of the yellow interval should be extended by 1 s to serve the driver appropriately in normal driving conditions and to meet minimum driver needs in deteriorated circumstances.

With this change in yellow timing the number of run red offenses would be reduced considerably, simply because 1-s prolongation of the yellow is not expected to change the drivers' behavior. Furthermore, more consistent decision making (in the sense that "emergency-type" braking and swerving are not needed anymore) might also be beneficial to traffic safety. On the other hand, the yellow time should not be longer than recommended because the stopping driver has to be "rewarded" with red (confirmation of appropriate behavior) and because an overlong yellow time might lead to greater variability in the decision making, resulting, for example, in an increase of the number of rear end collisions. Evidence for the latter has been found in literature on the safety effects of flashing green (3, 7).

Because the system is better adapted to "normal" behavior, enforcement measures might be much more effective. First, the group of offenders is expected to be much smaller, and second, the beginning of red no longer functions as a "necessary" extension of the yellow.

A 1-s extension of the yellow was evaluated in a 1-year before-and-after study in both urban and rural locations. All locations were provided with vehicle-actuated signal control. In the city of Leeuwarden, which has 23 signalized intersections, the yellow interval was changed from 3 to 4 s. The change was made without shortening the all-red time so that possible interfering effects would be prevented. The behavior of automobile drivers was observed at four selected intersections and was then analyzed quantitatively from time-lapse video recordings. The cameras recorded about 15,000 traffic signal cycles, in which 7,000 "deciding" vehicles (those nonstopping plus the automobiles that stopped first after the beginning of the yellow) were registered. The extension of the yellow interval at rural intersections from 4 to 5 s was evaluated on a rural route to Leeuwarden that had seven signalized intersections. At two intersections the behavior of about 3,000 deciding vehicles was observed for about 4,500 traffic signal cycles. Details about method, procedure, and quantitative analysis may be found elsewhere (2, 8).

One year after the timing was changed, the number of run-red offenses at urban intersections appears to have been halved, from 1.1 to 0.5 percent of the total number of vehicles and from 13.4 to 6.7 percent of the number of deciding vehicles. At rural intersections the same reduction was achieved. The proportion of nonstopping automobiles with a stop line–passing time (TPS) greater than or equal to a given time t after the onset of yellow (Figure 1) shows a small shift of about 0.13 s for the urban intersections (D_{max} = 6.0, Kolmogorov-Smirnov test, one-tailed, $p < 0.01$). Results for the "between" period (6 months after the extension of the yellow) differ by this same amount from the "before" period, but the between period does not differ significantly from the 1 yr after period. At rural intersections, no significant difference was found between the before and 1 yr after periods.

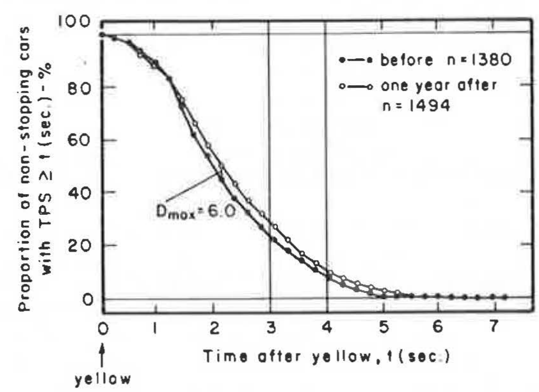

FIGURE 1 Proportion of the nonstopping cars with TPS ≥ t after the onset of the yellow, for before and 1 yr after the extension of yellow at urban intersections.

By including the information on the stopping drivers, the probability of stopping with respect to the potential time to the stop line (TTS) was calculated, taking into account the individual approach speed of each vehicle and assuming that vehicles will continue with a constant speed. A log-linear model fit resulted in a shift of 0.17 s between before and after situation. This (relatively small) change in driver behavior appears to be present for the after 6 months period as well and does not change thereafter.

From this field study it is concluded that a 1-s extension of the yellow time (from 3 to 4 s in urban areas and from 4 to 5 s in rural areas) cuts the total number of run-red offenses in half, although a small adaptation effect was also observed. However, after 6 months, no further change in driver behavior was found. The results are completely in agreement with the expectations of a simple model for calculating the required yellow time.

Type of Traffic Signal Control

It might be expected that different types of traffic signal control will differ in terms of exposure to potential offenses and will also perhaps differ in the process used by the drivers to decide whether to stop or to proceed. If a vehicle-actuated control strategy is used, the green phase, in principle, will be extended as long as there are vehicles in a given detection area. When this control strategy operates well, it will result in much fewer potential red runners than does fixed time cycle control, in which the times selected for ending the green are independent of the traffic at the moment.

Zegeer and Deen (*9*) found a large reduction in the number of run red offenses when a green extension system was applied. The reduction was due mainly to the difference in exposure, but what about the differences in the discipline at the red light at the moment that the decision is required? The expectations of automobile drivers approaching the traffic light might differ for different types of control, resulting in different decision behavior in general, regardless of the duration of the yellow.

In the field study on the effects of yellow timing that was described in the previous section, all the traffic signal controls were vehicle actuated. In Figure 2, the probability of stopping at intersections with vehicle-actuated control is compared with data on fixed time control, available from the literature (*10–12*). All data sets are based on field studies, except those of Mahalel et al. (*10*). They conducted a laboratory experiment on the decision-making behavior of automobile drivers at traffic lights. A remarkable 1-s shift exists between the probability of stopping for a fixed cycle time control and that of stopping for a vehicle-actuated control. Evidently, a vehicle-actuated control leads to a shift in the criterion that drivers use. The drivers decide to proceed in an earlier stage of the approach process, when compared with the decision made for fixed-time control. In the field studies, it was observed that the characteristics of the decision process itself are not different, as is indicated by the similarity of the slopes of the curves at a 0.5 probability of stopping. The slope of the laboratory experiment at this point tends to be different, indicating a somewhat deviant behavior in more artificial circumstances (*10*).

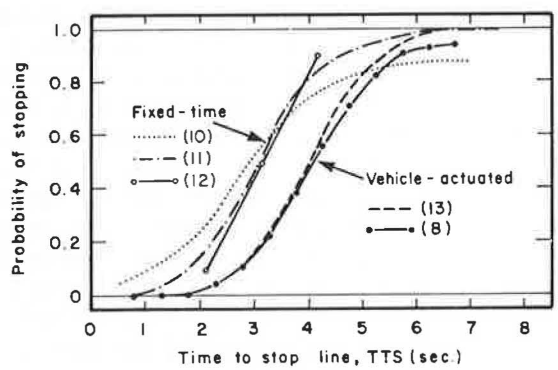

FIGURE 2 Probability of stopping as a function of the time to stop line (TTS) for fixed time (*10–12*) versus vehicle-actuated (*8, 13*) signal control systems.

Given this dependency of behavior on the type of traffic signal control, the hypothesis can be formulated that automobile drivers who are accustomed to vehicle-actuated control will adapt their behavior and expect to see a response when approaching the traffic signal during green. Contrary to the fixed time control situation, the drivers expect that the green interval will continue. Therefore, when the green phase ends (that is, the maximum green extension period has been reached), their expectancy is violated, with consequences for their decision-making processes (*1*). It is expected that similar effects will occur in other situations. For example, in a string of interconnected signalized intersections (progressive signal control systems), the experience at previous intersections causes the expectancy that the driver will have or get the green interval at the next one as well.

To conclude, in spite of a negative effect on the decision-making behavior of automobile drivers per se, a vehicle-actuated traffic control system reduces the number of run-red offenses substantially in comparison with fixed time control. This reduction is mainly caused by a reduction in exposure, however.

RED SIGNALS AT DRAWBRIDGES

In the Netherlands, drawbridges are common. The operation of these bridges, especially on roads with heavy traffic flows, is sometimes problematic. On a freeway, for example, bridgekeepers sometimes have problems conducting the opening procedure because drivers persist in proceeding. The procedure is automatic once started by the bridgekeeper and consists of prewarning flashing signals (combined with a sign showing a drawbridge) located 900, 600, and 300 m before the bridge (the flashing starts successively about 30 s before the red signal), the onset of alternating flashing red lights above the stop line on the ramps of the bridge, the closing of the gates (8 s after red), and finally the opening of the bridge. The bridgekeeper can interrupt this procedure as needed if the descending gate threatens to hit a proceeding automobile. According to complaints by the bridgekeepers, 1 out of 5 or 10 closures has to be interrupted (*14*).

Why don't the drivers stop? In this case, it is not a matter of poor visibility of the signals or the gates. One assumption is that motorists, because they are familiar with this system and have visual contact with the bridgekeeper, know that the bridgekeeper is able to interrupt the procedure. The interaction between motorist and bridgekeeper and their mutual anticipation results in a nonstopping motorist. However, hiding the bridgekeeper behind one-directional curtains was not an effective countermeasure (*14*).

From video recordings, the behavior of the nonstopping automobiles was analyzed in more detail. Recordings of 82 closures with 348 nonstopping automobiles after the onset of the red signal were analyzed. Figure 3 gives the proportion of the nonstopping automobiles with a TPS greater than or equal to a given time *t* after the onset of red. It can be seen that 12 percent of the nonstopping automobiles pass the stop line after the gates begin to close (8 s after the onset of red).

If the behavior at this drawbridge is compared with that at signalized rural intersections with similar approach speeds, it is obvious that motorists make the decision to proceed much earlier at the drawbridge. On the basis of a speed of 100 km/hr, a reaction time of 1 s, and an average decleration rate of 3 m/s^2, a clearance interval of 5.6 s would be sufficient. Yet 30 percent of the nonstopping automobiles pass the stop line later than this, compared to 6 percent for rural intersections. Even with a speed of 140 km/hr, drivers would be able to stop before the stop line at a time distance of 7.5 s. Yet 14 percent of the

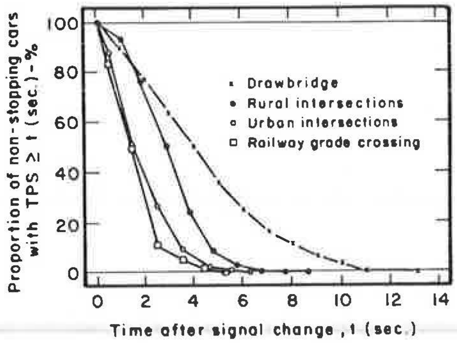

FIGURE 3 Proportion of nonstopping automobiles with TPS ≥ *t* after the onset of the yellow (for road intersections) or red (for drawbridges and railway crossings) signal.

nonstopping automobiles pass later. It appears that drivers are not *willing* to stop, rather than not *able* to stop. Given our knowledge of motorist behavior at signalized intersections, however, the functioning of the drawbridge signalization itself is questionable. About 30 s before the red signal is activated, the prewarning signal at 900 m starts flashing. Even when the prewarning signal is perceived, the uncertainty about the exact time that the red will appear is great because of the long interval (30 s) that passes with no complementary message to the motorist. The signaling system is in the same state at the end of the cycle, when the gates have been opened again. At the end of the cycle the warning lights are used as a warning for the queue of standing cars. In itself, this measure is effective to an extent, but it devalues the main function of these lights. Approaching drivers might be uncertain about whether the beginning or the end of the cycle is indicated.

At a speed of 100 km/hr, motorists who have just missed the warning signal will reach the stop line after 32.5 s. At that moment, the red signal is on for 2.5 s. Logically, it cannot be expected that those motorists will decide to stop at the onset of the red signal. They will proceed and inevitably run the red. Thus the meaning of the red signal is rather ambiguous. Following drivers see those automobiles in front running a red light. Because their situation is only marginally different (often heavy traffic flows with small headways), they are easily tempted to proceed as well (the previously mentioned mitschleppeffekt). A separate yellow signal, preceding the red signal and combined with a much shorter warning time, might prevent a lot of these situations.

RED SIGNALS AT RAILWAY GRADE CROSSINGS

Recently, a study was conducted on the decision-making behavior of motorists who were approaching railway grade crossings that had signal control and were without gates (*15*). At this type of railway crossing, the yellow interval is absent: The signal changes directly from the safe situation (flashing white) to the unsafe situation (alternating flashing red). This is another situation in which road users may not able to stop and therefore necessarily proceed through red.

The behavior of 660 motorists at the onset of the red signal was analyzed at two railway crossings. In contrast with the behavior at the drawbridge, motorists at this type of railway crossing are more willing to stop than those at signalized urban intersections with comparable approach speeds (Figure 3). The probability of stopping at a given time distance appears to be higher at railway grade crossings, even in comparison with the data on fixed time control. At railway grade crossings the lack of a yellow phase seems to work in the opposite direction; that is, the obedience to the red light is better than at signalized intersections. However, a detailed analysis of the stopping vehicles indicated that some of the stops were realized only with a high deceleration rate (> 4 m/s^2), and a few of them ended very close to the first railway track. Such "emergency" stops can easily lead to a stop on the track itself or to a rear end collision by a following automobile. Because of these risks, this kind of behavior is not desirable. It can be prevented by a separately signalized clearance interval with a yellow indication.

CONCLUSIONS AND RECOMMENDATIONS

At signalized road intersections, an extension of the yellow time by 1 s resulted in the number of run red offenses being cut in half. In the literature, evidence was found that 4 s of yellow for 50 km/hr zones and 5 s for 80 km/hr zones are optimum values. Longer times are not desirable because of expected secondary effects.

The comparison between behavior at vehicle-actuated and fixed time control systems indicates that motorists use expectations about the functioning of the signals and act accordingly. The relatively small number of run red offenses at vehicle-actuated signals is mainly due to a low exposure of deciding vehicles. Discipline at the red light per se appears to be less than for fixed time signals, probably because of a difference in expectancy.

Although the willingness of motorists to stop at drawbridges seems to be poor, at least some of the offenses can be explained by operational deficiencies of the signal system itself. The lack of a separate indication for the clearance interval is a major one. By introducing a yellow interval, only one interpretation of the red signal is possible, namely "stop." At railway grade crossings the absence of the yellow interval appears to have an opposite effect. The sudden onset of the red signal sometimes causes a kind of a panic reaction, with a very abrupt stop. This is especially the case with motorists who are so close to the railway track that proceeding would have been more appropriate. Although the obedience to the red light is better at grade crossings than at signalized intersections, large deviations from normal behavior occur. A separate yellow signal would therefore be advantageous at railway grade crossings as well.

Because the same behavior is demanded from the road user in all three examples given, one uniform signalization for stopping is recommended: a steady red signal preceded by a yellow interval that is properly designed to serve the driver adequately in normal driving as well as to meet minimal driver needs in deteriorated circumstances. Specifically, 4 s of yellow for 50 km/hr zones and 5 s for 80 km/hr zones are recommended.

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