Prediction of the Expected Safety Performance of Rural Two-Lane Highways

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FOREWORD

This report documents the algorithm for predicting the safety performance of rural two-lane highways that forms the basis for the Crash Prediction Module of the Interactive Highway Safety Design Model. The algorithm estimates the effect on safety performance of roadway segment parameters including lane width, shoulder width, shoulder type, horizontal curves, grades, driveway density, two-way left-turn lanes, passing lanes, and roadside design, and of intersection parameters including skew angle, traffic control, exclusive left- and right-turn lanes, sight distance, and driveways. The algorithm enables highway agencies to estimate the safety performance of existing or proposed highways and to compare the expected safety performance of geometric design alternatives.

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Research and Development

NOTICE

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16. Abstract

This report presents an algorithm for predicting the safety performance of a rural two-lane highway. The accident prediction algorithm consists of base models and accident modification factors for both roadway segments and at-grade intersections on rural two-lane highways. The base models provide an estimate of the safety performance of a roadway or intersection for a set of assumed nominal or base conditions. The accident modification factors adjust the base model predictions to account for the effects on safety for roadway segments of lane width, shoulder width, shoulder type, horizontal curves, grades, driveway density, two-way left-turn lanes, passing lanes, roadside design and the effects on safety for at-grade intersections of skew angle, traffic control, exclusive left- and right-turn lanes, sight distance, and driveways.

The accident prediction algorithm is intended for application by highway agencies to estimate the safety performance of an existing or proposed roadway. The algorithm can be used to compare the anticipated safety performance of two or more geometric alternatives for a proposed highway improvement.

The accident prediction algorithm includes a calibration procedure that can be used to adapt the predicted results to the safety conditions encountered by any particular highway agency on rural two-lane highways. The algorithm also includes an Empirical Bayes procedure that can be applied to utilize the safety predictions provided by the algorithm together with actual site-specific accident history data.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

Contents

1. INTRODUCTION

One of the most critical gaps in the management of highway safety is the lack of a reliable method for estimating the safety performance of an existing or planned roadway. Accident record systems have been developed and maintained by highway agencies to monitor the safety performance of their roadways, but these provide historical or retrospective data. Effective management requires a prospective viewpoint. Highway engineers need to know not what the safety performance of a roadway was in the recent or distant past, but what it is now and what it is likely to be in the future if particular proposed actions are taken.

In the past, when current or future safety performance estimates for a roadway were needed, they have been developed by one of four approaches: averages from historical accident data, predictions from statistical models based on regression analysis, results of before-after studies, and expert judgments made by experienced engineers. Each of these methods, used alone, has significant weaknesses which are described below. A new approach combining elements of each of these methods into an accident prediction algorithm is then described. This new accident prediction algorithm, developed specifically for application to rural two-lane highways, is the subject of this report.

Estimates from Historical Accident Data

Historical accident data are an important indicator of the safety performance of a roadway, but they suffer from the weakness of being highly variable. Given this high variability, it is difficult to estimate the long-term expected accident rate using a relatively short-duration sample of 1 to 3 years of accident data. This is especially true for rural roadway sections and intersections where accidents are very rare events and many locations experience no accidents, or at most one accident, over a period of several years. If a location has experienced no accidents in the past several years, it is certainly not correct to think that it will never experience an accident, yet the available data for that site alone provide an insufficient basis for estimating its long-term expected safety performance.

Roadway improvement programs based on safety are often managed with accident surveillance systems that use accident records to identify high-accident locations. A high-accident location is a roadway section or intersection identified because it experienced more than a specified threshold number of accidents during a recent period (typically 1 to 3 years). Each high-accident location is investigated by the engineering staff of the responsible highway agency and, at locations where a particular accident pattern is clearly evident and an appropriate countermeasure is feasible, an improvement project may be programmed and constructed. The decisionmaking concerning such projects often involves a benefit-cost or cost-effectiveness calculation based on the expected percentage reduction in accidents from the level of recent accident experience found by the accident surveillance program. However, both statistical theory and actual experience show that, because of the random nature of accidents, locations with high short-term accident experience are likely to experience fewer accidents in the future even if no improvement is made. This phenomenon, known as *regression to the mean*, makes it difficult both to identify potential problem locations through accident surveillance and estimate the potential (or actual) effectiveness of improvements made at such locations.

Estimates from Statistical Models

Safety analysts have, for many years, applied statistical techniques to develop models to predict the accident experience of roadways and intersections. Such models are developed by obtaining a database of accident and roadway characteristics (e.g., traffic volumes, geometric design features, and traffic control features) data from highway agency records, selecting an appropriate functional form for the model, and using regression analysis to estimate the values of the coefficients or parameters in that model. Historically, most such models were developed with multiple regression analysis. Recently, researchers have begun to use Poisson and negative binomial regression analyses which are theoretically better suited to accident data based on small counts (i.e., zero or nearly zero accidents at many sites). However, regardless of the statistical technique used, accident prediction models never quite seem to meet the expectations of their developers and potential users.

Regression models are very accurate tools for predicting the expected total accident experience for a location or a class of locations, but they have not proved satisfactory in isolating the effects of individual geometric or traffic control features. There is a strong temptation to interpret each coefficient in a regression model as representing the true effect of an incremental change in its associated roadway feature. This is a reasonable assumption is some cases, but not in others. A key drawback of regression models is that they are based on statistical correlations between roadway characteristics and accidents that do not necessarily represent cause-and-effect relationships. Furthermore, if the independent variables in the model are strongly correlated to one another, it is difficult to separate their individual effects. In addition, if a variable in the model is strongly correlated to an important variable that happens not to be included in the available data base, the coefficient of the variable in the model may represent the effect of the unavailable variable rather than its own effect. Thus, the value of the coefficient of a particular geometric feature may be a good estimate of the actual effect of that feature on safety, or it may be merely an artifact of, or a surrogate for, its correlation to other variables.

As an example, consider the following negative binomial regression model developed in a recent FHWA study to predict the accident experience at urban, four-leg intersections with STOP control on the minor road:(1)

$$Y = e^{-5.073}\,(X_1)^{0.635}\,(X_2)^{0.294}\,\exp(-0.969\;X_3)\,\exp(-0.518\;X_4)\,(X_5)^{-0.091}\,\exp(0.340\;X_6)\,\exp(0.087\;X_7)\,\exp(-0.331\;X_8)\,\exp(-0.175\;X_9) \qquad \textbf{(1)}$$

where:

- Y = expected number of total multiple-vehicle accidents in a 3-year period;
- X_1 = average daily traffic on major road (veh/day);
- X_2 = average daily traffic on minor road (veh/day);
- X_3 = 1 if left-turn are prohibited on one or more major-road approaches; 0 otherwise;
- X_4 = 1 if no access control is present along the major road approaches; 0 otherwise;
- X_5 = average lane width on major road (ft)*;
- X_6 = 1 if major road has three or fewer through lanes in both directions of travel combined; 0 otherwise;
- X_7 = 1 if major road has four or five through lanes in both directions of travel combined; 0 otherwise;
- X_8 = 1 if there is no channelization for free right turns; 0 otherwise; and
- X_9 = 1 if the intersection has no lighting; 0 otherwise.

This model, overall, provides quite reliable predictions of the total accident experience of urban, four-leg, STOP-controlled intersections. In addition, the coefficients of many of the terms appear to reasonably represent the expected effects of their associated variables. However, two of the variables in the model have coefficients that are in a direction opposite to that which safety engineers normally presume for those variables. Specifically, the negative coefficient of the access control factor (X_4) implies that more accidents would be expected at an intersection with access-controlled approaches than at an intersection without access-controlled approaches. Furthermore, the negative coefficient of the lighting factor (X_9), implies that lighted intersections have more accidents than unlighted intersections. Such interpretations are unreasonable. The negative signs for the access control and lighting variables in equation (1) could result merely from correlations of access control and lighting with the variables already accounted for in the model, such as traffic volumes, or with other important variables that are not included in the model because no data for those variables are available. It is also possible that lighting has been installed as an accident countermeasure at high-accident locations, so that lighting appears to be associated with locations that have more accidents. Thus, while regression equations may provide useful predictive models, their coefficients may be unreliable indicators of the incremental effects of individual roadway features on safety.

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* Average lane width in this equation is specified in conventional units of measure (feet). See the explanation in the section entitled Units of Measure on page 5 of this report.

Estimates from Before-and-After Studies

Before-and-after studies have been used for many years to evaluate the effectiveness of highway improvements in reducing accidents. However, most before-and-after studies reported in the literature have design flaws such that the study design cannot account for the effects of regression to the mean. Therefore, the potential user of the before-and-after study results cannot be certain whether they represent the true effectiveness of the potential improvement in reducing accidents or an overoptimistic forecast that is biased by regression to the mean.

Safety experts are generally of the opinion that, if the potential bias caused by regression to the mean can be overcome, a before-and-after study may provide the best method to quantify the safety effects of roadway geometric and traffic control features. Hauer[(2)] has developed a new approach that remedies the problem of regression to the mean that has, in the past, caused before-and-after studies to provide unreliable results. However, very few of these well-designed before-and-after studies have been conducted.

Estimates from Expert Judgment

Expert judgment, developed from many years of experience in the highway safety field, can have an important role in making reliable safety estimates. Experts may have difficulty in making quantitative estimates with no point of reference, but experts are usually very good at making comparative judgments (e.g., A is likely to be less than B, or C is likely to be about 10 percent larger than D). Thus, experts need a frame of reference based on historical accident data, statistical models, or before-and-after study results to make useful judgments.

A New Approach

This report presents a new approach to accident prediction that combines the use of historical accident data, regression analysis, before-and-after studies, and expert judgment to make safety predictions that are better than those that could be made by any of these three approaches alone. The recommended approach to accident prediction has its basis in published safety literature, including both before-and-after evaluations and regression models, is sensitive to the geometric features that are of greatest interest to highway designers, and incorporates judgments made by a broadly based group of safety experts.

This report shows how this new approach can be implemented in an accident prediction algorithm for rural two-lane highways. This same approach can potentially be adapted in the future to rural multilane highways, urban arterial streets, and rural or urban freeways.

The Federal Highway Administration (FHWA) is currently developing an Interactive Highway Safety Design Model (IHSDM) for use by highway designers to incorporate more explicit consideration of safety into the highway design process. IHSDM will consist of a set of computer tools that can work interactively with the Computer-Aided Design (CAD) systems used by many agencies to design highway improvements. The components of the IHSDM will include a Crash Prediction Module (CPM), Roadside Safety Module (RSM), Intersection Diagnostic Review Module (DRM), Design Consistency Module (DCM), Policy Review Module (PRM), Driver/Vehicle Module (D/VM), and Traffic Analysis Module (TAM). Initial priority in IHSDM development is being given to evaluation of rural two-lane highways.

The accident prediction algorithm presented in this report has been developed for incorporation in the IHSDM as the CPM for rural two-lane highways, but is also suitable for use as a stand-alone model to predict the safety performance of rural two-lane highways. This report documents how the accident prediction algorithm was developed and how it will function within the IHSDM.

Organization of this Report

The remainder of this report is organized as follows. Section 2 presents an overview of the accident prediction algorithm and its two primary components, base models and accident modification factors. A more detailed description of the base models and accident modification factors is presented in sections 3 and 4, respectively. Section 5 presents the results of sensitivity analyses conducted with the accident prediction algorithm, and section 6 explains how the accident prediction algorithm will be implemented within the IHSDM. The conclusions and recommendations of the report are presented in section 7 and a list of references is presented in section 8.

Appendix A identifies the members of the expert panels that developed the accident modification factors. Appendix B documents the development of the base models. Appendix C presents a calibration procedure that can be used by any highway agency to adapt the accident prediction algorithm to their own local conditions and to the safety performance of their highways. Appendix D documents the definitions of the roadside hazard ratings used in the accident prediction algorithm to represent roadside design features.

Units of Measure

The text of this report presents all measured quantities in SI (metric) units with equivalent quantities in conventional (English) units following in parentheses. However, virtually all of the research on which the report is based was conducted using conventional units of measure. Therefore, all equations in the report, like equation (1) above, use conventional units. A metric conversion chart is included for the convenience of readers. The software developed to implement the accident prediction algorithm will allow users to provide input and obtain output at their option in either SI or conventional units.

2. OVERVIEW OF THE ACCIDENT PREDICTION ALGORITHM

This section of the report presents an overview of the accident prediction algorithm for rural two-lane highways. Separate accident prediction algorithms have been developed for roadway segments and for three types of at-grade intersections. These separate algorithms can be used together to predict the total accident experience for an entire highway section or improvement project.

In the development of these algorithms, it was decided the roadway segment accident prediction algorithm would predict all *non-intersection-related* accidents for each of the segments that make up a highway project. Non-intersection-related accidents include accidents that occur near an intersection but are not related to the intersections. For example, a ran-off-road accident or a head-on collision that occurs within 15 m (50 ft) of an intersection, but it considered by the investigating officer to be unrelated to the intersection, would be classified as a non-intersection-related accident. The intersection accident prediction algorithms predict the additional intersection-related accidents that occur at or rear the intersection and occur because of the presence of the intersection. For modeling purposes, only accidents that occurred within 76 m (250 ft) of the intersection and occurred because of the presence of the intersection were considered to be *intersection- related accidents.* The total predicted accident frequency for any highway project is the sum of the predicted frequency of non-intersection-related accidents for each of the roadway segments and the predicted frequency of intersection-related accidents for each of the at-grade intersections that make up the project.

The accident prediction algorithms for roadway segments and at-grade intersections are each composed of two components: base models and accident modification factors. These components and the manner in which they are combined are described below.

Accident Prediction Algorithm for Roadway Segments

The base model for roadway segments is the best available regression model for predicting the total accident frequency of a roadway segment on a rural two-lane highway. The base model, like all regression models, predicts the value of a dependent variable as a function of a set of independent variables. For the roadway segment model, the dependent variable is the total expected accident frequency on the roadway segment during a specified time period. The independent variables used to predict accident frequency are descriptors of the traffic volumes, geometric design features, and traffic control features of the roadway segment. The specific regression model to be used as the base model for rural two-lane roadway segments is presented in section 3 of this report.

As discussed in the introduction to this report, regression models like the base model are useful in predicting overall accident frequency, but their coefficients cannot necessarily be relied upon to represent the incremental effects of individual geometric design and traffic control features. Therefore, the base model will be used only to estimate the expected accident frequency for a specified set of nominal base conditions, such as 3.6-m (12-ft) lane widths and 1.8-m (6-ft) shoulder widths. This base estimate of accident frequency will then be adjusted with accident modification factors (AMFs) that represent the safety effects of individual geometric design and traffic elements. The general formulation of the algorithm predicting roadway segment accident frequency and combining the base models and AMFs is shown below:

$$N_{rs} = N_{br}\,(AMF_{1r}\ AMF_{2r} \cdots AMF_{nr}) \qquad \textbf{(2)}$$

where:

N_{rs} = predicted number of total roadway segment accidents per year after application of accident modification factors;

N_{br} = predicted number of total roadway segment accidents per year for nominal or base conditions; and

$AMF_{1r} \cdots AMF_{nr}$ = accident modification factors for roadway segments.

The AMFs are multiplicative factors used to adjust the base accident frequency for the effect of individual geometric design and traffic control features. Each AMF is formulated so that the nominal or base condition is represented by an AMF of 1.00. Conditions associated with higher accident experience than the nominal or base condition will have AMFs greater than 1.00 and conditions associated with lower accident experience than the nominal or base condition will have AMFs less than 1.00. For example, if the accident frequency predicted by the base model for roadway segments (N_{br}) is based on 3.6-m (12-ft) lanes, but a particular roadway section of interest has 3.3-m (11-ft) lanes, the AMF for lane width might have a value of 1.15. This AMF implies that a two-lane roadway segment with 3.3-m (11-ft) lanes would be expected to experience 15 percent more accidents than a comparable roadway section with 3.6-m (12-ft) lanes.

The effect of average daily traffic (ADT) volume on predicted accident frequency is incorporated through the base models, while the effects of geometric design and traffic control features are incorporated through the AMFs.

The formulation shown in equation (2) allows the AMF for each geometric design and traffic control element to be based solely on the most reliable information concerning the safety effects of that particular element. The best method for considering the safety effects of lane width can be selected as the basis for the AMF without being constrained by the treatment of lane width in the base model or by the formulation of any other AMF. Thus, each AMF in the accident prediction algorithm can be based on the best and most applicable research available, as selected and interpreted by knowledgeable experts. In fact, two panels of experts, identified in appendix A, were formed to establish the AMFs presented in this report. Section 4 of the report presents the AMFs used in predicting roadway segment accidents and documents their development.

Accident Prediction Algorithm for At-Grade Intersections

The structure of the accident prediction algorithm for at-grade intersections is similar to the algorithm for roadway sections presented above. The predicted frequency of accidents that occur at or are related to an at-grade intersection is determined as:

$$N_{int} = N_{bi} \, (AMF_{1i} \; AMF_{2i} \cdots AMF_{ni}) \qquad \textbf{(3)}$$

where:

N_{int} = predicted number of total intersection-related accidents per year after application of accident modification factors;

N_{bi} = predicted number of total intersection-related accidents per year for nominal or base conditions; and

$AMF_{1i}, \cdots, AMF_{ni}$ = accident modification factors for intersections.

Separate base models have been formulated for three-leg intersections with STOP control, four-leg intersections with STOP control, and four-leg signalized intersections. The AMFs used in the accident prediction algorithm for these three intersection types also differ, but the algorithms for all three intersection types are structured as shown in equation (3). The base models for at-grade intersections are presented in section 3 of this report, and the AMFs for at-grade intersections are presented in section 4.

The effect of traffic volume on predicted accident frequency for at-grade intersections is incorporated through the base models, while the effect of geometric and traffic control features are incorporated through the AMFs. Each of the base models for at-grade intersections incorporates separate effects for the ADTs on the major- and minor-road legs, respectively.

Predicted Accident Frequency for an Entire Project

The accident prediction algorithm will be applied in IHSDM to estimate the safety performance of entire proposed projects or extended highway sections. The total predicted accident frequency for an entire project or an extended highway section can be determined as:

$$N_t = \sum_{\substack{\text{all} \\ \text{segments}}} N_{rs} + \sum_{\substack{\text{all} \\ \text{intersections}}} N_{int} \qquad \textbf{(4)}$$

where: N_t = predicted accident frequency for an entire project or an extended highway section.

Estimated Accident Severity and Accident Type Distributions

In addition to predictions of accident frequency based on equations (2) and (3), the accident prediction algorithm will also provide estimates of the accident severity and accident type distributions for roadway segments and at-grade intersections. Tables 1 and 2 present default estimates of the accident severity and accident type distributions, respectively, that are used in the accident prediction algorithm. The default accident severity and accident type distributions in tables 1 and 2 are based on data from the FHWA Highway Safety Information System (HSIS) for Illinois, Michigan, Minnesota, and North Carolina. These default distributions for accident severity and accident type can be replaced with data suitable for the rural two-lane highway system of a particular highway agency as part of the calibration process described in section 3 and appendix D of this report. Use of distributions applicable to a specific State or geographic region is particularly appropriate because some percentages in the tables, such as the percentage of animal-related accidents on roadway segments in table 2, clearly vary geographically.

Strengths and Weaknesses of this Approach

The strengths of the accident prediction algorithms formulated as shown in equations (2) and (3) are as follows:

- The accident prediction algorithms make quantitative estimates of accident frequency.
- The base models serve as scale factors to assure that the magnitude of the predicted accident frequency is appropriate, while the AMFs assure that the predicted accident frequency is sensitive to site-specific geometric design and traffic control features.
- The use of AMFs that are separate from the base models assures that the effects of individual geometric design and traffic control features are not dependent on inappropriate regression coefficients that are too large, too small, or in the wrong direction. Each AMF has been developed by a panel of experts to represent the

Table 1. Default Distribution for Accident Severity Level on Rural Two-Lane Highways.

	Percentage of total accidents			
Accident severity level	Roadway segments[a]	Three-leg STOP-controlled intersections[b]	Four-leg STOP-controlled intersections[b]	Four-leg signalized intersections[b]
Fatal	1.3	1.1	1.9	0.4
Incapacitating Injury	5.4	5.0	6.3	4.1
Nonincapacitating injury	10.9	15.2	12.8	12.0
Possible injury	14.5	18.5	20.7	21.2
Total fatal plus injury	32.1	39.8	41.7	37.7
Property damage only	67.9	60.2	58.3	62.3
TOTAL	100.0	100.0	100.0	100.0

[a] Based on HSIS data for Illinois (1992), Michigan (1995), Minnesota (1996), and North Carolina (1995).

[b] Based on HSIS data for Michigan (1995) and Minnesota (1996).

Table 2. Default Distribution for Accident Type and Manner of Collision on Rural Two-Lane Highways.

Accident type and manner of collision	Percentage of total accidents			
	Roadway segments[a]	Three-leg STOP-controlled intersections[b]	Four-leg STOP-controlled intersections[b]	Four-leg signalized intersections[b]
SINGLE-VEHICLE ACCIDENTS				
Collision with animal	30.9	2.1	0.6	0.3
Collision with bicycle	0.3	0.7	0.3	1.0
Collision with parked vehicle	0.7	0.1	0.1	0.1
Collision with pedestrian	0.5	0.4	0.2	1.3
Overturned	2.3	2.1	0.6	0.4
Ran off road	28.1	10.4	4.5	1.9
Other single-vehicle accident	3.6	3.9	1.4	1.6
Total single-vehicle accidents	66.3	19.7	7.7	6.6
MULTIPLE-VEHICLE ACCIDENTS				
Angle collision	3.9	29.8	51.4	28.5
Head-on collision	1.9	2.0	1.4	1.8
Left-turn collision	4.2	6.4	5.9	9.0
Right-turn collision	0.6	0.4	0.2	0.4
Rear-end collision	13.9	26.2	17.2	36.2
Sideswipe opposite-direction collision	2.4	2.9	1.7	2.0
Sideswipe same-direction collision	2.6	4.5	4.4	5.5
Other multiple-vehicle collision	4.1	8.1	10.1	10.0
Total multiple-vehicle accidents	33.7	80.3	92.3	93.4
TOTAL ACCIDENTS	100.0	100.0	100.0	100.0

[a] Based on HSIS data for Illinois (1992), Michigan (1995), Minnesota (1996), and North Carolina (1995).

[b] Based on HSIS data for Michigan (1995) and Minnesota (1996).

best information currently available on the safety effects of that particular geometric design or traffic control feature.

- Equations (2) and (3) give the accident prediction algorithm a modular structure. Individual base models or AMFs can be easily replaced as better information becomes available.

The potential weaknesses of the structure for the accident prediction algorithms based on equations (2) and (3) are as follows:

- Accident frequencies are known to vary from agency to agency, even between roads that are nominally similar, because of differences in factors such as accident reporting thresholds, accident reporting practices, animal populations, driver population, and climate. However, such variations are not reflected in the base models that were each developed with data for only one or two States. For this reason, a calibration procedure has been provided in this report to allow highway agencies to adapt the accident prediction algorithm to their own local safety conditions. This calibration procedure involves the estimation of calibration factors appropriate for a particular highway agency that can be incorporated directly in equations (2) and (3).

- The accident prediction algorithm, as formulated in equations (2) and (3), is based on data for many locations and on expert judgment, but does not take advantage of knowledge of the actual accident history of the location being evaluated. Actual accident history data should be available for many existing locations evaluated with the accident prediction algorithm. For this reason, a procedure based on the Empirical Bayes (EB) approach is provided in this report to combine the results of the accident prediction algorithm with actual site-specific accident history data.

- The use of separate AMFs for each geometric design and traffic control element treats the safety effects of these individual elements as independent and ignores potential interactions between them. It is likely that such interactions exist and, ideally, they should be accounted for in the accident prediction algorithm. However, such interactions are poorly understood and none could be quantified by the expert panels that participated in this research. It is the assessment of the expert panels that the AMFs presented in this report represent the current state of knowledge about the safety effects of geometric design and traffic control elements and cannot be improved without further research. If future research leads to a better understanding of interactions between the safety effects of various geometric features, those research results can then be incorporated in the accident prediction algorithm.

Structure of the Accident Prediction Algorithm

The structure of the accident prediction algorithm, including base models, accident modification factors, calibration factors, and EB procedure is illustrated in figure 1. The flow diagram in figure 1 addresses the application of the accident prediction algorithm to a single roadway segment or at-grade intersection. Section 6 of the report illustrates the application of the algorithm to a project made up of numerous roadway segments and intersections.

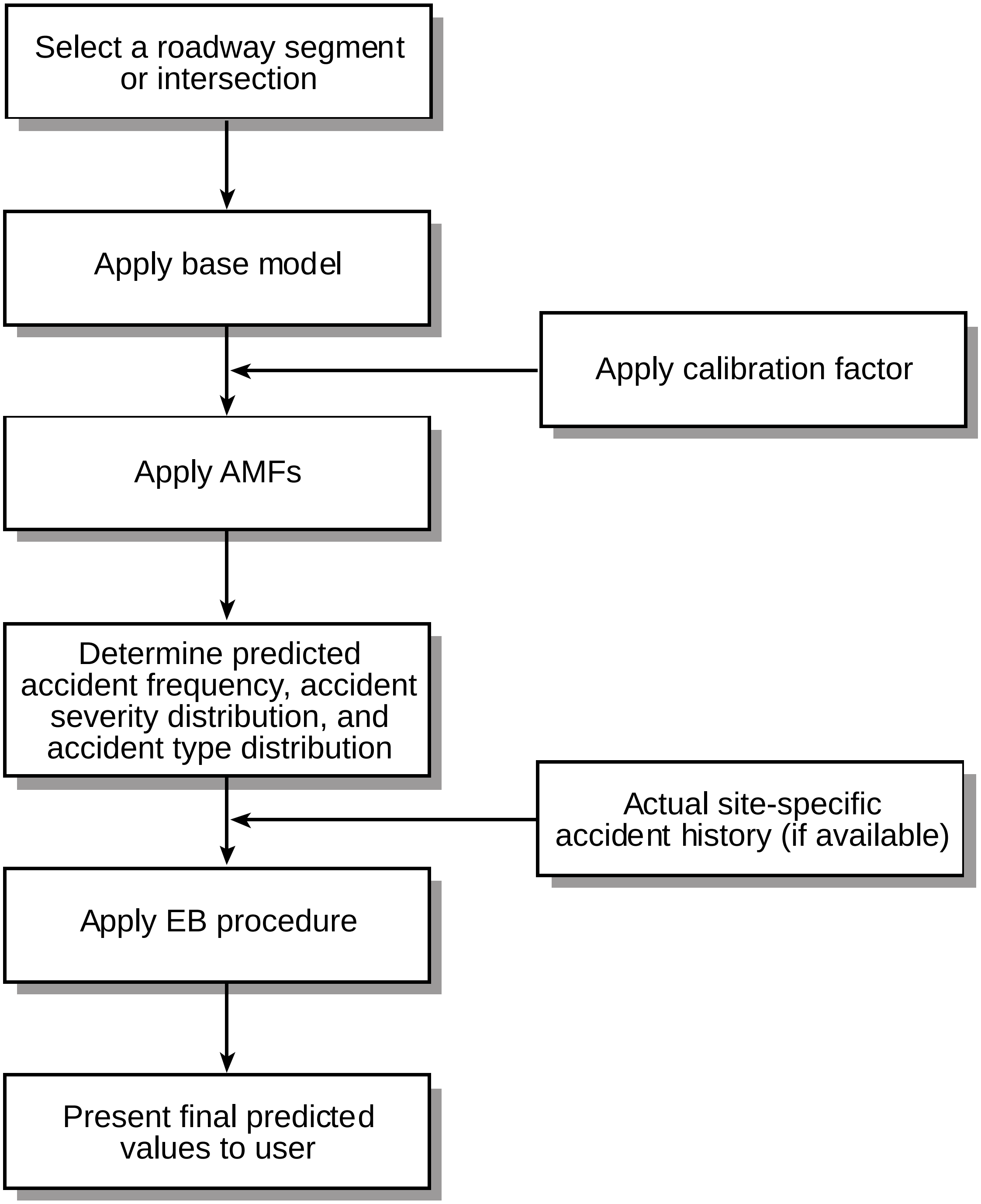

Figure 1. Flow Diagram of the Accident Prediction Algorithm for a Single Roadway Segment or Intersection.

3. BASE MODELS

This section of the report presents the base models used as part of the accident prediction algorithm. The base models for roadway segments and at-grade intersection are addressed separately in the following discussion. The base models were developed in separate studies by Vogt and Bared.[3,4,5] The development of the base models and the choices made among alternative models that were considered are presented in appendix B.

Base Model for Roadway Segments

The base model for roadway segments is presented below:

$$N_{br} = EXPO\ \exp(0.6409 + 0.1388\,STATE - 0.0846\,LW - 0.0591\,SW + 0.0668\,RHR + 0.0084\,DD)$$
$$\left(\Sigma WH_i\ \exp(0.0450\,DEG_i)\right)\ \left(\Sigma WV_i\ \exp(0.4652\,V_i)\right)\ \left(\Sigma WG_i\ \exp(0.1048\,GR_i)\right) \quad \textbf{(5)}$$

where: N_{br} = predicted number of total accidents per year on a particular roadway segment;

EXPO = exposure in million vehicle-miles of travel per year = (ADT)(365)(L)(10^{-6});

ADT = average daily traffic volume (veh/day) on roadway segment;

L = length of roadway segment (mi);

STATE = location of roadway segment (0 in Minnesota, 1 in Washington);

LW = lane width (ft); average lane width if the two directions of travel differ;

SW = shoulder width (ft); average shoulder width if the two directions of travel differ;

RHR = roadside hazard rating; this measure takes integer values from 1 to 7 and represents the average level of hazard in the roadside environment along the roadway segment. (For definitions of the roadside hazard rating categories, see appendix D; for the development of the roadside hazard ratings, see Zegeer.[6]);

DD = driveway density (driveways per mi) on the roadway segment;

WH_i = weight factor for the i^{th} horizontal curve in the roadway segment; the proportion of the total roadway segment length represented by the portion of the i^{th} horizontal curve that lies within the segment. (The weights, WH_i, must sum to 1.0.);

DEG_i = degree of curvature for the i^{th} horizontal curve in the roadway segment (degrees per 100 ft);

WV_j = weight factor for the j^{th} crest vertical curve in the roadway segment; the proportion of the total roadway segment length represented by the portion of the j^{th} crest vertical curve that lies within the segment. (The weights, WV_j, must sum to 1.0.);

V_j = crest vertical curve grade rate for the j^{th} crest vertical curve within the roadway segment in percent change in grade per 31 m (100 ft) = $|g_{j2}-g_{j1}|/l_j$;

g_{j1}, g_{j2} = roadway grades at the beginning and end of the j^{th} vertical curve (percent);

l_j = length of the j^{th} vertical curve (in hundreds of feet);

WG_k = weight factor for the k^{th} straight grade segment; the proportion of the total roadway segment length represented by the portion of the k^{th} straight grade segment that lies within the segment. (The weights, WG_k, must sum to 1.0.); and

GR_k = absolute value of grade for the k^{th} straight grade on the segment (percent).

This model was developed with negative binomial regression analysis for data from 619 rural two-lane highway segments in Minnesota and 712 roadway segments in Washington obtained from the FHWA HSIS. These roadway segments including approximately 1,130 km (700 mi) of two-lane roadways in Minnesota and 850 km (530 mi) of roadways in Washington. The database available for model development included 5 years of accident data (1985-1989) for each roadway segment in Minnesota and 3 years of accident data (1993-1995) for each roadway segment in Washington. The model predicts the total non-intersection accident frequency for any roadway segment for which the independent variables shown in equation (5) are known. The model predictions are reliable only within the ranges of independent variables for which data were available in the database used to develop the model (see table 30 in appendix B).

When the accident prediction model is employed to predict the expected accident experience for any specified roadway section, equation (5) is used in the following manner:

- The exposure variable (EXPO) in million vehicle-miles of travel is computed using the actual ADT and segment length (L) for the roadway section and a duration of 1 year (365 days). This assures the accident frequency predicted by the base model has units of accidents per year.

- The STATE variable in base model is set equal to zero, representing Minnesota conditions. This is done for consistency with the base models for three- and four-leg STOP-controlled intersections, both of which are based solely on Minnesota data. It should be noted that the calibration procedure described later in this chapter can be used to adapt the base models to the safety conditions of any State other than Minnesota. Calibration would even be desirable to apply the algorithm in Minnesota to a time period other than the period for which the base models were developed.

- The remaining variables in the model are set to the following nominal or base conditions:

Lane width (LW)	3.6 m (12 ft)
Shoulder width (SW)	1.8 m (6 ft)
Roadside hazard rating (RHR)	3
Driveway density (DD)	3 driveways per km (5 driveways per mi)
Horizontal curvature	None
Vertical curvature	None
Grade	Level (0 percent)

With the default values given above, the base model in equation (5) reduces to:

$$N_{br} = (ADT)\ (L)\ (365)\ (10^{-6})\ \exp(-0.4865) \qquad \textbf{(6)}$$

Base Models for At-Grade Intersections

Base models have been developed for three types of at-grade intersections on rural two-lane highways. These are:

- Three-leg intersections with STOP control on the minor-road approach.
- Four-leg intersections with STOP control on the minor-road approach.
- Four-leg signalized intersections.

The base models for each of these intersection types predict total accident frequency per year for intersection-related accidents within 76 m (250 ft) of a particular intersection.

These models address intersections that have only two lanes on both the major- and minor-road legs. Intersections on multilane highways or intersections between a two-lane highway and a multilane highway may be addressed in a future improvement to the accident prediction algorithm. The base models for each of the three intersection types are presented below.

Three-Leg STOP-Controlled Intersections

The base model for three-leg intersections with STOP control on the minor-road leg is presented below:

$$N_{bi} = \exp(-11.28 + 0.79\,ln\ ADT_1 + 0.49\,ln\ ADT_2 + 0.19\,RHRI + 0.28\,RT) \qquad \textbf{(7)}$$

where:

where:	ADT_1	=	average daily traffic volume (veh/day) on the major road;
	ADT_2	=	average daily traffic volume (veh/day) on the minor road;
	RHRI	=	roadside hazard rating within 76 m (250 ft) of the intersection on the major road [see description of the variable RHR in equation (5)]; and
	RT	=	presence of right-turn lane on the major road (0 = no right-turn lane present; 1 = right-turn lane present).

This model was developed with negative binominal regression analysis from data for 382 three-leg STOP-controlled intersections in Minnesota. The data base available for model development included 5 years of accident data (1985-1989) for each intersection. The model predicts the total intersection-related accident frequency for any three-leg STOP-controlled intersection for which the independent variables shown in equation (7) are known. The model predictions are reliable only within the ranges of independent variables for which data were available in the data base used to develop the model (see table 31 in appendix B).

When the accident prediction model is employed to predict the expected accident frequency for any specified three-leg STOP-controlled intersection on a two-lane highway, equation (7) is used in the following manner:

- The traffic volume variables (ADT_1 and ADT_2) are set equal to the actual ADTs of the major- and minor-road legs. If the ADTs differ between the two major-road legs, they should be averaged.

- The remaining variables in the model should be set equal to the following nominal or base conditions:

Roadside hazard rating (RHRI)	2
Presence of right-turn lane on the major road (RT)	None present (0)

With the default values of given above, the base model in equation (7) reduces to:

$$N_{bi} = \exp(-10.9 + 0.79\,ln\ ADT_1 + 0.49\,ln\ ADT_2) \quad \textbf{(8)}$$

Four-Leg STOP-Controlled Intersections

The base model for four-leg intersections with STOP control is presented below:

$$N_{bi} = \exp(-9.34 + 0.60\,ln\ ADT_1 + 0.61\,ln\ ADT_2 + 0.13\ ND_1 - 0.0054\,SKEW_4) \quad \textbf{(9)}$$

where:

ND_1 = number of driveways on the major-road legs within 76 m (250 ft) of the intersection; and

$SKEW_4$ = intersection angle (degrees) expressed as one-half of the angle to the right minus one-half of the angle to the left for the angles between the major-road leg in the direction of increasing stations and the right and left legs, respectively.

This model was developed with negative binominal regression from data for 324 four-leg STOP-controlled intersections in Minnesota. The database available for model development included 5 years of accident data (1985-1989) for each intersection. The model predicts the total intersection-related accident frequency for any four-leg STOP-controlled intersection for which the independent variables shown in equation (9) are known. The model predictions are reliable only within the ranges of independent variables for which data were available in the database used to develop the model (see table 38 in appendix B).

When the accident prediction model is employed to predict the expected accident frequency for any specified four-leg STOP-controlled intersection on a two-lane highway, equation (9) is used in the following manner:

- The traffic volume variables (ADT_1 and ADT_2) are set equal to the actual ADTs of the major- and minor-road legs, respectively. If the ADTs differ between either the two major- or minor-road legs, they should be averaged.

- The remaining variables in the model should be set equal to the following nominal or base conditions:

Number of driveways within 76 m (250 ft) of the intersection on the major road (ND_1)	No driveways
Intersection skew angle ($SKEW_4$)	0 degrees

With the default values of ND_1 and $SKEW_4$ given above, the base model in equation (9) reduces to:

$$N_{bi} = \exp(-9.34 + 0.60 \, ln \; ADT_1 + 0.61 \, ln \; ADT_2) \tag{10}$$

Four-Leg Signalized Intersections

The base model for four-leg signalized intersections is presented below:

$$\begin{aligned} N_{bi} = \exp(&-5.46 + 0.60 \, ln \; ADT_1 + 0.20 \, ln \; ADT_2 - 0.40 PROTLT - 0.018 PCTLEFT_2 \\ &+ 0.11 VEICOM + 0.026 PTRUCK + 0.041 ND_1) \end{aligned} \tag{11}$$

where:

PROTLT	=	presence of protected left-turn signal phase on one or more major-road approaches; = 1 if present; = 0 if not present
$PCTLEFT_2$	=	percentage of minor-road traffic that turns left at the signal during the morning and evening hours combined
VEICOM	=	grade rate for all vertical curves (crests and sags) within 76 m (250 ft) of the intersection along the major and minor roads
PTRUCK	=	percentage of trucks (vehicles with more than four wheels) entering the intersection for the morning and evening peak hours combined
ND_1	=	number of driveways within 76 m (250 ft) of the intersection on the major road.

This model was developed with negative binominal regression from data for 49 four-leg signalized intersections, 18 in California and 31 in Michigan. The data base available for model development included three years of accident data (1993-1995) for each intersection. The model predicts total intersection-related accident frequency for any four-leg signalized intersection for which the independent variables shown in equation (11) are known. The model predictions are reliable only within the ranges of independent variables for which data were available in the data base used to develop the model (see table 44 in appendix B).

When the accident prediction model is employed to predict the expected accident frequency for any specified four-leg intersection on a two-lane highway, equation (11) is used in the following manner:

- The traffic volume variables (ADT_1 and ADT_2) are set equal to the actual ADTs of the major- and minor-road legs, respectively. If the ADTs differ between either the major- or minor-road legs, they should be averaged.

- The remaining variables in the model should be set equal to the following nominal or base conditions:

Presence of protected left-turn signal phase (PROTLT)	No left-turn phase
Percentage of minor-road traffic turning left ($PCTLEFT_2$)	28.4 percent
Grade rate for vertical curves within 76 m (250 ft) of the intersection (VEICOM)	No vertical curves
Percentage of trucks entering the intersection (PTRUCK)	9.0 percent
Number of driveways within 76 m (250 ft) of the intersection on the major road (ND_1)	0 driveways

With the nominal or base values of PROTLT, $PCTLEFT_2$, VEICOM, and PTRUCK given above, the base model in equation (11) reduces to:

$$N_{bi} = \exp(-5.73 + 0.60\,ln\ ADT_1 + 0.20\,ln\ ADT_2) \qquad \textbf{(12)}$$

Calibration Procedure

The accident prediction algorithm is intended for use by highway agencies throughout the United States. Accident frequencies, even for nominally similar roadway sections or intersections, are known to vary widely from agency to agency. These variations are of two types, those that can be directly accounted for by the accident prediction algorithm and those that cannot.

States differ markedly both in terrain and in the history of the development of their highway system, resulting in state-to-state differences in roadway alignment, cross section, and intersection design. However, differences of this type can be accounted for by the AMFs in the accident prediction algorithm.

States also differ markedly in climate, animal population, driver populations, accident reporting threshold, and accident reporting practices. These variations may result in some States experiencing substantially more reported traffic accidents on rural two-lane highways than others. Such variations cannot be directly accounted for by the accident prediction algorithm. Therefore, a calibration procedure has been developed to allow highway agencies to adjust the accident prediction algorithm to suit the safety conditions present in their State.

The calibration procedure is implemented by a highway agency by determining the value of calibration factors for roadway segments and at-grade intersections from comparison of their own data to estimates from the accident prediction algorithm. The calibration factors are incorporated in equations (13) and (14) in the following fashion for roadway segments and at-grade intersections, respectively:

$$N_{rs} = N_{br}\ C_r\ (AMF_{1r}\ AMF_{2r}\ ...\ AMF_{nr}) \tag{13}$$

$$N_{int} = N_{bi}\ C_i\ (AMF_{1i}\ AMF_{2i}\ ...\ AMF_{ni}) \tag{14}$$

where:

C_r = calibration factor for roadway segments developed for use by a particular highway agency; and

C_i = calibration factor for at-grade intersections developed for use by a particular highway agency.

The calibration factors (C_r and C_i) will have values greater than 1.0 for highway agencies whose roadways, on the average, experience more accidents than the roadways used in the development of the accident prediction algorithm. The calibration factors for highway agencies whose roadways, on the average, experience fewer accidents than the roadways used in the development of the accident prediction algorithm will have values less than 1.0. The calibration factor for at-grade intersections (C_i) may have different values for each of the three intersection types for which base models have been developed. The calibration procedures for application by highway agencies is presented in appendix C.

It is generally expected that the calibration factors (C_r and C_i) would be determined by highway agencies based on statewide data. In larger and more diverse States, a highway agency might choose to develop separate calibration factors for individual highway districts or climate regions. It is also possible for users to provide a local calibration factor for smaller areas with distinct driver populations or climate conditions. However, use of the local calibration factor would require a special study to determine the safety performance of roads in that specific local area relative to the statewide or district-wide expected values.

In addition to estimates of accident frequency, the accident prediction algorithm includes default distributions of accident severity and accident type for rural two-lane highway roadway sections and intersections. These default distributions have been presented in tables 1 and 2 of this report. The calibration procedure presented in appendix C includes a capability for highway agencies who use the accident prediction algorithm to modify the default distributions of accident severity and accident type to match their own experience on rural two-lane highways.

4. ACCIDENT MODIFICATION FACTORS

The incremental effects of individual geometric design and traffic control elements are represented in the accident prediction algorithm by AMFs. This section describes the development of the AMFs and documents the AMFs for each geometric design and traffic control factor considered in the algorithm.

Development of Accident Modification Factors

AMFs are used in the accident prediction algorithm to represent the effects on safety of specific geometric design and traffic control features. The AMF for the nominal or base value of each geometric design traffic control feature has a value of 1.0. Any feature associated with higher accident experience than the nominal or base condition has an AMF with a value greater than 1.0; any feature associated with lower accident experience than the base condition has an AMF with a value less than 1.0. The multiplicative nature of the AMFs is illustrated in equations (13) and (14).

The AMFs were developed by two expert panels, one for roadway sections and one of at-grade intersections. These panels exercised expert judgment in reviewing the reported research findings concerning each geometric design and traffic control feature of interest and selecting an appropriate basis for an AMF. The members of the two expert panels that developed the AMFs are identified in appendix A.

Each expert panel selected a set of geometric and traffic control elements, including both roadway segment and at-grade intersection elements, as candidates for the development of AMFs. The candidates were selected based on the panel's initial assessments of those roadway segment and intersection features that are generally considered to be related to safety. A critical review of published and unpublished safety literature related to each geometric design and traffic control element was then undertaken. Each panel met and used the findings of the literature review as the basis for (1) selecting the final set of geometric and traffic control elements for which AMFs could be developed; and (2) quantifying those AMFs. For roadway segments, the final AMFs included all of the variables in the roadway segment base models plus additional variables. For at-grade intersections, the final AMFs did not include all of the variables in the intersection base models because the expert panel found that reliable estimates of safety effects from the literature were lacking for some variables in the base models and others were judged to be of relatively less importance.

For some geometric design or traffic control elements, the expert panel selected the results of one particular study that they considered most credible to serve as the basis for the AMF. In other cases, the expert panel combined the results of two or more studies to develop an AMF. In other cases, where reliable research results were lacking, the panel exercised its collective judgment to estimate values for an appropriate AMF; this was done, for example, where the panel felt comfortable in establishing a bound, such as an appropriate maximum or minimum value, for an AMF.

AMFs were based on a variety of sources including results of before-and-after accident evaluations, coefficients or parameter values from regression models, and expert judgment. The expert panel considered well-designed before-and-after evaluations to be the best source for AMFs. However, relatively few well-designed before-and-after studies of geometric design elements were found in the literature and, therefore, the expert panel had to rely in many cases on other types of studies. Coefficients or parameter values from regression models are considered less reliable, but were used when no before-and-after study results were available and the value of the coefficient in question was considered by the panel to be credible. Expert judgment alone was exercised in limited cases where no better results were available; even where an AMF was based primarily on expert judgment, the panel used all relevant research results in making that judgment. FHWA has many promising before-and-after evaluations underway, and it is hoped that the AMFs recommended here will be updated with those analysis results when they become available.

The expert panels faced many difficult judgments in determining the appropriate values of the AMFs. A broad range of literature was reviewed and many studies were assessed. Many of the studies reviewed were similar in quality, and the selection of one study over another may have depended as much on consistency with other selected AMFs as on the relative merits of the studies. Thus, the omission of any particular study from the final AMFs should not necessarily be interpreted as disparagement of that particular study. The sources on which each AMF is based are documented in the following discussion. Section 8 of this report not only identifies the sources cited in the text but also includes a complete bibliography of all sources consulted in the development of the AMFs.

While the individual AMFs were based on the panel's best judgment about the relative merits of the available research findings, the credibility of the model is supported by a sensitivity analysis whose results are presented in section 5 of this report.

The AMFs incorporated in the model include:

Roadway Segments

- Lane width.
- Shoulder width.
- Shoulder type.
- Horizontal curves:
    - length;
    - radius;
    - presence or absence of spiral transitions;

- ! superelevation.
- Grades.
- Driveway density.
- Two-way left-turn lanes.
- Passing lanes/short four-lane sections.
- Roadside design.

At-Grade Intersections

- Skew angle.
- Traffic control.
- Exclusive left-turn lanes.
- Exclusive right-turn lanes.
- Intersection sight distance.

An explanation of each AMF for roadway sections and for at-grade intersections is presented below.

Roadway Segments

The AMFs for geometric design and traffic control features of roadway segments are presented below.

Lane Width

The nominal or base value of lane width is 3.6 m (12 ft). Thus, 3.6-m (12-ft) lanes are assigned an AMF of 1.00. Figure 2 illustrates the recommended values of the AMF for lane widths from 2.7 to 3.6 m (9 to 12 ft). The AMF for any lane widths within the range of 2.7 to 3.6 m (9 to 12 ft) would be interpolated between the lines shown in figure 2. Lanes less than 2.7 m (9 ft) in width would be assigned an AMF equal to that for 2.7-m (9-ft) lanes. Lanes greater than 3.6 m (12 ft) in width would be assigned an AMF equal to that for 3.6 m (12-ft) lanes. As shown in the figure, the AMFs for lanes less than 3.6 m (12 ft) in width would be constant for all ADTs above 2,000 veh/day, but would decrease to a substantially smaller value over the range of traffic volumes between 400 and 2,000 veh/day. The AMFs then have constant, but lower, values in the range of ADT below 400 veh/day.

If the lane widths for the two directions of travel on a roadway segment differ, the AMF should be determined separately for the lane width in each direction of travel and the resulting AMFs should then be averaged.

The AMFs shown in figure 2 apply to single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents. The AMFs expressed on this basis must, therefore, be adjusted to total accidents within the accident prediction algorithm. This can be accomplished with the following equation:

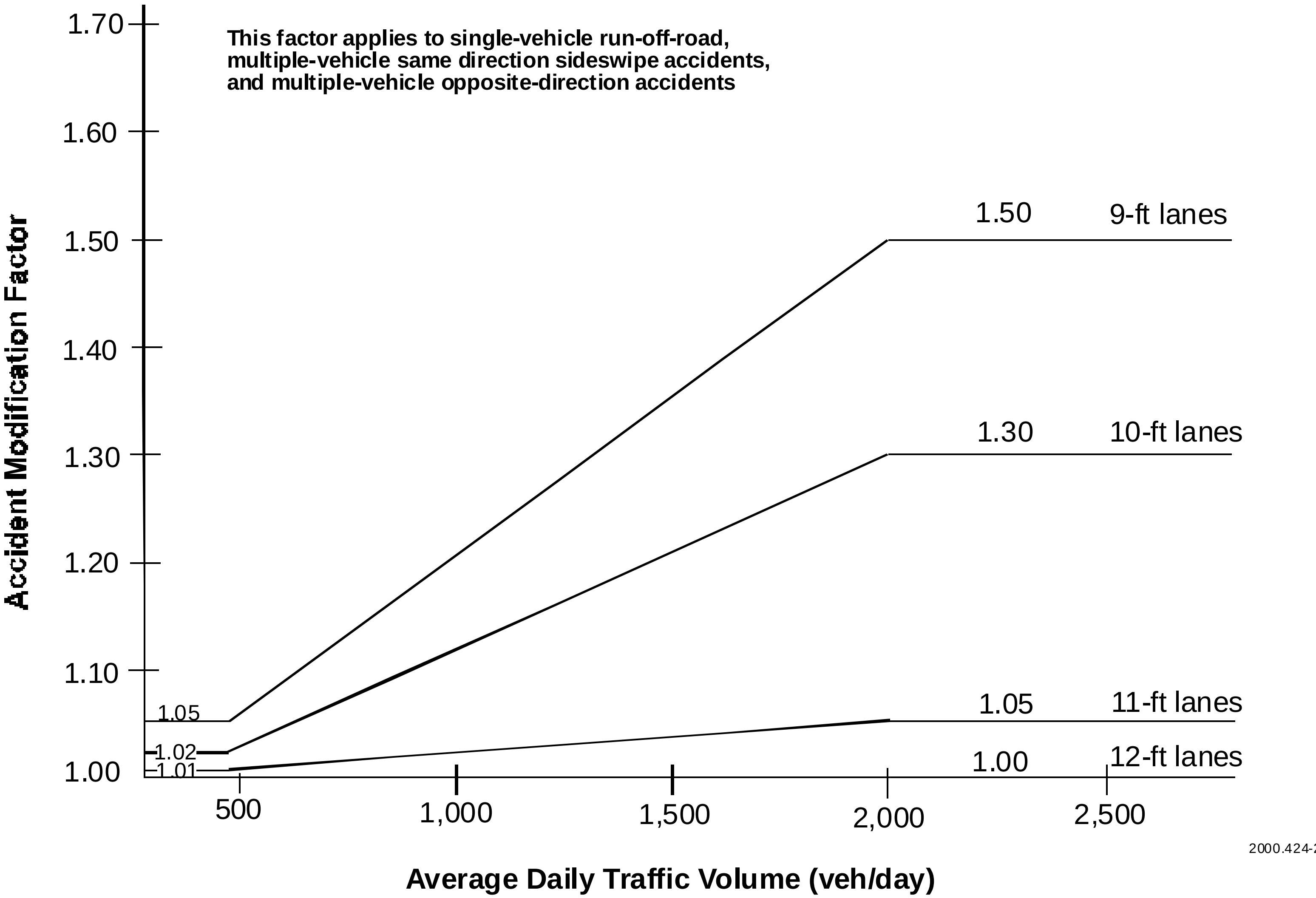

Figure 2. Recommended Accident Modification Factor for Lane Width.

$$AMF = (AMF_{ra} - 1.0)\, P_{ra} + 1.0 \tag{15}$$

where:

AMF	=	accident modification factor for total accidents;
AMF_{ra}	=	accident modification factor for related accidents (i.e., single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents), such as the accident modification factor for lane width shown in figure 2;
P_{ra}	=	proportion of total accidents constituted by related accidents.

The proportion of related accidents (P_{ra}) is estimated as 0.35 (i.e., 35 percent) based on the default distribution of accident types presented in table 2. This default accident type distribution, and therefore the value of P_{ra}, may be changed by a highway agency as part of the calibration process.

The AMFs for lane width for roadways with ADT greater than 2,000 veh/day are based on the results of Zegeer et al.(6) Furthermore, these values are reasonably consistent with the results of Zegeer et al.(7,8) and Miaou.(9) The AMF of 1.05 for 3.3-m (11-ft) lane widths on roadways with ADTs over 2,000 veh/day has been adjusted to a lower value than that indicated by Zegeer et al.(6) based on the expert panel's assessment of a broader set of studies concerning the safety performance of roadways with 3.3-m (11-ft) lanes. The AMFs for lane widths on roadways with ADTs less than 400 veh/day are based on the results of Griffin and Mak.(10) The transition lines in the ADT range from 400 to 2,000 veh/day are based on a judgment by the expert panel.

Shoulder Width and Type

The nominal or base value of shoulder width and type is a 1.8-m (6-ft) paved shoulder, which is assigned an AMF value of 1.00. Figure 3 illustrates the recommended AMF for shoulder widths that differ from 1.8 m (6 ft). Another AMF, presented below, adjusts for differences between gravel, turf, or composite shoulders and paved shoulders. The modification factors in figure 3 illustrate that, for ADTs above 2,000 veh/day, the effect of a 0.6-m (2-ft) change in shoulder width is equivalent to the effect of a 0.3-m (1-ft) change in lane width. For ADTs below 400 veh/day, the effect of a 0.3-m (1-ft) change in shoulder width is equivalent to the effect of a 0.3-m (1-ft) change in lane width. A linear transition between these effects occurs over the range of ADTs from 400 to 2,000 veh/day. AMFs for shoulder widths between 0 and 2.4 m (0 and 8 ft) should be interpolated between the lines in figure 3. Shoulders greater than

2.4 m (8 ft) in width should be assigned AMFs equal to those for 2.4 m (8 ft). The AMFs shown in figure 3 apply only to single-vehicle run-off-the-road and opposite-direction accidents.

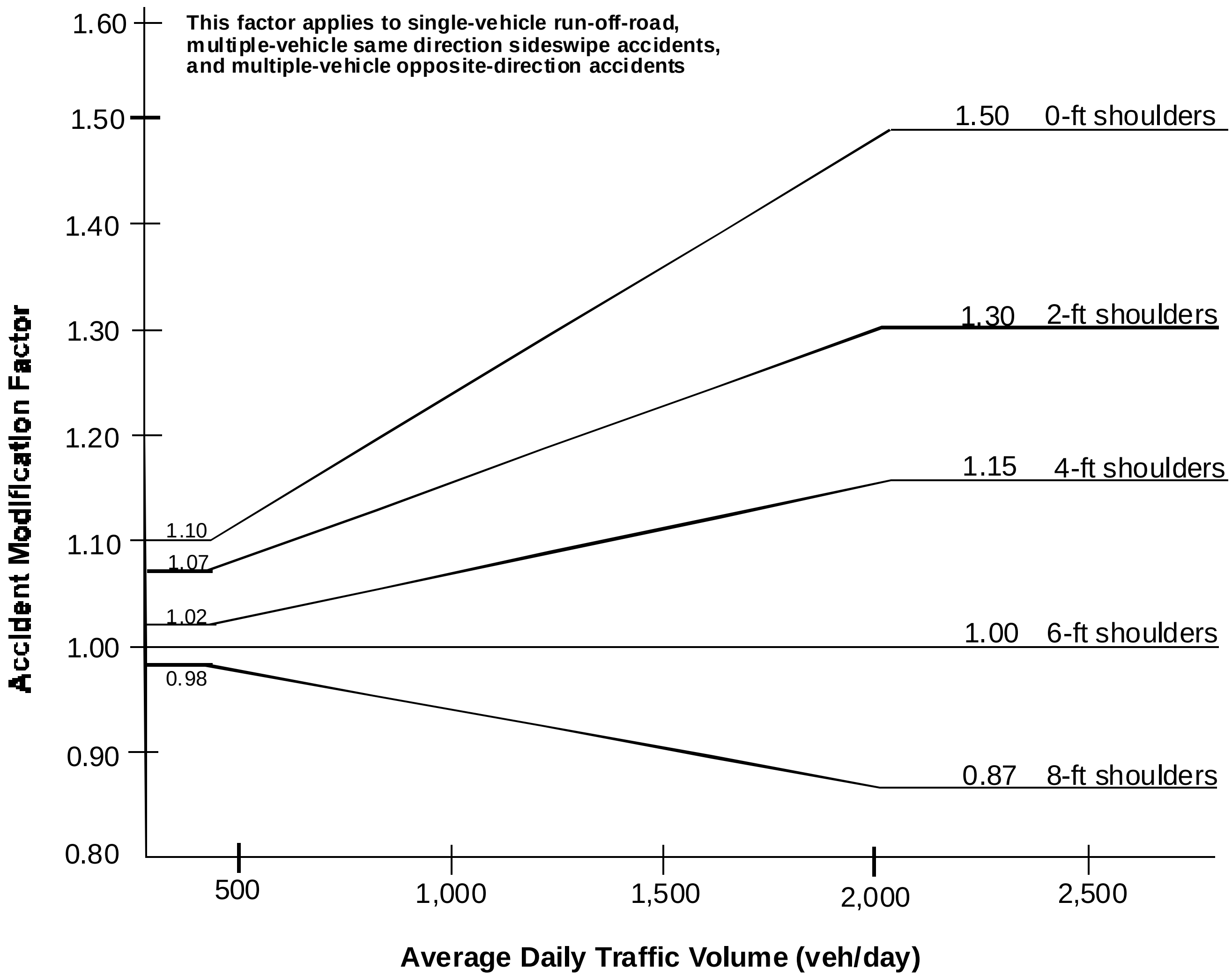

Figure 3. Accident Modification Factor for Shoulder Width.

The AMFs for shoulder width on rural two-lane highways with ADTs over 2,000 veh/day are based primarily on the results of Zegeer et al.[6] which suggests the 2:1 ratio between the effects of lane and shoulder widths. This finding is reasonably consistent with the results of Miaou[9,11] and Rinde.[12] The AMFs for roadways with ADTs less than 400 veh/day, which indicate a 1:1 ratio between the effects of lane width and shoulder width, are based on the work of Zegeer et al.[8] for low-volume roads. The transition curves in the ADT range from 400 to 2,000 veh/day are based on a judgment by the expert panel.

The nominal or base condition for shoulder type is the paved shoulder. Table 3 presents the recommended AMFs for gravel, turf, and composite shoulders as a function of shoulder width. The AMFs shown in table 3 apply to single-vehicle run-off-the-road and opposite-direction accidents.

Table 3. Accident Modification Factors for Shoulder Types on Two-Lane Highways.

Shoulder type	Shoulder width (ft)							
	0	1	2	3	4	6	8	10
Paved	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Gravel	1.00	1.00	1.01	1.01	1.01	1.02	1.02	1.03
Composite	1.00	1.01	1.02	1.02	1.03	1.04	1.06	1.07
Turf	1.00	1.01	1.03	1.04	1.05	1.08	1.11	1.14

Conversion: 1 ft = 0.305 m

Note: The values for composite shoulders in this table represent a shoulder for which 50 percent of the shoulder width is paved and 50 percent of the shoulder width is turf.

The panel made a judgment that the shoulder type effect observed by Miaou[9] was appropriate to characterize the difference between gravel and paved shoulders and that the effect observed by Zegeer et al.[6] was appropriate to characterize the difference between turf and paved shoulders. The composite shoulders represented by table 3 represent a shoulder for which 50 percent of the shoulder width is paved and 50 percent of the shoulder width is turf. The AMFs for composite shoulders are averages of the AMFs for paved and turf shoulders.

Table 3 assumes that the base condition for shoulder type is a paved shoulder. In fact, the data base used to develop the base model for roadway sections shown in equation (4) consisted of approximately 67 percent paved and composite shoulders and 33 percent gravel shoulders. However, the difference in safety performance between paved and gravel shoulders is so small that the magnitude of the AMFs is only minimally affected by the proportion of gravel shoulders in the data base. Therefore, the use of table 3 without modification is recommended. If, at any future time, the base model for roadway sections is replaced or updated, it is recommended that either (1) the model be based solely on roadway sections with paved shoulders; or (2) that shoulder type appear explicitly in the base model.

If the shoulder types and/or widths for the two directions of travel on a roadway segment differ, the AMF should be determined separately for the shoulder type and width in each direction of travel and the resulting AMFs should then be averaged.

The AMFs for shoulder width and type apply only to single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents. The AMFs expressed on this basis must, therefore, be adjusted to total accidents within the accident prediction algorithm. This can be accomplished with the following equation that is analogous to equation (15):

$$AMF = (AMF_{wra}\ AMF_{tra} - 1.0)\ P_{ra} + 1.0 \qquad \textbf{(16)}$$

where:

AMF_{wra} = accident modification factor for related accidents based on shoulder width (from figure 4); and

AMF_{tra} = accident modification factor for related accidents based on shoulder type (from table 3).

The proportion of related accidents (P_{ra}) is estimated as 0.35 (i.e., 35 percent) based on the default distribution of accident types presented in table 2. This default accident type distribution and therefore the value of P_{ra}, may be changed by a highway agency as part of the calibration process.

Horizontal Curves

Length, Radius, and Presence or Absence of Spiral Transitions

The nominal or base condition for horizontal alignment is a tangent roadway section. An AMF has been developed to represent the manner in which accident experience of curved alignments differs from that of tangents. This AMF applies to total roadway segment accidents, not just the related accident types considered above for lane and shoulder widths.

The AMF for horizontal curves has been determined from the regression model developed by Zegeer et al.[(13)] The Zegeer model includes the effects on accidents of length of horizontal curve, degree of horizontal curve, and presence or absence of spiral transition curves. The Zegeer model could also be used to introduce an effect of lane width on horizontal curves that differs from the effect of lane width on tangents. No data are available to represent any differential effect of shoulder width between horizontal curves and tangents.

The AMF for horizontal curvature is in the form of an equation and, thus, might be termed an accident modification function rather than an accident modification factor. The AMF for length, radius, and presence or absence of spiral transitions on horizontal curves is:

$$AMF = \frac{1.55 L_c + \frac{80.2}{R} - 0.012 S}{1.55 L_c} \qquad \textbf{(17)}$$

where: L_c = length of horizontal curve (mi);
R = radius of curvature (ft); and
S = 1 if spiral transition curve is present
0 if spiral transition curve is not present.

In applying the accident modification functions for curves with spiral transitions, the length variable (L_c) should represent the length of the circular portion of the curve.

Superelevation

The nominal or base condition for the AMF for the superelevation of a horizontal curve is the amount of superelevation required by the AASHTO Green Book.[14] The superelevation required by the AASHTO Green Book must be determined taking into account the value of maximum superelevation rate, e_{max}, established by highway agency policies. Policies concerning maximum superelevation rates for horizontal curves vary between highway agencies based on climate and other considerations. If no value of e_{max} specified by the particular highway agency has been incorporated in the IHSDM, then $e_{max} = 0.06$ will be assumed by default. The AMF for superelevation is based on the superelevation deficiency of a horizontal curve (i.e., the difference between the actual superelevation and the superelevation required by AASHTO policy). When the actual superelevation meets or exceeds that required by AASHTO policy, the value of the superelevation AMF is 1.00. The expert panel made a judgment that there would be no effect of superelevation deficiency on safety until the superelevation deficiency exceeds 0.01.

The general functional form of an AMF for superelevation is shown in figure 4, based on the work of Zegeer et al.[13,15] For a horizontal curve with 12-ft lanes and no spiral transitions, the Zegeer work suggests an AMF of the form:

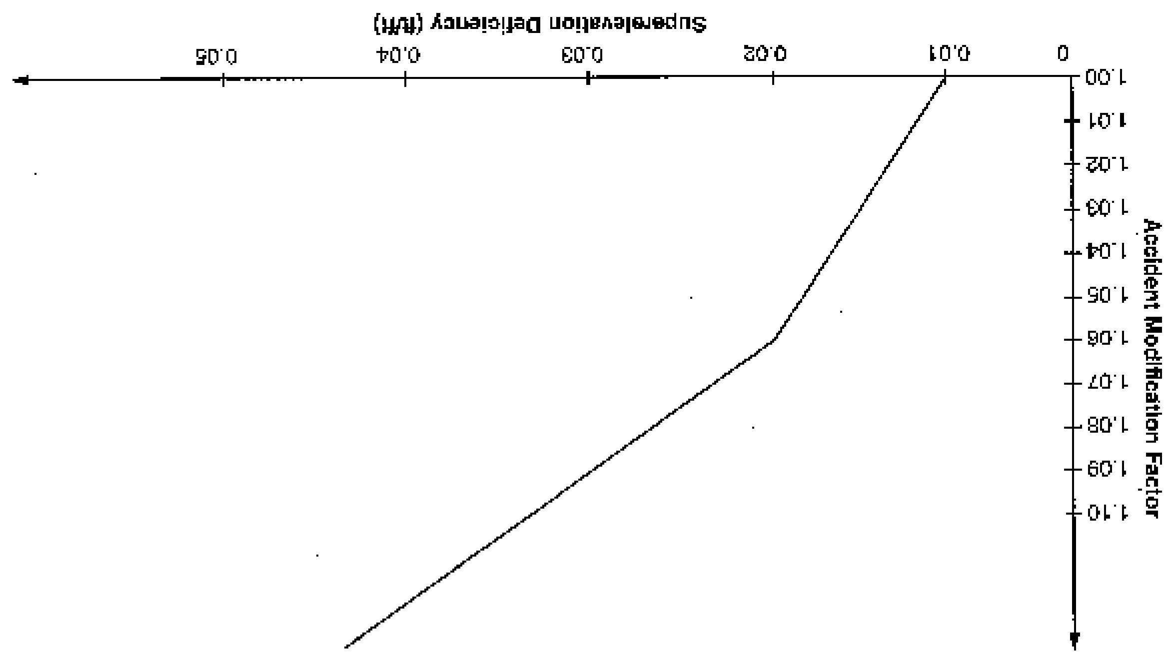

Figure 4. Accident Modification Factor for Superelevation Deficiency.

$$AMF = \frac{1.22 + \frac{1604}{R} + 9.52SD}{1.22 + \frac{1604}{R}} \tag{18}$$

where:

SD = superelevation deficiency.

However, the AMF in the form shown in equation (18) is not directly usable because it suggests that for any given superelevation deficiency, the value of AMF increases with increasing radius of curvature. In fact, it seems likely that the opposite should occur, with superelevation deficiencies being more important on curves with smaller radii.

Zegeer, et al. [15] indicate that the mean radius of horizontal curves in their study was 257.0 m (842.5 ft). For this mean radius, equation (18) indicates the following values of the AMF:

Superelevation deficiency	**AMF**
0.02	1.06
0.03	1.09
0.04	1.12
0.05	1.15

Based on these values and the expert panel's judgment that there is no effect on safety until the superelevation deficiency reaches 0.01, the following relationships which form the basis for figure 5 can be derived:

$$AMF = 1.00 \text{ for } SD < 0.01 \tag{19}$$

$$AMF = 1.00 + 6\,(SD - 0.01) \text{ for } 0.01 \leq SD < 0.02 \tag{20}$$

$$AMF = 1.06 + 3\,(SD - 0.02) \text{ for } SD \geq 0.02 \tag{21}$$

This AMF applies to total roadway segment accidents for roadway segments located on horizontal curves.

Grades

The nominal or base condition for grade is a level roadway (0% grade). Table 4 presents the accident modification factor for grades based on an analysis of two-lane highway grades in Utah conducted by Miaou.[16] This analysis considered accident and geometric data for approximately 4,000 km (2,500 mi) of two-lane roads with 88.5 km/h (55-mi/h) speed limits, 3.6-m (12-ft) lanes, and tangent alignment. Two analysis approaches were used: univariate analysis using smoothing techniques and negative binomial regression modeling. Both methods estimated the effect of vertical grade on accidents as approximately a 1.6 percent increase in accidents per 1 percent increase in grade. However, both studies found this effect to be not statistically significant.

Despite the lack of statistical significance of the results, a decision was reached by the expert panel to use the observed effect as the basis for an AMF because the result appeared reasonable to the expert panel and because no more reliable results are available. Table 4 presents AMFs for grade based on the observed 1.6 percent increase in accidents per 1 percent increase in percent grade. The AMFs in table 4 are applied to each individual grade section on the roadway being evaluated without respect to the sign of the grade. The sign of the grade is irrelevant because each grade on a two-lane highway is an upgrade for one direction of travel and a downgrade for the other. The grade factors are applied to the entire grade from one point of vertical intersectional (PVI) to the next (i.e., there is no special account taken of vertical curves). The AMFs in table 4 apply to total roadway segment accidents.

Table 4. Accident Modification Factors for Grade of Roadway Sections.

Grade (%)				
0	2	4	6	8
1.00	1.03	1.07	1.10	1.14

Note: This factor can be expressed as an effect of 1.6 percent per percent grade.

Driveway Density

The nominal or base condition for driveway density is three driveways per km (five driveways per mi). The AMF for driveway density is based on the following equation derived from the work of Muskaug:[17]

$$\text{AMF} = \frac{0.2 + [0.05 - 0.005\ ln\ (\text{ADT})]\ \text{DD}}{0.2 + [0.05 - 0.005\ ln\ (\text{ADT})]\ (5)} \qquad \textbf{(22)}$$

where:

ADT = annual average daily traffic volume of the roadway being evaluated (veh/day); and

DD = driveway density (driveways per mile).

The Muskaug study deals with injury accidents only but the expert panel made a judgment that the AMF shown in equation (22) can be applied to total roadway accidents of all severity levels.

The expert panel considered the Norwegian study by Muskaug to be the best available study on the safety effects of driveway density on rural two-lane highways. The panel was concerned, however, about reliance on an international data source and undertook a further review of relevant U.S. literature. The most applicable U.S. reference appeared to be *Transportation Research Circular 456*, and an analysis concluded that its results were consistent with the findings of Muskaug.[18] Still another review of HSIS data from Minnesota concluded that the effect of driveway density in accidents is consistent with the results of the Muskaug study. Therefore, the Muskaug results were retained as the basis for the driveway density AMF.

Passing Lanes

The nominal or base condition for passing lanes is the absence of a lane (i.e., the normal two-lane cross section). The AMF for a conventional passing or climbing lane added in one direction of travel on a two-lane highway is 0.75 for total accidents in both directions of travel over the length of the passing lane from the upstream end of the lane addition taper to the downstream end of the lane drop taper. This value assumes that the passing lane is operationally warranted and that the length of the passing lane is appropriate for the operational conditions on the roadway. An IHSDM procedure other than the accident prediction algorithm should be used to warn users if a passing lane is not operationally warranted or if an inappropriate passing lane length is used. Passing lanes are known to have traffic operational effects that extend 5 to 13 km (3 to 8 mi) downstream of the passing lane; while it might be presumed that these operational effects provide analogous safety benefits over a similar length of highway, no such effect is included in the accident prediction algorithm for lack of quantitative evidence of such a benefit.

The AMF for short four-lane sections (i.e., side-by-side passing lanes provided in opposite directions on the same roadway section) is 0.65 for total accidents over the length of the short four-lane section. This AMF applies to any portion of roadway where the cross section has four lanes and where both added lanes have been provided over a limited distance to increase passing opportunities. This AMF does not apply to extended four-lane highway sections.

The AMF for passing lanes is based primarily on the work of Harwood and St. John, with consideration also given to the results of Rinde and Nettleblad.[19, 12, 20] The AMF for short four-lane sections is based on the work of Harwood and St. John.[19] These AMFs apply to total roadway segment accidents within the passing lane and short four-lane sections.

Two-Way Left-Turn Lanes

The installation of a center two-way left-turn lane (TWLTL) on a two-lane highway to create a three-lane cross section can reduce accidents related to turning maneuvers at driveways. The AMF for installation of a TWLTL is:

$$AMF = 1 - 0.7 P_D \, P_{LT/D} \tag{23}$$

where:

P_D = driveway-related accidents as a proportion of total accidents; and
$P_{LT/D}$ = left-turn accidents susceptible to correction by a TWLTL as a proportion of driveway-related accidents.

The value of P_{AP} is estimated from the work of Hauer as:[21]

$$P_D = \frac{0.0047 DD + 0.0024 DD^2}{1.199 + 0.0047 DD + 0.0024 DD^2} \tag{24}$$

The value of $P_{LT/D}$ was estimated by the expert panel as 0.5.

The expert panel considers that equations (23) and (24) provides the best estimate of the AMF for TWLTL installation that can be made without data on the left-turn volumes within the TWLTL. Realistically, such volumes are seldom available to highway agencies for use in such analyses. The AMF, as adjusted in equation (23), applies to total roadway segment accidents. Equation (24) was initially developed to represent total access point density (driveways plus unsignalized intersections). However, it is used here to determine an AMF for driveway density alone, because the effects of left-turn lanes at intersections are considered separately below.

The AMF for TWLTL installation should not be applied unless the driveway density is greater than or equal to three driveways per km (five driveways per mi). If the driveway density is less than three driveways per km (five driveways per mi), the AMF for TWLTL installation is 1.00. TWLTL installation would, in any case, be inappropriate for roadway segments with driveway densities lower than this threshold.

Roadside Design

For purposes of the accident prediction algorithm, the quality of roadside design is represented by the roadside hazard rating (1 to 7 scale) developed by Zegeer et al.[6] No studies were found in the literature that presented satisfactory relationships between the roadside hazard rating and accident experience for two-lane highways. Therefore, the AMF for roadside design was derived directly from the base model for roadway sections presented in equation (5). The nominal or base value of roadside hazard rating employed in the base model for roadway sections is 3. The AMF is based on the ratio of the accident experience predicted by base model using the actual roadway section in question to the accident experience predicted by the base model using the nominal value of roadside hazard rating equal to 3. The AMF is:

$$AMF = \frac{\exp(-0.6869 + 0.0668\,RHR)}{\exp(-0.4865)} \qquad \textbf{(25)}$$

This AMF applies to total roadway segment accidents. Photographic examples and quantitative definitions for each roadside hazard rating (1 through 7) as a function of roadside design features such as side slope and clear zone width are presented in appendix D.

The expert panel encourages future development of AMFs for specific roadside design elements in as much detail as the roadway design factors in this accident prediction algorithm. For example, the algorithm could be made sensitive to the presence or absence of a guardrail at specific roadside slopes and at individual roadside obstacles. The Roadside Safety Analysis Program (RSAP) model currently being developed might be applied for this purpose rather than just as a benefit/cost tool for comparing roadside design alternatives.[22] However, the RSAP model is not yet complete and the consensus of the panel was that application of the RSAP model would be beyond the scope of this initial effort to develop the accident prediction algorithm.

At-Grade Intersections

The AMFs for geometric design and traffic control features of at-grade intersections are presented below. As explained above, AMFs have been developed only for those geometric design and traffic control features for which the expert panel found a suitable basis for quantifying an AMF.

Number of Intersection Legs

There is no separate AMF for the number of intersection legs. Instead, the effect of the number of intersection legs will be accounted for by the separate base models for three- and four-leg intersections like those presented in section 3 of this report. Base models have been developed for both three- and four-leg STOP-controlled intersections, while base models for signalized intersections have been developed for four-leg intersections only. No base models will be developed for intersections with more than four legs. Therefore, multi-leg intersections and three-leg signalized intersections will not be addressed by the initial version of the accident prediction algorithm.

Intersection Skew Angle

The nominal or base condition for intersection skew angle is 0 degrees of skew (i.e., an intersection angle of 90 degrees). The skew angle for an intersection was defined as the deviation from an intersection angle of 90 degrees and carries a positive or negative sign that indicates whether the minor road intersects the major road at an acute or obtuse angle. This sign was introduced into the base model because a Finnish study by Kulmala found that acute and obtuse skew angles affected safety differently.(23)

STOP-Controlled Intersections

The AMF for intersection angle at three-leg STOP-controlled intersections is derived from the base model for this intersection type. Because the intersection angle variable was not statistically significant at the 0.015 significance level, it did not appear in the base model in equation (7). Therefore, the base model presented in equation (49) in appendix B with additional variables, including the intersection angle variable, was used to derive the following AMF:

$$AMF = \frac{\exp\,(-12.15 + 1.001\,ln\ ADT_1 + 0.406\,ln\ ADT_2 + 0.0040 SKEW_3)}{\exp\,(-12.15 + 1.001\,ln\ ADT_1 + 0.406\,ln\ ADT_2)} \qquad \textbf{(26)}$$

where:

ADT_1 = average daily traffic volume for the major road;

ADT_2 = average daily traffic volume for the minor road; and

$SKEW_3$ = intersection angle (degrees) minus 90 for the angle between the major-road leg in the direction of increasing stations and a leg to the right; 90 minus intersection angle (degrees) for the angle between the major-road leg in the direction of increasing stations and a leg to the left.

Equation (26) reduces to:

$$AMF = \exp(0.0040\ SKEW_3) \quad \textbf{(27)}$$

The AMF for intersection angle at four-leg STOP-controlled intersections is based directly on the base model presented as equation (9) for this intersection type:

$$AMF = \frac{\exp(-9.15 + 0.534\,ln\ ADT_1 + 0.665\,ln\ ADT_2 - 0.0054\,SKEW_4)}{\exp(-9.15 + 0.534\,ln\ ADT_1 + 0.665\,ln\ ADT_2)} \quad \textbf{(28)}$$

$SKEW_4$ = intersection angle (degrees) expressed as one-half of the angle to the right minus one-half of the angle to the left for the angles between the major-road leg in the direction of increasing stations and the right and left legs, respectively

Equation (28) reduces to:

$$AMF = \exp(-0.0054\ SKEW_4) \quad \textbf{(29)}$$

The opposite signs of the coefficients of skew angle in equations (27) and (29) are a concern because the difference in sign implies that positive and negative skew angles (as defined above for available $SKEW_3$ and $SKEW_4$) have opposite effects on safety at three- and four-leg intersections. On further review, the expert panel decided that the results of the Kulmala study alone did not provide a sufficient basis for challenging the widely accepted view that any intersection skew that departs from a 90-degree angle, whether positive or negative, is detrimental to safety. Therefore, equations (27) and (29) have been recast as shown below. For a three-leg STOP-controlled intersection:

$$AMF = \exp\ (0.0040\ SKEW) \quad \textbf{(30)}$$

For a four-leg STOP-controlled intersection:

$$AMF = \exp\ (0.0054\ SKEW) \quad \textbf{(31)}$$

where:

SKEW = intersection skew angle (degrees), expressed as the absolute value of the difference between 90 degrees and the actual intersection angle.

These AMFs apply to total intersection accidents.

Signalized Intersections

Skew angle is a much less important factor in the operation of signalized intersection than in the operation of STOP-controlled intersections. Since the traffic signal separates most movements from conflicting approaches, the risk of collisions related to the skew angle between the intersecting approaches is limited at a signalized intersection. Therefore, the AMF for skew angle at four-leg signalized intersections is 1.00 for all cases.

Intersection Traffic Control

The safety differences between STOP-controlled and signalized intersections are accounted for by use of separate base models rather than by an AMF. However, an AMF for the difference between minor-leg and all-way STOP-controlled intersections has been developed and is discussed below. The nominal base case for STOP-controlled intersections has STOP signs on the minor leg(s) only. An AMF is provided for intersection with all-way STOP control. Minor-road YIELD controlled intersections are treated identically to minor-road STOP-controlled intersections in the accident prediction algorithm.

All-way STOP control is most appropriate for lower-speed roadways with relatively equal traffic volumes on all legs of the intersection. The *Manual on Uniform Traffic Control Devices* (MUTCD) includes specific warrants for all-way STOP control.[24] All-way STOP control should not be considered for an intersection unless these warrants are met. The AMF for conversion from minor-road to all-way STOP-control is 0.53. This AMF applies to total intersection-related accidents. The AMF value of 0.53 implies that an all-way STOP-controlled intersection experiences 47 percent fewer accidents than a two-way STOP-controlled intersection. This AMF is based on the findings of Lovell and Hauer.[25] They collected data for three local agencies (San Francisco, Philadelphia, and Toronto) and one State agency (Michigan). Of these, only the Michigan data pertains to low-volume, high-speed, rural roads. However, since the Michigan data set is small and its results are similar to those for the other three agencies, the recommended AMF is based on the combined data for all four agencies. The expert panel recommended that some other IHSDM module, possibly the diagnostic review module for design of at-grade intersections, should make clear to IHSDM users that all-way STOP-control should be used only when the established warrants are met. This is necessary to discourage indiscriminate use of all-way STOP-control, because it is likely that the substantial safety benefits of all-way STOP control shown below can be attained only when the warrants are met.

Intersection Left-Turn Lanes

The nominal or base condition for intersection left-turn lanes is the absence of left-turn lanes on the major-road approaches. The AMFs for presence of left-turn lanes on the major road are presented in table 5. These AMFs apply to total intersection-related accidents. The expert panel did not find any well-designed before-and-after studies on the accident reduction effectiveness of left-turn lanes. Therefore, the AMFs in the table represent a judgment by the expert panel combining results from several sources. The AMFs for installation of left-turn lanes at STOP-controlled intersections are based on studies by Agent, Bauer and Harwood, California Department of Public Works, Creasy and Agent, Dale, Ermer, Glennon, McCoy et al., McCoy and Malone, and Smith et al. The AMFs for installation of left-turn lanes at signalized intersections are based on studies by Agent, California Department of Public Works, Datta, Smith et al., and McCoy and Malone.[26, 1, 27, 29, 30, 31, 32, 33, 34, 35, 36, 37] The AMFs for installation of left-turn lanes on both approaches to a four-leg intersection are equal to the square of the corresponding AMF for installation of a left-turn lane on a single approach. No data are available to quantify the effect on safety of left-turn lanes on a minor road, so these will not be considered in the accident prediction algorithm.

Table 5. Accident Modification Factors for Installation of Left-turn Lanes on the Major-Road Approaches to Intersection on Two-Lane Rural Highways.

Intersection type	Intersection traffic control	Number of major-road approaches on which left-turn lanes are installed	
		One approach	Both approaches
Three-leg intersection	STOP sign[a]	0.78	!
	Traffic signal	0.85	!
Four-leg intersection	STOP sign[a]	0.76	0.58
	Traffic signal	0.82	0.67

[a] STOP signs on minor-road approach(es).

Intersection Right-Turn Lanes

The nominal or base condition for intersection right-turn lanes is the absence of right-turn lanes on the major-road approaches. The AMF for the presence of right-turn lanes at STOP-controlled intersections is 0.95 for a right-turn lane on one major-road approach and 0.90 for right-turn lanes on both major road approaches. These AMFs apply to total intersection-related accidents. The expert panel did not find any well-designed before-and-after studies on the accident reduction effectiveness of right-turn lanes. Therefore, the values of the AMFs are based on a judgment by the panel based on the work of Vogt and Bared, Kulmala, and Elvik.[3, 4, 5, 23, 38] No data are available to quantify the effect on safety of right-turn lanes on a minor road, so these are not considered in the accident prediction algorithm. Also, no effect is considered for the provision of a paved shoulder on an intersection approach unless that paved shoulder is marked as a right-turn lane.

No studies that the expert panel considered appropriate were found concerning the accident reduction effectiveness of right-turn lanes at signalized intersections. Therefore, the panel made a judgment that the effectiveness of right-turn lanes at signalized intersections should be estimated as half that found at STOP-controlled intersections. Thus, the AMF for the presence of right-turn lanes at signalized intersections is 0.975 for a right-turn lane on one major-road approach and 0.95 for right-turn lanes on both major-road approaches. These AMFs also apply to total intersection-related accidents.

Intersection Sight Distance

The nominal or base condition for intersection sight distance is the availability of adequate intersection sight distance along the major road in all quadrants of the intersection. The AMFs for intersection sight distance at intersections with STOP control on the minor leg(s) are:

- 1.05 if sight distance is limited in one quadrant of the intersection.
- 1.10 if sight distance is limited in two quadrants of the intersection.
- 1.15 if sight distance is limited in three quadrants of the intersection.
- 1.20 if sight distance is limited in four quadrants of the intersection.

These AMFs apply to total intersection-related accidents.

Sight distance in a quadrant is considered limited if the available sight distance is less than the sight distance specified by AASHTO policy for a design speed of 20 km/h less than the major-road design speed. Only sight distance restrictions due to roadway alignment and terrain are considered by the accident prediction algorithm. Sight distance restrictions due to specific obstructions (e.g., trees, bushes, poles, and buildings) are not available in the CAD system and, therefore, are not to be considered by the accident prediction algorithm.

The AMFs for intersection sight distance apply only to two-way STOP-controlled or YIELD-controlled intersections. An AMF of 1.00 is applicable to signal-controlled and all-way STOP-controlled intersections.

There was no single evaluation of the effects of intersection sight distance on accidents that the panel found to be most credible. Therefore, the recommended AMF was determined from the panel's best judgment based on the results of Kulmala, Brüde and Larsson, and Elvik.[39, 38] These results were assumed to represent sight distance improvements in all quadrants of an intersection. Therefore, this effect was proportioned on a per-quadrant basis for application in the accident prediction algorithm.

5. SENSITIVITY ANALYSIS RESULTS

A sensitivity analysis was performed to illustrate the estimated effect on safety of the various parameters and factors in the accident prediction algorithm. The results of this sensitivity analysis are present in this section.

All of the sensitivity analyses presented here were performed with the calibration factors for both roadway segments (C_r) and intersections (C_i) set equal to 1.00. Thus, these results do not represent the safety conditions experienced by any particular highway agency and should be interpreted primarily in a relative sense.

Roadway Segments

The sensitivity analysis for roadway segments first established the variation of accident frequency and accident rate with ADT for the base or nominal condition. Then specific AMFs in the prediction algorithm for roadway segments were varied one at a time. Finally, the predicted accident frequencies and accident rates for combinations of AMFs with extremely high and extremely low accident experience were determined.

Nominal or Base Condition

The nominal or base condition for evaluating roadway segments consists of the following combination of conditions:

- Lanes that are 3.6-m (12-ft).
- Paved shoulders that are 1.8-m (6-ft).
- Tangent roadway.
- Level grade (0 percent).
- Three driveways per km (5 driveways per mi).
- Roadside hazard rating = 3.
- No passing lanes or short four-lane sections.

Table 6 illustrates the variation of accident frequency and accident rate with the roadway segment ADT for the nominal or base condition. The table shows that for the nominal or base condition the accident frequency per mile per year increases linearly with increasing ADT, while the accident rate per million veh-mi remains constant.

Table 6 and the other tables in this section of the report are presented in conventional units because all of the equations and AMFs on which they are based are in conventional units (see sections 3 and 4).

Table 6. Sensitivity of Safety to ADT for Nominal Conditions for Roadway Segments.

ADT (veh/day)	Accidents per mi per year	Accidents per million veh-mi
400	0.09	0.61
1,000	0.22	0.61
3,000	0.67	0.61
5,000	1.12	0.61
10,000	2.24	0.61

Conversion: 1 mi = 1.61 km

Lane Width

Table 7 presents the sensitivity of safety to lane width while all other factors are held at their nominal or base conditions. The table shows that under low-volume conditions there is very limited sensitivity of safety to lane width, while the sensitivity is larger at higher volume levels. For ADTs above 2,000 veh/day, accident frequency is 16.5 percent higher for 2.7 m (9 ft) lanes than for 3.6 m (12 ft) lanes.

Table 7. Sensitivity of Safety to Lane Width on Roadway Segments.

ADT (veh/day)	Lane Width (ft)			
	9	10	11	12 BASE
	ACCIDENTS PER MILE PER YEAR			
400	0.09	0.09	0.09	0.09
1,000	0.24	0.23	0.23	0.22
3,000	0.79	0.74	0.68	0.67
5,000	1.32	1.24	1.14	1.12
10,000	2.64	2.48	2.28	2.24
	ACCIDENTS PER MILLION VEHICLE-MILES			
400	0.63	0.62	0.62	0.61
1,000	0.66	0.64	0.62	0.61
3,000	0.72	0.68	0.63	0.61
5,000	0.72	0.68	0.63	0.61
10,000	0.72	0.68	0.63	0.61

Conversion: 1 mi = 1.61 km; 1 ft = 0.305 m

Shoulder Type and Width

Table 8 presents the sensitivity of safety to shoulder type and width while all other factors are held at their nominal or base condition. Like the lane width effect, there is very limited sensitivity of safety to shoulder type and width at low volume levels. For ADTs above 2,000 veh/day, accident frequency can differ by a maximum of 25 percent among various combinations of shoulder type and width.

Table 8. Sensitivity of Safety to Shoulder Type and Width on Roadway Segments.

ADT (veh/day)	None 0 (BASE)	Paved 2	Paved 4	Paved 6	Paved 8	Gravel 2	Gravel 4	Gravel 6	Gravel 8	Turf 2	Turf 4	Turf 6	Turf 8
	ACCIDENTS PER MILE PER YEAR												
400	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
1,000	0.24	0.24	0.23	0.22	0.22	0.24	0.23	0.23	0.22	0.24	0.23	0.23	0.23
3,000	0.79	0.74	0.71	0.67	0.64	0.75	0.71	0.68	0.65	0.75	0.72	0.69	0.67
5,000	1.32	1.24	1.18	1.12	1.07	1.24	1.18	1.13	1.08	1.25	1.20	1.15	1.11
10,000	2.64	2.48	2.36	2.24	2.14	2.49	2.37	2.26	2.16	2.51	2.40	2.31	2.22
	ACCIDENTS PER MILLION VEHICLE-MILES												
400	0.64	0.63	0.62	0.61	0.61	0.63	0.62	0.62	0.61	0.64	0.63	0.63	0.64
1,000	0.67	0.65	0.63	0.61	0.60	0.65	0.63	0.62	0.61	0.66	0.64	0.63	0.63
3,000	0.72	0.68	0.65	0.61	0.59	0.68	0.65	0.62	0.59	0.69	0.66	0.63	0.61
5,000	0.72	0.68	0.65	0.61	0.59	0.68	0.65	0.62	0.59	0.69	0.66	0.63	0.61
10,000	0.72	0.68	0.65	0.61	0.59	0.68	0.65	0.62	0.59	0.69	0.66	0.63	0.61

Shoulder Type and Width (ft)

Conversion: 1 ft = 0.305 m; 1 mi = 1.61 km

Horizontal Curvature

Tables 9 and 10 present the sensitivity of safety to factors related to horizontal curvature. Table 9 compares the safety performance of a tangent roadway with various combinations of horizontal curve length and radius with and without spiral transitions.

The values in the table are computed with all factors other than horizontal curvature set to their nominal or base conditions. The table shows that the safety performance of long flat curves is only slightly worse than the safety performance of a tangent roadway. However, short sharp curves can have much higher accident rates. A horizontal curve with a length of 31 m (100 ft) and a radius of 31 m (100 ft) on a roadway segment would be expected to have an accident rate over 28 times as high as a tangent section on the same roadway. Addition of spiral transition curves can reduce accident frequencies up to a maximum of 6.6 percent for the curves selected for table 9.

Table 10 shows the expected effect on safety of superelevation deficiencies for horizontal curves. the table shows, as indicated in equation (21) that a superelevation deficiency of 0.02

increases accidents on the curve by 6 percent and a deficiency of 0.04 increases accidents by 12 percent.

Table 9. Sensitivity of Safety to Horizontal Curve Length and Radius on Roadway Segments.

ADT (veh/day)	Tangent	Curve Length = 100 ft Radius (ft)			Curve Length = 500 ft Radius (ft)			Curve Length = 1,000 ft Radius (ft)			Curve Length = 2,000 ft Radius (ft)		
		100	200	500	500	1,000	2,000	1,000	2,000	5,000	1,000	2,000	5,000
ACCIDENTS PER MILE PER YEAR—Curves without spiral transitions													
400	0.09	2.55	1.32	0.58	0.19	0.14	0.11	0.11	0.10	0.09	0.10	0.10	0.09
1,000	0.22	6.37	3.30	1.45	0.47	0.35	0.29	0.29	0.26	0.24	0.26	0.24	0.23
3,000	0.67	19.11	9.89	4.36	1.41	1.04	0.86	0.86	0.77	0.71	0.77	0.72	0.69
5,000	1.12	31.84	16.48	7.27	2.35	1.74	1.43	1.43	1.28	1.18	1.28	1.20	1.15
10,000	2.24	63.69	32.97	14.53	4.70	3.47	2.86	2.86	2.55	2.37	2.55	2.40	2.31
ACCIDENTS PER MILLION VEHICLE-MILES—Curves without spiral transitions													
400	0.61	17.45	9.03	3.98	1.29	0.95	0.78	0.78	0.70	0.65	0.70	0.66	0.63
1,000	0.61	17.45	9.03	3.98	1.29	0.95	0.78	0.78	0.70	0.65	0.70	0.66	0.63
3,000	0.61	17.45	9.03	3.98	1.29	0.95	0.78	0.78	0.70	0.65	0.70	0.66	0.63
5,000	0.61	17.45	9.03	3.98	1.29	0.95	0.78	0.78	0.70	0.65	0.70	0.66	0.63
10,000	0.61	17.45	9.03	3.98	1.29	0.95	0.78	0.78	0.70	0.65	0.70	0.66	0.63
ACCIDENTS PER MILE PER YEAR—Curves with spiral transitions													
400	0.09	2.51	1.26	0.54	0.18	0.13	0.11	0.11	0.10	0.09	0.10	0.09	0.09
1,000	0.22	6.28	3.20	1.36	0.45	0.33	0.27	0.28	0.25	0.23	0.25	0.24	0.23
3,000	0.67	18.83	9.61	4.08	1.35	0.99	0.80	0.83	0.74	0.68	0.75	0.71	0.68
5,000	1.12	31.28	16.02	6.81	2.26	1.64	1.34	1.38	1.23	1.14	1.25	1.18	1.13
10,000	2.24	62.77	32.05	13.61	4.51	3.29	2.67	2.77	2.46	2.27	2.50	2.35	2.26
ACCIDENTS PER MILLION VEHICLE-MILES—Curves with spiral transitions													
400	0.61	17.20	8.78	3.73	1.24	0.90	0.73	0.76	0.67	0.62	0.69	0.64	0.62
1,000	0.61	17.20	8.78	3.73	1.24	0.90	0.73	0.76	0.67	0.62	0.69	0.64	0.62
3,000	0.61	17.20	8.78	3.73	1.24	0.90	0.73	0.76	0.67	0.62	0.69	0.64	0.62
5,000	0.61	17.20	8.78	3.73	1.24	0.90	0.73	0.76	0.67	0.62	0.69	0.64	0.62
10,000	0.61	17.20	8.78	3.73	1.24	0.90	0.73	0.76	0.67	0.62	0.69	0.64	0.62

Conversion: 1 ft = 0.305 m; 1 mi = 1.61 km

Table 10. Sensitivity of Safety to Horizontal Curve Superelevation Deficiency on Roadway Segments.

ADT (veh/day)	Curve Length = 100 ft; Curve Radius = 200 ft; Superelevation Deficiency			Curve Length = 500 ft; Curve Radius = 1,000 ft; Superelevation Deficiency			Curve Length = 1,000 ft; Curve Radius = 2,000 ft; Superelevation Deficiency			Curve Length = 2,000 ft; Curve Radius = 2,000 ft; Superelevation Deficiency		
	0.00	0.02	0.04	0.00	0.02	0.04	0	0.02	0.04	0	0.02	0.04
	ACCIDENTS PER MILE PER YEAR											
400	1.32	1.40	1.47	0.14	0.15	0.16	0.10	0.11	0.11	0.10	0.10	0.11
1,000	3.29	3.49	3.68	0.35	0.37	0.39	0.26	0.27	0.29	0.24	0.25	0.27
3,000	9.87	10.46	11.05	1.04	1.10	1.17	0.77	0.81	0.86	0.72	0.76	0.81
5,000	16.45	17.44	18.42	1.74	1.84	1.94	1.28	1.35	1.43	1.20	1.27	1.34
10,000	32.90	34.88	36.85	3.47	3.68	3.89	2.55	2.70	2.86	2.40	2.54	2.68
	ACCIDENTS PER MILLION VEHICLE-MILES											
400	9.01	9.55	10.10	0.95	1.01	1.06	0.70	0.74	0.78	0.66	0.70	0.74
1,000	9.01	9.55	10.10	0.95	1.01	1.06	0.70	0.74	0.78	0.66	0.70	0.74
3,000	9.01	9.55	10.10	0.95	1.01	1.06	0.70	0.74	0.78	0.66	0.70	0.74
5,000	9.01	9.55	10.10	0.95	1.01	1.06	0.70	0.74	0.78	0.66	0.70	0.74
10,000	9.01	9.55	10.10	0.95	1.01	1.06	0.70	0.74	0.78	0.66	0.70	0.74

Conversion: 1 ft = 0.305 m; 1 mi = 1.61 km

Grade

Table 11 illustrates the sensitivity of safety to roadway grades. The table shows that, as also indicated in table 4, steeper grades increase accidents by 1.6 percent per 1-percent increase in grade.

Table 11. Sensitivity of Safety to Percent Grade on Roadway Segments.

ADT (veh/day)	Percent Grade				
	0 BASE	2	4	6	8
	ACCIDENTS PER MILE PER YEAR				
400	0.09	0.09	0.10	0.10	0.10
1,000	0.22	0.23	0.21	0.25	0.25
3,000	0.67	0.69	0.72	0.74	0.76
5,000	1.12	1.16	1.20	1.23	1.27
10,000	2.24	2.32	2.39	2.47	2.55
	ACCIDENTS PER MILLION VEHICLE-MILES				
400	0.61	0.63	0.66	0.68	0.70
1,000	0.61	0.63	0.66	0.68	0.70
3,000	0.61	0.63	0.66	0.68	0.70
5,000	0.61	0.63	0.66	0.68	0.70
10,000	0.61	0.63	0.66	0.68	0.70

Conversion: 1 mi = 1.61 km

Driveway Density

Table 12 presents the sensitivity of safety to driveway density for roadway segments while all other factors remain at their nominal or base conditions. The table shows that a roadway segment with 19 driveways per km (30 driveways per mi) can experience up to four times as many accidents as a similar roadway segment with no driveways. The sensitivity of safety to driveway density is greater at lower ADTs than at higher ADTs, although the absolute magnitudes of the predicted accident frequencies at low ADT are very low. Nevertheless, it might be more reasonable to expect greater sensitivity of accidents to driveways at higher ADTs than at lower ADTs. Further research on this issue would be desirable.

Table 12 also shows the predicted accident frequency and accident rate for two-lane highway sections with two-way left-turn lanes (TWLTLs). The AMF for TWLTLs is based on equations (23) and (24). The accident reduction effectiveness of a TWLTL ranges from 2 to 23 percent as a function of driveway density.

Table 12. Sensitivity of Safety to Driveway Density on Roadway Segments.

ADT (veh/day)	Driveway Density (driveways per mi)						
	0	5 BASE	10	15	20	25	30
	ACCIDENTS PER MILE PER YEAR						
400	0.06	0.09	0.12	0.15	0.18	0.21	0.24
1,000	0.16	0.22	0.29	0.35	0.41	0.47	0.54
3,000	0.54	0.67	0.81	0.94	1.08	1.21	1.34
5,000	0.95	1.12	1.30	1.47	1.65	1.82	2.00
10,000	2.04	2.24	2.45	2.65	2.85	3.05	3.25
	ACCIDENTS PER MILLION VEHICLE-MILES						
400	0.41	0.61	0.82	1.03	1.23	1.44	1.64
1,000	0.44	0.61	0.79	0.96	1.13	1.30	1.47
3,000	0.49	0.61	0.74	0.86	0.98	1.11	1.23
5,000	0.52	0.61	0.71	0.81	0.90	1.00	1.10
10,000	0.56	0.61	0.67	0.73	0.78	0.84	0.89
	ACCIDENTS PER MILE PER YEAR—WITH TWLTL[a]						
400	0.06	0.09	0.11	0.13	0.15	0.17	0.18
1,000	0.16	0.22	0.27	0.31	0.34	0.38	0.41
3,000	0.54	0.66	0.75	0.83	0.90	0.97	1.04
5,000	0.95	1.10	1.21	1.30	1.38	1.46	1.54
10,000	2.04	2.19	2.28	2.33	2.38	2.44	2.50
	ACCIDENTS PER MILLION VEHICLE—MILES[a]						
400	0.41	0.60	0.76	0.90	1.03	1.15	1.26
1,000	0.44	0.60	0.73	0.84	0.94	1.04	1.13
3,000	0.49	0.60	0.69	0.76	0.82	0.88	0.95
5,000	0.52	0.60	0.66	0.71	0.76	0.80	0.84
10,000	0.56	0.60	0.62	0.64	0.65	0.67	0.69

Conversion: 1 mi = 1.61 km

Passing Lanes

Table 13 presents the sensitivity of safety to passing lanes and short four-lane sections on roadway segments. The table shows that, as explained in section 4 of this report, installation of passing lanes to increase passing opportunities reduces accidents by 25 percent and installation of short four-lane sections to increase passing opportunities reduces accidents by 35 percent.

Table 13. Sensitivity of Safety to Presence of Passing Lanes and Short Four-Lane Sections on Roadway Segments.

ADT (veh/day)	Passing Lane Present ? No BASE	Passing Lane Present ? Yes	Short Four-Lane Section Present ? No BASE	Short Four-Lane Section Present ? Yes
	ACCIDENTS PER MILE PER YEAR			
400	0.09	0.07	0.09	0.06
1,000	0.22	0.17	0.22	0.15
3,000	0.67	0.50	0.67	0.44
5,000	1.12	0.84	1.12	0.73
10,000	2.24	1.68	2.24	1.46
	ACCIDENTS PER MILLION VEHICLE-MILES			
400	0.61	0.46	0.61	0.40
1,000	0.61	0.46	0.61	0.40
3,000	0.61	0.46	0.61	0.40
5,000	0.61	0.46	0.61	0.40
10,000	0.61	0.46	0.61	0.40

Conversion: 1 mi = 1.61 km

Roadside Design

Table 14 presents the sensitivity of safety to roadside hazard rating on roadway segments while all other factors are held at their nominal or base conditions. The table shows that roadside hazard rating can increase total accident frequency by up to 50 percent over the full range of roadside hazard ratings.

Table 14. Sensitivity of Safety to Roadside Hazard Rating on Roadway Segments.

ADT (veh/day)	Roadside hazard rating 1	2	3 BASE	4	5	6	7
	ACCIDENTS PER MILE PER YEAR						
400	0.08	0.08	0.09	0.10	0.10	0.11	0.12
1,000	0.20	0.21	0.22	0.24	0.26	0.27	0.29
3,000	0.59	0.63	0.67	0.72	0.77	0.82	0.88
5,000	0.98	1.05	1.12	1.20	1.26	1.37	1.47
10,000	1.96	2.10	2.24	2.40	2.56	2.74	2.93
	ACCIDENTS PER MILLION VEHICLE-MILES						
400	0.54	0.58	0.61	0.66	0.70	0.75	0.80
1,000	0.54	0.58	0.61	0.66	0.70	0.75	0.80
3,000	0.54	0.58	0.61	0.66	0.70	0.75	0.80
5,000	0.54	0.58	0.61	0.66	0.70	0.75	0.80
10,000	0.54	0.58	0.61	0.66	0.70	0.75	0.80

Conversion: 1 mi = 1.61 km

Combinations of Geometric Design and Traffic Control Features

Table 15 presents the sensitivity of safety to extreme combinations of geometric design and traffic control features. The low accident frequency combination represents the "best" combination of features considered in the previous sensitivity analyses. Specifically, this low accident frequency combination includes:

- Lanes that are 3.6-m (12-ft).
- Paved shoulders that are 2.4-m (8-ft).
- Tangent roadway.
- Level grade (0 percent).
- No driveways.
- Roadside hazard rating = 1.
- Short four-lane sections used to increase passing opportunities.

The accident frequencies and rates shown in the table represent levels that are unlikely to be improved further through geometric design or traffic control modifications.

By contrast, the high accident frequency combination represents the "worst" combination of features considered in the previous sensitivity analyses. Specifically, the high accident frequency combination includes:

- Lanes that are 2.7-m (9-ft).
- No shoulders.
- Horizontal curve with length of 31 m (100 ft), radius of 31 m (100 ft), no spiral transition curve, and a superelevation deficiency of 0.04.
- An 8-percent grade.
- Nineteen driveways per km (30 driveways per mi).
- Roadside hazard rating = 7.
- No passing lanes or short four-lane sections.

The accident frequencies and rates shown in the table are extremely high, but the combination of geometric and traffic control features they represent is so extreme that it is unlikely to exist in the real world.

Table 15. Sensitivity of Safety to Extreme Combinations of Geometric Design and Traffic Control Features.

ADT (veh/day)	Low-Accident Frequency Combination[a]	High-Accident Frequency Combination[a]
	ACCIDENTS PER MILE PER YEAR	
400	0.03	11.87
1,000	0.09	29.59
3,000	0.29	87.35
5,000	0.51	129.87
10,000	1.11	211.25
	ACCIDENTS PER MILLION VEHICLE-MILES	
400	0.23	81.31
1,000	0.25	81.07
3,000	0.27	79.77
5,000	0.28	71.16
10,000	0.30	57.88

Conversion: 1 mi = 1.61 km

[a] These combinations of geometric design and traffic control features are defined in the accompanying text.

Three-Leg STOP-Controlled Intersections

A sensitivity analysis was performed with the accident prediction algorithm for three-leg STOP-controlled intersections. The nominal or base condition for this analysis consisted of the following geometric design conditions:

- No major-road left- or right-turn lanes.
- No skew angle (90-degree intersection angles).
- No intersection sight distance deficiencies.

The accident frequencies per year for this condition for various combinations of major- and minor-road ADT are shown in table 16. Table 16 also shows the predicted accident frequencies for various combinations of major-road left- and right-turn lanes. As indicated in table 5 installation of a major-road left-turn lane at a three-leg STOP-controlled intersection is expected to reduce accident frequency by 22 percent. Installation of a major-road right-turn lane is expected to reduce accident frequency by 5 percent.

Table 17 presents the sensitivity of safety to skew-angle for three-leg STOP-controlled intersections. A skew angle of 10 degrees results in an accident frequency 4 percent higher than a 90-degree intersection, while a skew angle of 45 degrees results in an accident frequency 20 percent higher than a 90-degree intersection.

Table 18 presents the sensitivity of safety to of intersection sight distance limitations at three-let STOP-controlled intersections. As indicated in section 4 of this report, intersection sight distance limitations can increase accident frequency by 5 percent per quadrant.

Table 16. Sensitivity of Safety to Major-Road Turn Lanes at Three-Leg STOP-Controlled Intersections.

Major-road ADT (veh/day)	Minor-road ADT (veh/day)	No TLs BASE	One LTL	One RTL	One LTL & one RTL
	ACCIDENTS PER YEAR				
400	50	0.01	0.01	0.01	0.01
	100	0.02	0.02	0.02	0.01
	400	0.04	0.03	0.04	0.03
1,000	100	0.04	0.03	0.04	0.03
	500	0.09	0.07	0.09	0.07
	1,000	0.13	0.10	0.12	0.09
3,000	100	0.10	0.08	0.09	0.07
	500	0.22	0.17	0.21	0.16
	1,000	0.30	0.24	0.29	0.23
	3,000	0.52	0.41	0.50	0.39
5,000	100	0.15	0.11	0.14	0.11
	500	0.32	0.25	0.31	0.24
	1,000	0.46	0.36	0.43	0.34
	3,000	0.78	0.61	0.74	0.58
	5,000	1.00	0.78	0.95	0.74
10,000	100	0.25	0.20	0.24	0.19
	500	0.56	0.44	0.53	0.42
	1,000	0.79	0.61	0.75	0.58
	3,000	1.35	1.05	1.28	1.00
	5,000	1.73	1.35	1.65	1.28
	10,000	2.43	1.90	2.31	1.80

Note: TL=turn lane; LTL=left-turn lane; RTL=right-turn lane.

Table 17. Sensitivity of Safety to Skew Angles at Three-Leg STOP-Controlled Intersections.

Major-road ADT (veh/day)	Minor-road ADT (veh/day)	Skew angle (degrees) 0 BASE	10	15	30	45
		ACCIDENTS PER YEAR				
400	50	0.01	0.01	0.01	0.01	0.01
	100	0.02	0.02	0.02	0.02	0.02
	400	0.04	0.04	0.04	0.05	0.05
1,000	100	0.04	0.04	0.04	0.05	0.05
	500	0.09	0.09	0.10	0.10	0.11
	1,000	0.13	0.14	0.14	0.15	0.16
3,000	100	0.10	0.10	0.11	0.11	0.12
	500	0.22	0.23	0.23	0.25	0.26
	1,000	0.30	0.31	0.32	0.34	0.36
	3,000	0.52	0.54	0.55	0.59	0.62
5,000	100	0.15	0.16	0.16	0.17	0.18
	500	0.32	0.34	0.34	0.37	0.39
	1,000	0.46	0.47	0.48	0.51	0.55
	3,000	0.78	0.81	0.83	0.88	0.94
	5,000	1.00	1.04	1.06	1.13	1.20
10,000	100	0.25	0.26	0.27	0.28	0.30
	500	0.56	0.58	0.59	0.63	0.67
	1,000	0.79	0.82	0.84	0.89	0.94
	3,000	1.35	1.40	1.43	1.52	1.61
	5,000	1.73	1.80	1.84	1.95	2.07
	10,000	2.43	2.53	2.58	2.74	2.91

Table 18. Sensitivity of Safety to Limited Intersection Sight Distance at Three-Leg STOP-Controlled Intersections.

Major-road ADT (veh/day)	Minor-road ADT (veh/day)	Number of quadrants with limited ISD: 0 BASE	1	2
		ACCIDENTS PER YEAR		
400	50	0.01	0.02	0.02
	100	0.02	0.02	0.02
	400	0.04	0.04	0.04
1,000	100	0.04	0.04	0.05
	500	0.09	0.10	0.10
	1,000	0.13	0.13	0.14
3,000	100	0.10	0.10	0.11
	500	0.22	0.23	0.24
	1,000	0.30	0.32	0.33
	3,000	0.52	0.55	0.57
5,000	100	0.15	0.15	0.16
	500	0.32	0.34	0.36
	1,000	0.46	0.48	0.50
	3,000	0.78	0.82	0.86
	5,000	1.00	1.05	1.10
10,000	100	0.25	0.27	0.28
	500	0.56	0.59	0.62
	1,000	0.79	0.83	0.87
	3,000	1.35	1.42	1.48
	5,000	1.73	1.82	1.91
	10,000	2.43	2.55	2.68

Note: ISD = intersection sight distance.

Four-Leg STOP-Controlled Intersections

A sensitivity analysis was performed with the accident prediction algorithm for four-leg STOP-controlled intersections. The nominal or base condition for this analysis consisted of the following geometric design conditions:

- No major-road left- or right-turn lanes.
- No skew-angle (90-degree intersection angle).
- No intersection sight distance deficiencies.

The accident frequencies per year for this condition for various combinations of major- and minor-road ADT are shown in table 19. Table 19 also shows the predicted accident frequencies for various combinations of major-road left- and right turn lanes. As indicated in Table 5, a single major-road left-turn lane at a four-leg STOP-controlled intersection is expected to reduce accident frequency by 24 percent and two major-road left turn lanes are expected to reduce accident frequency by 42 percent. A single major-road right-turn lane would reduce accident frequency by 5 percent and two major-road right-turn lanes would reduce accident frequency by 10 percent.

Table 20 presents the sensitivity of safety to intersection skew angle for four-leg STOP-controlled intersections. A skew angle of 10 degrees results in an accident frequency 6 percent higher than a 90-degree intersection, while a skew angle of 45 degrees results in an accident frequency 28 percent higher than a 90-degree intersection.

Table 21 presents the sensitivity of safety to deficiencies of intersection sight distance at four-leg STOP-controlled intersections. As indicated in section 4 of this report, intersection sight distance deficiencies can increase accident frequency by 5 percent per quadrant.

Four-Leg Signalized Intersections

A sensitivity was performed with the accident prediction algorithm for four-leg signalized intersections. The nominal or base condition for this analysis consisted of a four-leg signalized intersection with no major-road left- or right-turn lanes. The accident frequencies per year for this condition for various combinations of major- and minor-road ADT are shown in table 22. Table 22 also shows predicted accident frequencies for various combinations of major-road left- and right-turn lanes. As indicated in table 5, a single major-road left-turn lane at a four-leg signalized intersection is expected to reduce accident frequency by 18 percent and two major-road left-turn lanes are expected to reduce accident frequency by 33 percent. By contrast, a single major-road right-turn lane would reduce accident frequency by 2.5 percent and two major-road right-turn lanes would reduce accident frequency by 5 percent.

The predicted accident frequency at a four-leg signalized intersection is not sensitive to intersection skew angle or intersection sight distance limitations.

Table 19. Sensitivity of Safety to Major-Road Turn Lanes at Four-Leg STOP-Controlled Intersections.

Major-Road ADT (veh/day)	Minor-Road ADT (veh/day)	No TLs BASE	One LTL	Two LTLs	One RTL	Two RTLs	One LTL & One RTL	One LTL & Two RTLs	One RTL & Two LTLs	Two LTLs & Two RTLs
		ACCIDENTS PER YEAR								
400	50	0.03	0.02	0.02	0.03	0.03	0.02	0.02	0.02	0.02
	100	0.05	0.04	0.03	0.05	0.05	0.04	0.03	0.03	0.03
	400	0.12	0.09	0.07	0.12	0.11	0.09	0.08	0.07	0.06
1,000	100	0.09	0.07	0.05	0.09	0.08	0.06	0.06	0.05	0.05
	500	0.25	0.19	0.15	0.23	0.22	0.18	0.17	0.14	0.13
	1,000	0.37	0.28	0.21	0.36	0.34	0.27	0.25	0.20	0.19
3,000	100	0.18	0.14	0.10	0.17	0.16	0.13	0.12	0.10	0.09
	500	0.47	0.36	0.27	0.45	0.43	0.34	0.32	0.26	0.25
	1,000	0.72	0.55	0.42	0.69	0.65	0.52	0.49	0.40	0.38
	3,000	1.42	1.08	0.82	1.35	1.27	1.03	0.97	0.78	0.74
5,000	100	0.24	0.18	0.14	0.23	0.22	0.17	0.16	0.13	0.13
	500	0.64	0.46	0.37	0.61	0.58	0.46	0.44	0.35	0.33
	1,000	0.98	0.74	0.57	0.94	0.89	0.71	0.67	0.54	0.51
	3,000	1.92	1.46	1.11	1.83	1.73	1.39	1.31	1.06	1.00
	5,000	2.63	2.00	1.53	2.50	2.36	1.90	1.80	1.45	1.37
10,000	100	0.37	0.28	0.21	0.35	0.33	0.27	0.25	0.20	0.19
	500	0.98	0.74	0.57	0.93	0.88	0.71	0.67	0.54	0.51
	1,000	1.49	1.13	0.86	1.42	1.34	1.08	1.02	0.82	0.78
	3,000	2.92	2.22	1.69	2.77	2.62	2.11	2.00	1.61	1.52
	5,000	3.98	3.02	2.31	3.78	3.58	2.87	2.72	2.19	2.08
	10,000	6.08	4.63	3.53	5.77	5.47	4.39	4.16	3.35	3.17

Note: TL=turn lane; LTL=left-turn lane; RTL=right-turn lane.

Table 20. Sensitivity of Safety to Skew Angle at Four-Leg STOP-Controlled Intersections.

Major-Road ADT (veh/day)	Minor-Road ADT (veh/day)	Skew Angle (degrees) 0 BASE	10	15	30	45
		ACCIDENTS PER YEAR				
400	50	0.03	0.04	0.04	0.04	0.04
	100	0.05	0.06	0.06	0.06	0.07
	400	0.12	0.13	0.13	0.15	0.16
1,000	100	0.09	0.10	0.10	0.11	0.12
	500	0.25	0.26	0.27	0.29	0.31
	1,000	0.37	0.40	0.41	0.44	0.48
3,000	100	0.18	0.19	0.19	0.21	0.23
	500	0.47	0.50	0.51	0.56	0.61
	1,000	0.72	0.76	0.74	0.85	0.92
	3,000	1.42	1.49	1.54	1.66	1.81
5,000	100	0.24	0.25	0.26	0.28	0.31
	500	0.64	0.68	0.70	0.76	0.82
	1,000	0.98	1.04	1.07	1.16	1.25
	3,000	1.92	2.03	2.09	2.26	2.45
	5,000	2.63	2.71	2.85	3.09	3.35
10,000	100	0.37	0.39	0.40	0.43	0.47
	500	0.98	1.03	1.06	1.15	1.25
	1,000	1.49	1.57	1.62	1.75	1.90
	3,000	2.92	3.08	3.16	3.43	3.72
	5,000	3.98	4.20	4.32	4.68	5.08
	10,000	6.08	6.41	6.59	7.15	7.75

Table 21. Sensitivity of Safety to Limited Intersection Sight Distance Deficiencies at Four-Leg STOP-Controlled Intersections.

Major-Road ADT (veh/day)	Minor-Road ADT (veh/day)	Number of quadrants with limited ISD 0 BASE	1	2	3	4
		ACCIDENTS PER YEAR				
400	50	0.03	0.04	0.04	0.04	0.04
	100	0.05	0.06	0.06	0.06	0.06
	400	0.12	0.13	0.14	0.14	0.15
1,000	100	0.09	0.10	0.10	0.11	0.11
	500	0.25	0.26	0.27	0.28	0.29
	1,000	0.37	0.39	0.41	0.43	0.45
3,000	100	0.18	0.19	0.20	0.20	0.21
	500	0.47	0.50	0.52	0.55	0.57
	1,000	0.72	0.76	0.80	0.83	0.87
	3,000	1.42	1.49	1.56	1.63	1.70
5,000	100	0.24	0.25	0.27	0.28	0.29
	500	0.64	0.68	0.71	0.74	0.77
	1,000	0.98	1.03	1.08	1.13	1.18
	3,000	1.92	2.02	2.12	2.21	2.31
	5,000	2.63	2.76	2.89	3.02	3.15
10,000	100	0.37	0.38	0.40	0.42	0.44
	500	0.98	1.03	1.08	1.12	1.17
	1,000	1.49	1.57	1.64	1.72	1.79
	3,000	2.92	3.06	3.21	3.35	3.50
	5,000	3.98	4.18	4.38	4.58	4.78
	10,000	6.08	6.38	6.68	6.99	7.29

Table 22. Sensitivity of Safety to Major-Road Turn Lanes at Four-Leg Signalized Intersections.

Major-Road ADT (veh/day)	Minor-Road ADT (veh/day)	No TLs BASE	One LTL	Two LTLs	One RTL	Two RTLs	One LTL & One RTL	One LTL & Two RTLs	One RTL & Two LTLs	Two LTLs & Two RTLs
		ACCIDENTS PER YEAR								
400	50	0.26	0.21	0.17	0.25	0.25	0.21	0.20	0.17	0.16
	100	0.30	0.24	0.20	0.29	0.28	0.24	0.23	0.19	0.19
	400	0.39	0.32	0.26	0.38	0.37	0.31	0.31	0.26	0.25
1,000	100	0.51	0.42	0.34	0.50	0.49	0.41	0.40	0.34	0.33
	500	0.71	0.58	0.48	0.69	0.67	0.57	0.55	0.46	0.45
	1,000	0.82	0.67	0.55	0.80	0.77	0.65	0.64	0.53	0.52
3,000	100	0.99	0.82	0.67	0.97	0.95	0.80	0.78	0.65	0.63
	500	1.37	1.13	0.92	1.34	1.30	1.10	1.07	0.90	0.87
	1,000	1.58	1.29	1.06	1.54	1.50	1.26	1.23	1.03	1.00
	3,000	1.96	1.61	1.32	1.92	1.87	1.57	1.53	1.28	1.25
5,000	100	1.35	1.11	0.91	1.32	1.28	1.08	1.05	0.88	0.86
	500	1.87	1.53	1.25	1.82	1.77	1.49	1.45	1.22	1.19
	1,000	2.14	1.76	1.44	2.09	2.04	1.71	1.67	1.40	1.36
	3,000	2.67	2.19	1.79	2.60	2.54	2.13	2.08	1.74	1.70
	5,000	2.96	2.42	1.98	2.88	2.81	2.36	2.30	1.93	1.88
10,000	100	2.05	1.68	1.37	2.00	1.95	1.64	1.60	1.34	1.30
	500	2.83	2.32	1.89	2.76	2.69	2.26	2.20	1.85	1.80
	1,000	3.25	2.66	2.18	3.17	3.08	2.60	2.53	2.12	2.07
	3,000	4.05	3.32	2.71	3.94	3.84	3.23	3.15	2.64	2.57
	5,000	4.48	3.67	3.00	4.37	4.26	3.58	3.49	2.93	2.85
	10,000	5.15	4.22	3.45	5.02	4.89	4.11	4.01	3.36	3.28

Note: TL=turn lane; LTL=left-turn lane; RTL=right-turn lane.

6. IMPLEMENTATION OF THE ACCIDENT PREDICTION ALGORITHM WITHIN THE IHSDM

The accident prediction algorithm is intended to help the user to make unbiased estimates of the expected safety performance for any given geometric design alternative for a specific highway improvement project. Complete evaluation of one or more proposed geometric design alternatives for a particular project will require the user to determine, for comparative purposes, both the safety performance of the current design and the expected future safety performance of that current design if nothing is done to change the roadway (the "do nothing" or "baseline" alternative).

This section of the report describes the implementation of the accident prediction algorithm within the IHSDM. Procedures are presented by which the algorithm can be used to make unbiased estimates of the:

- Expected safety performance of one or more geometric design alternatives for a planned roadway that has not yet been constructed.
- Recent or current safety performance of an existing roadway considering both the predicted safety performance of the roadway and its observed accident history.
- Expected safety performance of the existing roadway in the future if the geometrics are left unchanged (the ADT may change, of course).
- Expected safety performance of one or more proposed geometric design alternatives for improving the existing roadway.

This section describes two methods for producing these estimates—one without and one with consideration of site-specific accident history data for the project of interest. The first procedure described is used when no site-specific accident history data are available. This first procedure is applicable to planned roadways that have not yet been constructed and to existing roadways where, for whatever reason, site-specific accident history data are not available to the analyst. The second procedure described is used when site-specific accident history are available. This procedure incorporates an Empirical Bayes (EB) approach to combining estimates from the accident prediction algorithm and site-specific accident history data. Examples of the EB procedure are presented later in this section.

Site-specific accident history data make an important contribution to increasing the accuracy of predictions of the expected safety performance of highway facilities. Therefore, the analyst should seek to obtain and use site-specific accident history data and apply the EB procedure whenever possible.

The procedures presented here can be applied to any existing two-lane highway or to any two-lane highway improvement project that retains the basic two-lane character of the facility.

Two-lane highway improvements evaluated with these procedures can include addition of a third or fourth lane over a short distance to improve passing opportunities on the highway. Thus, the procedures can evaluate passing lanes that create a three-lane cross section and short four-lane sections that are operationally equivalent to side-by-side passing lanes. Such added lanes do not normally exceed 3.2 km (2 mi) in length. The procedures do not address widening of a two-lane highway to a four-lane cross section for an extended length. It is hoped that appropriate procedures for four-lane highways will be developed in the future so that analysis of two-lane to four-lane widening projects will be possible within the IHSDM.

Accident Prediction When Site-Specific Accident History Data are not Available

The accident prediction algorithm is intended to estimate the expected accident frequency for any specified geometric design alternative and for any specified evaluation period. The specified geometric design alternative to which the algorithm is applied can be either the existing roadway (i.e., the "do-nothing" alternative), a proposed geometric design improvement to the existing roadway, or a proposed roadway that has not yet been constructed. This first procedure is applicable only to a project for which no site-specific accident history data are available. The algorithm can be used to compare the expected safety performance of several geometric alternatives by applying the algorithm separately to each alternative for the same evaluation period and comparing the results. When no site-specific accident history data are available, the duration of the evaluation period may be one year or any multiple of one year. The accident prediction algorithm is applied in IHSDM to any specific geometric design alternative in a series of straight forward steps, as follows:

- Step 1—Define the limits of the project and determine the geometrics of the project for which the expected safety performance is to be predicted.

- Step 2—Divide the project into individual homogeneous roadway segments and intersections.

- Step 3—Determine the geometric design and traffic control features for each individual roadway segment and intersection.

- Step 4—Determine the ADTs for each roadway segment and intersection during each year for which the expected safety performance is to be predicted.

- Step 5—Select an individual roadway segment or intersection for evaluation. If there are no more roadway segments or intersections to be evaluated, go to step 13.

- Step 6—Select a particular year of the specified evaluation period for the roadway segment or intersection of interest. If there are no more years to be evaluated for that roadway segment or intersection, go to step 12.

- Step 7—Apply the appropriate base model to determine the predicted accident frequency for nominal or base conditions for the selected year.

- Step 8—Multiply the result obtained in step 7 by the appropriate calibration factor for a specific State or geographical region.

- Step 9—Multiply the result obtained in step 8 by the appropriate AMFs representing safety differences between the nominal or base conditions and the actual geometrics and traffic control of the roadway segment or intersection.

- Step 10—Estimate the expected distribution accident severities and accident types for the roadway segment or intersection from the default distributions of accident severity and accident type.

- Step 11—If there is another year to be evaluated for the selected roadway segment or intersection, return to step 6. Otherwise, proceed to step 12.

- Step 12—If there is another roadway segment or intersection to be evaluated, return to step 5. Otherwise, proceed to step 13.

- Step 13—Summarize and present the predictions in useful formats for the IHSDM user.

Figure 5 presents a flow diagram of the accident prediction algorithm incorporating these steps. Each of these steps is described below:

Step 1—Define the limits of the project and determine the geometrics of the project for which the expected safety performance is to be predicted.

The project evaluated can represent either an existing roadway or a design alternative for a proposed improvement project. The geometric design features of the project and the traffic control at each intersection must be documented. The geometric design features are determined from either a plan of the existing roadway available in the CAD system or from data entered by the user. If CAD data are to be used, a program must be developed to interrogate the CAD file, determine the geometrics of the project, and store those geometric data in a format that can be used by IHSDM.

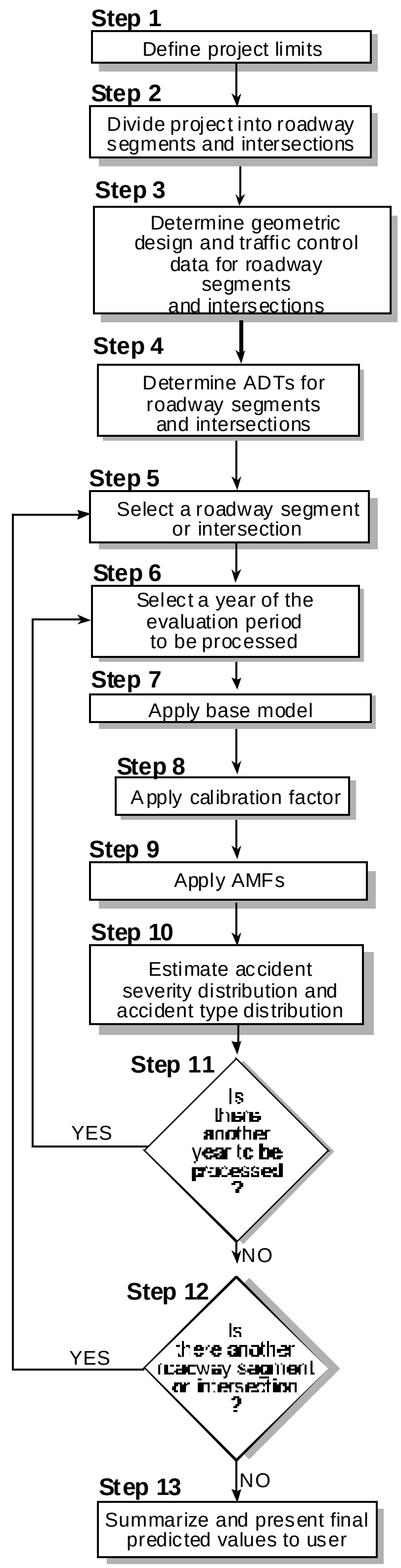

Figure 5. Flow Diagram of the Accident Prediction Algorithm When No Site-Specific Accident History Data Are Available.

Step 2—Divide the project into individual homogeneous roadway segments and intersections.

The next step is to divide the project into individual homogeneous roadway segments and intersections. The roadway must be divided into homogeneous segments. A new homogeneous segment begins at each intersection where the value of one of the following variables changes:

- Average daily traffic volume (veh/day).
- Lane width.
- Shoulder width.
- Shoulder type.
- Driveway density (driveways per mile).
- Roadside hazard rating.

Also, a new analysis section starts at any of the following locations:

- Intersection.
- Beginning or end of a horizontal curve.
- Point of vertical intersection (PVI) for a crest vertical curve, a sag vertical curve, or an angle point at which two different roadway grades meet.
- Beginning or end of a passing lane or short four-lane section provided for the purpose of increasing passing opportunities.
- Beginning or end of a center two-way left-turn lane.

Step 3—Determine the geometric design and traffic control features for each individual roadway segment and intersection.

For each roadway segment, the following geometric and traffic control features must be quantified:

- Length of segment (mi).
- ADT (veh/day).
- Lane width (ft).
- Shoulder width (ft).
- Shoulder type (paved/gravel/composite/turf).
- Presence or absence of horizontal curve (curve/tangent).
- Length of horizontal curve (mi), if the segment is located on a curve. [This represents the total length of the horizontal curve, even if the curve extends beyond the limits of the roadway segment being analyzed.]
- Radius of horizontal curve (ft), if the segment is located on a curve.
- Presence or absence of spiral transition curve, if the segment is located on a curve. [This represents the presence or absence of a spiral transition curve at the beginning and end of

the horizontal curve, even if the beginning and/or end of the horizontal curve are beyond the limits of the segment being analyzed.]

- Superelevation of horizontal curve, if the segment is located on a horizontal curve.
- Grade (percent), considering each grade as a straight grade from PVI to PVI (i.e., ignoring the presence of vertical curves).
- Driveway density (driveways per mi).
- Presence or absence of a passing lane to increase passing opportunities.
- Presence or absence of a short four-lane Section to increase passing opportunities.
- Presence or absence of a two-way left-turn lane.
- Roadside hazard rating.

For each intersection, the following geometric and traffic control features must be quantified:

- Number of intersection legs (3 or 4).
- Type of traffic control (minor-road STOP, all-way STOP, minor-road YIELD control, or signal).
- Intersection skew angle (degrees departure from 90 degrees, with a + or - sign indicating the direction of the departure).
- Number of major-road approaches with intersection left-turn lanes (0, 1, or 2).
- Number of major-road approaches with intersection right-turn lanes (0, 1, or 2).
- Number of intersection quadrants with deficient intersection sight distance (0, 1, 2, 3, or 4).

The values of these geometric and traffic control parameters for roadway segments and intersections will be determined from the CAD system, from existing data files, or from data supplied by the user.

Step 4—Determine the ADTs for each roadway segment and intersection during each year for which the expected safety performance is to be predicted.

For each roadway segment and for the major- and minor-road approaches to each intersection, ADT data are needed for each year of the period to be evaluated. Ideally, these ADT data will already be available in a file or they will be entered by the user. If ADTs are available for every roadway segment, the major-road ADTs for intersection approaches can be determined without additional data being supplied by the user. If the ADTs on the two major-road legs of an intersection differ, the average of the two ADT values should be used for the intersection. For a three-leg intersection, the user should enter the ADT of the minor-road leg. For a four-leg intersection, the user should enter the average of the ADTs for the two minor-road legs.

In many cases, it is expected that ADT data will not be available for all years of the evaluation period. In that case, the analyst should interpolate or extrapolate as appropriate to obtain an estimate of ADT for each year of the evaluation period. If the analyst does not do this, the following default rules are applied within the accident prediction algorithm to estimate the ADTs for years for which the data are not available. If ADT data are available for only a single year, that same value is assumed to apply to all years of the before period. If two or more years of ADT data are available, the ADTs for intervening years are computed by interpolation. ADTs for years before the first year for which data are available are assumed to be equal to the ADT for that first year; ADTs for years after the last year for which data are available are assumed to be equal to the last year (i.e., no extrapolation is used by the algorithm).

Step 5—Select an individual roadway segment or intersection for evaluation. If there are no more roadway segments or intersections to be evaluated, go to step 13.

Roadway segments and intersections are evaluated one at a time. Steps 6 through 11, described below, are repeated for each roadway segment and intersection.

Step 6—Select a particular year of the specified evaluation period for the roadway segment or intersection of interest. If there are no more years to be evaluated for that roadway segment or intersection, go to step 12.

The individual years of the evaluation period are evaluated one year at a time for any particular roadway segment or intersection. Separate estimates are made for each year because several of the AMFs considered in step 9 are dependent on the ADT of the roadway segment or intersection, which may change from year to year. Steps 7 through 10, described below, are repeated for each year of the evaluation period as part of the evaluation of any particular roadway segment or intersection.

Step 7—Apply the appropriate base model to determine the predicted accident frequency for nominal or base conditions for the selected year.

The predicted accident frequency for nominal or base conditions is determined with one of the following base models:

- Roadway segments . . . . Equation (6).
- Three-leg STOP-controlled intersections . . . . Equation (8).
- Four-leg STOP-controlled intersections . . . . Equation (10).
- Four-leg signalized intersections . . . . Equation (12).

The ADT(s) used in the base model should be the ADT(s) for the selected year of the evaluation period.

Step 8—Multiply the result obtained in step 7 by the appropriate calibration factor.

The calibration factors used in step 8 are the calibration factor for roadway segments (C_r) and the calibration factor for intersections (C_i) discussed in section 3 and appendix C of this report.

Step 9—Multiply the result obtained in step 8 by the appropriate AMFs representing safety differences between the nominal or base conditions and the actual geometrics and traffic control of the roadway segment or intersection.

The AMFs for roadway segments and intersections are those described in Section 4 of this report. Steps 8 and 9 together implement equations (13) and (14).

Step 10—Estimate the expected distribution of accident severities and accident types for the roadway segment or intersection of interest from the default distributions of accident severity and accident type.

The predictions of accident frequencies are supplemented by breaking down those frequencies by accident severity and by accident type. This can provide the IHSDM user with greater insight about safety conditions within the project. The accident severity and accident type estimates are based on the default distributions of accident severity and accident type presented in Tables 1 and 2, respectively. These default distributions may be changed by IHSDM users as part of the calibration process.

Step 11—If there is another year to be evaluated for the selected roadway segment or intersection, return to step 6. Otherwise, proceed to step 12.

This step creates a loop through steps 7 to 10 that is repeated for each year of the evaluation period for each of the individual roadway segments and intersections within the project.

Step 12—If there is another roadway segment or intersection to be evaluated, return to step 5. Otherwise, proceed to step 13.

This step creates a loop through Steps 6 to 11 that is repeated for each roadway segment and intersection within the project.

Step 13—Summarize and present the predictions in useful formats for the IHSDM user.

In the final step, the predicted accident frequencies are summarized and presented in useful formats for IHSDM users. The data presented include, at a minimum:

- Accident frequencies for the project as a whole including:
    - Total accident frequency.
    - Accident frequency by severity level.
    - Accident frequency by accident type.

- Accident frequencies for individual roadway segments and intersections, expressed as accident rates per mi per year or accident rate per million veh-mi for roadway segments and accident rates per million entering vehicles for intersections, so that accident "hot spots" that might be corrected through design improvements are evident.

Estimated accident frequencies could also be broken down by individual years of the evaluation period. However, this is not normally done because the combined estimates across all years of the evaluation period are generally of greatest interest to safety analysts. Predicted accident frequencies for a multiyear period are likely to be more accurate than predicted accident frequencies for any particular year.

Accident Prediction When Site-Specific Accident History Data are Available

Consideration of site-specific accident history data in the accident prediction algorithm increases the accuracy of the predicted accident frequencies. When at least 2 years of site-specific accident history data are available for the project being evaluated, and when the project meets certain criteria discussed below, the accident history data should be used. When considering site-specific accident history data, the algorithm must consider both the existing geometric design and traffic control for the project (i.e., the conditions that existed during the *before* period while the accident history was accumulated) and the proposed geometric design and traffic control for the project (i.e., the conditions that will exist during the *after* period, the future period for which accident predictions are being made). The EB procedure discussed below provides a method to combine predictions from the algorithm with site-specific accident history data.

Situations in Which the EB Procedure Should and Should Not Be Applied

The applicability of the EB procedure depends on the availability of observed accident history data and the type of improvement project being evaluated. If no observed accident history data are available, application of the EB procedure is infeasible and should not be considered. If observed accident history data are available, the applicability of the EB procedure depends on the type of improvement project being evaluated.

The EB procedure should be applied for the following improvement types whenever observed accident history data are available:

- Sites at which the roadway geometrics and traffic control are not being changed (e.g., the "do-nothing" alternative).

- Projects in which the roadway cross Section is modified but the basic number of lanes remains the same. This would include, for example, projects for which lanes or shoulders were widened or the roadside was improved, but the roadway remained a rural two-lane highway.

- Projects in which minor changes in alignment are made, such as flattening individual horizontal curves while leaving most of the alignment intact.

- Projects in which a passing lane or a short four-lane section is added to a rural two-lane highway to increase passing opportunities.

- Any combination of the above improvements.

The EB procedure is not applicable to the following types of improvements:

- Projects in which a new alignment is developed for at least 50 percent of the project length. In this case, the procedure used when no site-specific accident history data are available, as described above, should be applied because there is no reason why the accident history of the old alignment should be used as a predictor of future accident frequency on the new alignment. In others words, there is no reason to think that the new roadway will have substantially higher (or lower), accident experience, simply because the existing roadway has high (or low) accident experience. For cases in which the user is concerned that a particular geographic area or corridor has higher or lower accident experience than expected, a special study may be performed to revise the calibration factor accordingly.

- Individual intersections at which the basic number of intersection legs or type of traffic control is changed as part of a project. The EB procedure can be applied to the rest of any project containing such an intersection, but the intersection itself should be omitted.

The reason that the EB procedure is not used for these project types is that the observed accident data for a past time period is not necessarily indicative of the accident experience that is likely to occur in the future, after such a major geometric improvement. When the EB procedure does not apply, the accident prediction algorithm without the EB procedure (described earlier in this section of the report) is used to determine the expected safety performance for a project during some future time period. When the EB procedure does apply, the accident prediction algorithm, including the EB procedure, is applied as described below.

Empirical Bayes Procedure

The EB procedure provides a methodology to combine the accident frequencies predicted by the accident prediction algorithm (N_p) with the accident frequency from the site-specific accident history data (O). The EB procedure uses a weighted average of N_p and O. This procedure constitutes Step 9 of the step-by-step methodology presented below. A previous application of the EB methodology in before-and-after safety evaluations of intersections converted from STOP to signalized control is presented by Griffith.[40]

The expected accident frequency considering both the predicted and observed accident frequencies is computed as:

$$E_p = w\,(N_p) + (1-w)\,O \qquad \textbf{(32)}$$

where:

E_p = expected accident frequency based on a weighted average of N_p and N_o;
N_p = number of accidents predicted by the accident prediction algorithm during a specified period of time (equal to N_{rs} for a roadway segment or N_{int} for an intersection);
w = weight to be placed on the accident frequency predicted by the accident prediction algorithm; and
O = number of accidents observed during a specified period of time.

The weight placed on the predicted accident frequency is determined in the EB procedure as:

$$w = \frac{1}{1 + k(N_p)} \qquad \textbf{(33)}$$

where:

k = overdispersion parameter of the relevant base model of the accident prediction algorithm.

The formulation of Equation (33) shows an inverse relationship between the magnitude of the accident frequency predicted by the algorithm, N_p and the weight, w. Therefore, as the value of N_p predicted by the algorithm increases, the weight placed on N_p decreases. This relationship implies that the higher the expected accident frequency predicted by the algorithm for a particular location, the more the reliance that should be placed on the observed site-specific accident history and the less the reliance that should be placed on the model prediction itself. By contrast, when the model prediction is smaller, less reliance should be placed on the observed site-specific accident history and greater reliance should be placed on the model prediction.*

Table 23 shows the values of the overdispersion parameters (k) for the four base models used in the accident prediction algorithm.

The EB procedure works best if the roadway segments and intersections to which it is applied contain at least a specified minimum number of predicted accidents. The minimum accident frequency needed for application of the EB procedure is generally 1/k, where k is the overdispersion parameter of the relevant base model. In the accident prediction algorithm, this l/k criterion is normally applied to the accident frequency for fatal and injury accidents because the frequency of fatal and injury accidents is usually less than the frequency of property-damage-only accidents and is always less than or equal to the total accident frequency. Where the fatal and injury accident frequency of particular roadway segments or intersections is less than 1/k, such segments and intersections may be aggregated into larger analysis units for application of the EB procedure.

In the accident prediction algorithm, the EB procedure is applied separately to the total predicted and observed accident frequencies and to the predicted and observed frequencies for two accident severity levels: fatal and injury accidents and property-damage-only accidents. Because the EB procedure is applied separately, the predicted fatal and injury accident and property-damage-only accident frequencies may not sum to the predicted total accident frequencies. A proportional adjustment to the predicted accident frequencies for the individual severity levels is made to correct this discrepancy.

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* Equation (33) follows from the theoretical development of the EB approach by Hauer.(2) Hauer defines the weight in the EB procedure as $(1+\mathrm{Var}\{\hat{e}\}/E\{\hat{e}\})^{-1}$, where $E\{\hat{e}\}$ is the expected mean accident frequency and $\mathrm{Var}\{\hat{e}\}$ is the variance of accident frequency. The expected mean accident frequency is best estimated by the model prediction, N_p. Negative binomial regression modeling is based on the assumption that $\mathrm{Var}\{\hat{e}\}=k(E\{\hat{e}\})^2$. Therefore, it follows that the ratio $\mathrm{Var}\{\hat{e}\}/E\{\hat{e}\}$ can be estimated by k/N_p, which leads to Equation (33).

Table 23. Overdispersion Parameters for Base Models and Minimum Accident Frequencies for EB Procedure.

Geometric element	Overdispersion Parameter for base model (k)	Minimum accident frequency for EB procedure[a] (1/k)
Roadway segment	0.31	3
Three-leg STOP-controlled intersection	0.54	2
Four-leg STOP-controlled intersection	0.24	4
Four-leg signalized intersection	0.11	9

[a] Rounded for application in the EB procedure. Normally, this minimum accident frequency criterion is applied in the EB procedure to the predicted fatal and injury accident frequency.

If an EB analysis unit combines roadway segments and intersections, or more than one type of intersection, two additional factors must be accounted for. First, the overdispersion parameter, k, in the denominator of equation (33) is no longer uniquely defined, because two or more base models with differing overdispersion parameters must be considered. Second, it can no longer be assumed, as is normally done, that the expected numbers of accidents for the roadway segments and intersections, or for the different types of intersections, are statistically correlated with one another. Rather, an estimate of expected accidents should be computed based on the assumption that the different entities are statistically independent (ρ=0) and on the alternative assumption that they are perfectly correlated (ρ=1). The expected accident frequency is then estimated as the average of the estimates for ρ=0 and ρ=1. The following equations implement this approach:

$$w_0 = \frac{1}{1 + \dfrac{k_{rs}N_{rs}^2 + k_{i1}N_{i1}^2 + k_{i2}N_{i2}^2 + k_{i3}N_{i3}^2}{N_{rs} + N_{i1} + N_{i2} + N_{i3}}} \tag{34}$$

$$E_0 = w_0(N_{rs} + N_{i1} + N_{i2} + N_{i3}) + (1 - w_0)(O_{rs} + O_{i1} + O_{i2} + O_{i3}) \tag{35}$$

$$w_1 = \frac{1}{1 + \dfrac{\sqrt{k_{rs}}N_{rs} + \sqrt{k_{i1}}N_{i1} + \sqrt{k_{i2}}N_{i2} + \sqrt{k_{i3}}N_{i3}}{N_{rs} + N_{i1} + N_{i2} + N_{i3}}} \tag{36}$$

$$E_1 = w_1(N_{rs}+N_{i1}+N_{i2}+N_{i3}) + (1-w_1)(O_{rs}+O_{i1}+O_{i2}+O_{i3}) \quad \textbf{(37)}$$

$$E_{au} = \frac{E_0 + E_1}{2} \quad \textbf{(38)}$$

where:

- w_0 = weight placed on predicted accident frequency when accident frequencies for different roadway elements are statistically independent (ρ=0);
- w_1 = weight placed on predicted accident frequency when accident frequencies for different roadway elements are perfectly correlated (ρ=1);
- E_0 = expected accident frequency based on the assumption that different roadway elements are statistically independent (ρ=0);
- E_1 = expected accident frequency based on the assumption that different roadway elements are perfectly correlated (ρ=1);
- E_{au} = expected accident frequency for an analysis unit made up of two or more roadway segments or intersections;
- k_{rs} = overdispersion parameter for roadway segments (analogously, a subscript of i1 represents three-leg STOP-controlled intersections, i2 represents four-leg STOP-controlled intersections, and i3 represents four-leg signalized intersections);
- N_{rs} = predicted total number of accidents for all roadway segments within the EB analysis unit (analogous for subscripts i1, i2, and i3);
- O_{rs} = observed total number of accidents for all roadway segments within the EB analysis unit (analogous for subscripts i1, i2, and i3).

Example Application of the EB Procedure

The following discussion presents a numerical example to illustrate the application of the EB procedure. The example shows the application of the EB procedure to combine predicted accident frequencies for the period from 1989 through 1997 for roadway segments and intersections within a project site with the observed site-specific accident history for that same time period. The combined estimates of accident frequency resulting from application of the EB procedure represent an unbiased estimate of safety conditions during the period prior to construction of a proposed project at this site (the “before period”) and can subsequently be used in estimating the effect on safety of proposed alternative geometric design improvements to this site (the “after” period). The example illustrates the EB procedure, but does not illustrate the entire accident prediction methodology that incorporates the EB procedure; that methodology is presented later in this section of the report.

The example addresses two hypothetical roadway segments and one hypothetical intersection. Roadway segment 1 is a two-lane highway segment 1.6 km (1 mi) in length with an initial ADT of 2,000 veh/day. Over the 9-year period (1989-1997), the ADT varies up and down reaching 2,200 veh/day in 1997. Roadway segment 2 is a longer 8-km (5-mi) two-lane highway segment with a lower, but faster growing, ADT than segment 1. Intersection 1 is a four-leg STOP-controlled intersection located on segment 1. The intersection has a minor-road volume that increases from 500 veh/day in 1989 to 600 veh/day in 1997.

The first portion of the example shows the application of the EB procedure to roadway segment 1. This roadway segment was predicted to experience 4.2 accidents during the 9-year period (1989-1997), but actually experienced 6 accidents. Table 24 shows the application of the EB procedure which determines that the expected accident frequency for the roadway segment, considering both the predicted and observed values, is 5.2 accidents. Of these, it is expected that 3.3 would be fatal and injury accidents and 1.9 would be property-damage-only accidents.

While the computations in table 24 follow the EB procedure, the table shows that the predicted fatal and injury accident frequency for roadway segment 1 before application of the EB procedure was 1.4 accidents during the 9-year period. This does not meet the minimum accident frequency criterion (l/k) of three fatal and injury accidents for roadway segments shown in table 23. Thus, rather than relying on the results shown in table 24, it would be better to combine roadway segment 1 with another roadway segment before applying the EB procedure.

Table 25 presents the application of the EB procedure to roadway segment 2 in a manner analogous to table 24. Roadway segment 2 was predicted to experience 10.3 accidents during the 9-year period, but actually experienced 14 accidents. The table shows that the expected accident frequency for roadway segment 2, considering both the predicted and observed values, is 13.1 accidents during the 9-year period.

Roadway segment 2 was originally predicted to experience 3.3 fatal and injury accidents per year, so the minimum accident criterion for application of the EB procedure shown in table 23 is met. However, because the criterion was not met for roadway segment 1, it would be desirable to combine roadway segments 1 and 2 into a single EB analysis unit. Table 26 shows that if roadway segments 1 and 2 were combined into a single analysis unit for application of the EB procedure, table 26 shows that the expected accident frequency for the combined roadway segment during the 9-year period would be 19 accidents, including 9.8 fatal and injury accidents and 9.2 property-damage-only accidents. Simple proportions based on the original predicted accident frequencies show

Table 24. Application of Empirical Bayes Procedure to Roadway Segment 1.

Year	89	90	91	92	93	94	95	96	97				
Calibration Factor (C_r)	1.01	1.01	1.01	0.98	0.98	0.98	1.05	1.05	1.05				
AADT (veh/day)	2,000	1,800	1,800	1,900	2,000	2,100	2,200	2,300	2,200				
ACCIDENT FREQUENCY (1989 - 1997)										Sum (1989-1997)	Weights[c]	Expected Before Period Accident Frequency[d]	Expected Before Period Accident Frequency (Corrected)
Predicted Total[a]	0.461	0.415	0.415	0.425	0.447	0.469	0.527	0.551	0.527	4.234	0.432	5.236	5.236
Predicted F+ I[b]	0.148	0.133	0.133	0.136	0.143	0.151	0.169	0.177	0.169	1.359	0.704	2.735	3.366[e]
Predicted PDO[b]	0.313	0.281	0.281	0.288	0.303	0.319	0.358	0.374	0.358	2.875	0.529	1.520	1.871
Observed Total	1	0	1	1	0	0	1	1	1	6			
Observed F + I	1	0	1	1	0	0	1	1	1	6			
Observed PDO	0	0	0	0	0	0	0	0	0	0			

NOTE: F + I = Fatal and injury accidents; PDO = property-damage-only accidents.

[a] from equation (13).

[b] based on 32.1 percent fatal and injury accidents and 67.9 percent property-damage-only accidents for roadway segments from table 1.

[c] from equation (33) with k = 0.31 for roadway segments from table 23.

[d] from equation (32).

[e] from equation (40).

[f] from equation (41).

Table 25. Application of Empirical Bayes Procedure to Roadway Segment 2.

Year	89	90	91	92	93	94	95	96	97				Expected
Calibration Factor (C_r)	1.01	1.01	1.01	0.98	0.98	0.98	1.05	1.05	1.05			Expected	Before Period
AADT (veh/day)	700	600	600	650	900	1,000	1,100	1,200	1,250	Sum		Before Period Accident	Accident Frequency
ACCIDENT FREQUENCY (1989 - 1997)										(1989-1997)	Weights [c]	Frequency[d]	(Corrected)
Predicted Total[a]	0.891	0.764	0.764	0.803	1.111	1.235	1.455	1.588	1.654	10.263	0.239	13.106	13.106
Predicted F+ I[b]	0.286	0.245	0.245	0.258	0.357	0.396	0.467	0.510	0.531	3.295	0.495	4.662	4.953[e]
Predicted PDO[b]	0.605	0.518	0.518	0.545	0.755	0.838	0.988	1.078	1.123	6.969	0.316	7.674	8.153[f]
Observed Total	3	1	0	1	2	1	4	1	1	14			
Observed F + I	1	1	0	0	1	1	1	1	0	6			
Observed PDO	2	0	0	1	1	0	3	0	1	8			

NOTE: F + I = Fatal and injury accidents; PDO = property-damage-only accidents.

[a] from equation (13).

[b] based on 32.1 percent fatal and injury accidents and 67.9 percent property-damage-only accidents for roadway segments from table 1.

[c] from equation (33) with k = 0.31 for roadway segments from table 23.

[d] from equation (32).

[e] from equation (40).

[f] from equation (41).

Table 26. Application of Empirical Bayes Procedure to Roadway Segments 1 and 2 Combined.

Year	89	90	91	92	93	94	95	96	97	Sum (1989-1997)	Weights[c]	Expected Before Period Accident Frequency[d]	Expected Before Period Accident Frequency (Corrected)
Calibration Factor (C_r)	1.01	1.01	1.01	0.98	0.98	0.98	1.05	1.05	1.05				
ACCIDENT FREQUENCY (1989 - 1997)													
Predicted Total (Seg 1)[a]	0.461	0.415	0.415	0.425	0.447	0.469	0.527	0.551	0.527				
Predicted Total (Seg 2)[a]	0.891	0.764	0.764	0.803	1.111	1.235	1.455	1.588	1.654				
Predicted Total (Combined)	1.351	1.178	1.178	1.227	1.558	1.704	1.982	2.138	2.180	14.498	0.182	18.998	18.998
Predicted F+ I (Combined)[b]	0.434	0.378	0.378	0.394	0.500	0.547	0.636	0.686	0.700	4.654	0.409	8.992	9.792[e]
Predicted PDO (Combined)[b]	0.918	0.800	0.800	0.833	1.058	1.157	1.346	1.452	1.481	9.844	0.247	8.455	9.207[f]
Observed Total (Combined)	4	1	1	2	2	1	5	2	2	20			
Observed F + I (Combined)	2	1	1	1	1	1	2	2	1	12			
Observed PDO (Combined)	2	0	0	1	1	0	3	0	1	8			

NOTE: F + I = Fatal and injury accidents; PDO = property-damage-only accidents.

[a] from equation (13).

[b] based on 32.1 percent fatal and injury accidents and 67.9 percent property-damage-only accidents for roadway segments from table 1.

[c] from equation (33) with k = 0.31 for roadway segments from table 23.

[d] from equation (32).

[e] from equation (40).

[f] from equation (41).

that roadway segment 1 would be expected to experience 5.5 accidents during the 9-year period (including 2.9 fatal and injury accidents and 2.6 property-damage-only accidents) and roadway segment 2 would be expected to experience 13.5 accidents (including 6.9 fatal and injury accidents and 6.6 property-damage-only accidents). The procedure for performing this proportional allocation is described below in step 10 of the methodology incorporating the EB procedure.

Table 27 illustrates the application of the EB procedure to a four-leg STOP-controlled intersection, designated as intersection 1. The intersection was predicted by the accident prediction algorithm to experience 3.9 accidents in 9 years, but actually experienced 3 accidents. The table shows that the expected accident frequency for the 9-year period, combining both predicted and observed values, is 3.4 accidents, including 1.4 fatal and injury accidents and 2.0 property-damage-only accidents. However, table 27 also shows that the original predicted fatal and injury accident frequency for the intersection is 1.2 accidents, which is less than the minimum accident frequency criterion (l/k) shown in table 23 as 4 accidents for a four-leg STOP-controlled intersection. Therefore, it would be desirable to combine intersection 1 into a larger analysis unit with another intersection for application of the EB procedure. Since there are no other intersections available, intersection 1 should be combined into an analysis unit with roadway sections 1 and 2. Combining intersections with roadways sections is less desirable than combining them with other intersections, but is still acceptable.

Table 28 illustrates the application of the EB procedure to the combined data for an entire project consisting of roadway segments 1 and 2 and intersection 1. The table indicates that the expected accident frequency for the entire project for a 9-year period is 21.0 accidents including 9.9 fatal and injury accidents and 11.1 property-damage-only accidents. This estimate uses computation based on equations (32) through (36) because roadway segments and four-leg STOP-controlled intersections have base models with different overdispersion parameters (0.31 and 0.24, respectively). Using proportional allocation back to the original roadway segments and intersections, roadway segment 1 would be expected to experience 4.8 accidents in the 9-year period (including 2.2 fatal and injury accidents and 2.5 property-damage-only accidents), roadway segment 2 would be expected to experience 11.8 accidents (including 5.5 fatal and injury accidents and 6.3 property-damage-only accidents), and intersection 1 would experience 4.4 accidents (including 2.2 fatal and injury accidents and 2.3 accidents). The procedure for performing this proportional allocation is described below in step 10 of the methodology incorporating the EB procedure.

As noted above, this example illustrates the use of the EB procedure in estimating expected accident frequencies for conditions in the period before construction of an improvement at the site in question. The procedure described in step 11 of the methodology presented below illustrates how the results obtained in the example can be used in estimating the expected future ("after" period) accident frequencies for one or more alternative geometric design improvements.

Table 27. Application of Empirical Bayes Procedure to Intersection 1 (Four-Leg STOP-Controlled Intersection).

Year	89	90	91	92	93	94	95	96	97				
Calibration Factor(C_i)	1.03	1.03	1.03	0.96	0.96	0.96	1.04	1.04	1.04				
Major-Road AADT (veh/day)	2,000	1,800	1,800	1,900	2,000	2,100	2,200	2,300	2,200				
Minor-Road AADT (veh/day)	500	550	550	530	550	580	600	620	600				
ACCIDENT FREQUENCY (1989 - 1997)										Sum (1989-1997)	Weights[c]	Expected Before Period Accident Frequency[d]	Expected Before Period Accident Frequency (Corrected)
Predicted Total[a]	0.402	0.400	0.400	0.377	0.398	0.423	0.481	0.504	0.481	3.866	0.519	3.449	3.449
Predicted F+ I[b]	0.129	0.129	0.129	0.121	0.128	0.136	0.154	0.162	0.154	1.241	0.771	1.415	1.431[e]
Predicted PDO[b]	0.273	0.272	0.272	0.256	0.270	0.287	0.327	0.342	0.327	2.625	0.613	1.997	2.019[f]
Observed Total	0	0	1	0	0	1	0	0	1	3			
Observed F + I	0	0	1	0	0	0	0	0	1	2			
Observed PDO	0	0	0	0	0	1	0	0	0	1			

NOTE: F + I = Fatal and injury accidents; PDO = property-damage-only accidents.
[a] from equation (13).
[b] based on 41.7 percent fatal and injury accidents and 58.3 percent property-damage-only accidents for four-leg STOP-controlled intersections from table 1.
[c] from equation (33) with k = 0.24 for four-leg STOP-controlled intersections from table 23.
[d] from equation (32).
[e] from equation (40).
[f] from equation (41).

Table 28. Application of Empirical Bayes Procedure to Roadway Segments 1 and 2 and Intersection 1 Combined.

Year	89	90	91	92	93	94	95	96	97	Sum (1989-1997)	Weight[d] w_0	Expected Before Period Accident Frequency[e] E_0	Weight[f] w_1	Expected Before Period Accident Frequency[g] E_1	Expected Before Period Accident Frequency[h] E_{au}	Expected Before Period Accident Frequency (Corrected)
	ACCIDENT FREQUENCY (1989 - 1997)															
Predicted Total (Seg 1)[a]	0.461	0.415	0.415	0.425	0.447	0.469	0.527	0.551	0.527	4.234						
Predicted Total (Seg 2)[a]	0.891	0.764	0.764	0.803	1.111	1.235	1.455	1.588	1.654	10.263						
Predicted Total (Int 1)[b]	0.402	0.400	0.400	0.377	0.398	0.423	0.481	0.504	0.481	3.866						
Predicted Total (Combined)	1.754	1.578	1.578	1.604	1.956	2.127	2.463	2.642	2.611	18.364	0.211	22.023	0.648	19.995	21.009	21.009
Predicted F+ I (Seg 1)[c]	0.148	0.133	0.133	0.136	0.143	0.151	0.169	0.177	0.169	1.359						
Predicted F+ I (Seg 2)[c]	0.286	0.245	0.245	0.258	0.357	0.396	0.467	0.510	0.531	3.295						
Predicted F + I (Int 1)[c]	0.129	0.129	0.129	0.121	0.128	0.136	0.154	0.162	0.154	1.241						
Predicted F+ I (Combined)	0.563	0.507	0.507	0.515	0.628	0.683	0.791	0.848	0.854	5.895	0.454	10.319	0.648	8.746	9.532	9.940[i]
Predicted PDO (Seg 1)[c]	0.313	0.281	0.281	0.288	0.303	0.319	0.358	0.374	0.358	2.875						
Predicted PDO (Seg 2)[c]	0.605	0.518	0.518	0.545	0.755	0.838	0.988	1.078	1.123	6.969						
Predicted PDOI (Int 1)[c]	0.273	0.272	0.272	0.256	0.270	0.287	0.327	0.342	0.327	2.625						
Predicted PDO (Combined)	1.191	1.072	1.072	1.089	1.328	1.444	1.672	1.794	1.807	12.469	0.282	9.979	0.648	11.249	10.614	11.068[j]
Observed Total (Combined)	4	1	2	2	2	2	5	2	3	23						
Observed F + I (Combined)	2	1	2	1	1	1	2	2	2	14						
Observed PDO (Combined)	2	0	0	1	1	1	3	0	1	9						

NOTE: F + I = Fatal and injury accidents; PDO = property-damage-only accidents.

[a] from equation (13).

[b] from equation (14).

[c] based on 32.1 percent fatal and injury accidents and 67.9 percent property-damage-only accidents for roadway segments and 41.7 percent fatal and injury accidents and 58.3 percent property-damage-only accidents from four-leg STOP-controlled intersections from table 1.

[d] from equation (34).

[e] from equation (35).

[f] from equation (36).

[g] from equation (37).

[h] from equation (38).

[i] from equation (40).

[j] from equation (41).

Step-by-Step Methodology for Applying the Accident Prediction Algorithm Including the EB Procedure

The accident prediction algorithm including the EB procedure, like the algorithm presented that does not include the EB procedure, is intended to estimate the expected accident frequency for any specified geometric design alternative for a two-lane highway project and for any specified evaluation period. The specified geometric design alternative to which the algorithm is applied can be either the existing roadway (i.e., the "do-nothing" or "baseline" alternative) or a proposed geometric design improvement. For any given geometric alternative, the algorithm incorporates consideration of actual reported accident frequencies for the existing roadway for a previous time period designated as the "before" period. The accident data for this same before period can be used in evaluating the expected safety performance of one or more proposed geometric alternatives for the same project. The algorithm is employed separately to each alternative for the same evaluation period, and the results for these various alternatives are then compared. The accident prediction algorithm including the EB procedure is applied to any specified geometric alternative for any specified evaluation period in 12 steps as follows:

- Step 1—Define the limits of the project and determine the geometrics of the project during the previous period for which observed accident history data are available (the before period) and for the future period for which the expected safety performance of the project is to be predicted (the after period).

- Step 2—Divide the project into individual homogeneous roadway segments and intersections

- Step 3—Determine the geometric design and traffic control features for each individual roadway segment and intersection for both the existing and proposed roadway.

- Step 4—Determine the ADTs for each roadway segment and the major- and minor-road ADTs for each intersection during each year of the before period for which observed accident history data are available and for each year of the after period for which the expected safety performance is to be predicted.

- Step 5—Apply the accident prediction algorithm to each of the individual roadway segments and intersections that make up the existing roadway.

- Step 6—Aggregate roadway segments to join together complete horizontal curve and tangent sections.

- Step 7—Determine the observed accident history during the before period for each of the aggregated roadway segments from step 6 and for each intersection. If accident locations are not available in sufficient detail to identify the individual accident history for each aggregated roadway segment and each intersection, aggregate roadway segments and/or intersections further, as necessary, until EB analysis units are obtained for which observed accident frequencies can be determined.

- Step 8—Aggregate roadway segments and/or intersection further, if necessary, into larger EB analysis units to ensure a minimum accident sample size for each analysis unit.

- Step 9—For each EB analysis unit, as joined in step 8, apply the EB procedure by computing the expected accident frequency for the before period as a weighted average of the predicted and observed accident frequencies.

- Step 10—For each EB analysis unit, disaggregate the expected total accident experience during the before period back to the original roadway segments and intersections identified in step 2.

- Step 11—Apply ADT growth factors and/or AMFs for geometric changes to convert the expected accident frequency for the before period to an expected accident frequency for the proposed project during the specified future time period.

- Step 12—Summarize and present the predictions in useful formats for the IHSDM user.

Figure 6 presents a flow chart illustrating the steps in the accident prediction algorithm incorporating consideration of site-specific accident history data. These steps are described below in greater detail.

Step 1—Define the limits of the project and determine the geometrics of the project during the previous period for which observed accident history data are available (the before period) and for the future period for which the expected safety performance of the project is to be predicted (the after period).

The geometric design features of the project and the traffic control at each intersection must be documented both before-and-after the planned improvement. The geometrics of the existing roadway, which were in place during the before period, are determined from either a plan of the existing roadway available in the CAD system or from data entered by the user. If CAD data are to be used, a program must be developed to interrogate the CAD file, determine the geometrics of the project, and store those geometric data in a format that can be used by the IHSDM. The same sort of data must be obtained for the proposed improvement which, if constructed, will be in place during the after period whose safety performance is to be estimated.

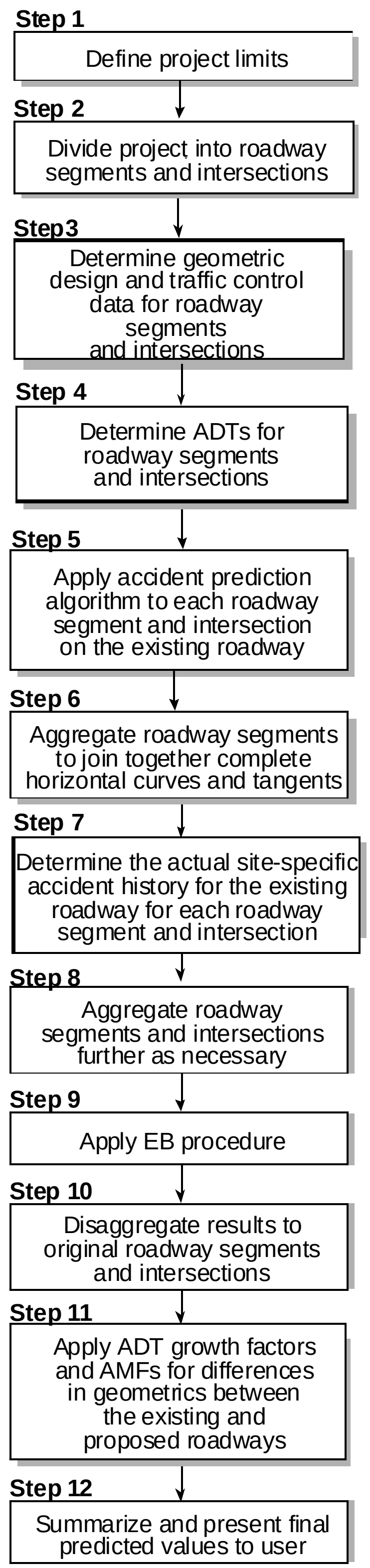

Figure 6. Flow Diagram of the Accident Prediction Algorithm When Site-Specific Accident History Data Are Available.

Step 2—Divide the project into individual homogeneous roadway segments and intersections

The next step is to divide the project into individual homogeneous roadway segments and intersections. The roadway must be divided into homogeneous segments. A new homogeneous segment begins at each intersection where the value of one of the following variables changes:

- Average daily traffic volume (veh/day).
- Lane width.
- Shoulder width.
- Shoulder type.
- Driveway density (driveways per mile).
- Roadside hazard rating.

Also, a new analysis section starts at any of the following locations:

- Intersection.
- Beginning or end of a horizontal curve.
- Point of vertical intersection (PVI) for a crest vertical curve, a sag vertical curve, or an angle point at which two different roadway grades meet.
- Beginning or end of a passing lane or short four-lane section provided for the purpose of increasing passing opportunities.
- Beginning or end of a center two-way left-turn lane.

This segmentation process is applied to both the existing (before period) roadway and the proposed (after period) roadway. Each station that represents a division point for either the existing or proposed roadway must be used as the beginning point of anew segment for purposes of the analysis. In addition, each intersection is treated as a separate entity for analysis.

Step 3—Determine the geometric design and traffic control features for each individual roadway segment and intersection for both the existing and proposed roadway.

For each roadway segment, the following geometric and traffic control features must be quantified for both the existing and proposed roadways:

- Length of segment (mi).
- ADT (veh/day).
- Lane width (ft).
- Shoulder width (ft).
- Shoulder type (paved/gravel/composite/turf).
- Presence or absence of horizontal curve (curve/tangent).

- Length of horizontal curve (mi), if the segment is located on a curve. [This represents the total length of the horizontal curve, even if the curve extends beyond the limits of the roadway segment being analyzed.]
- Radius of horizontal curve (ft), if the segment is located on a curve.
- Presence or absence of spiral transition curve, if the segment is located on a curve. [This represents the presence or absence of a spiral transition curve at the beginning and end of the horizontal curve, even if the beginning and/or end of the horizontal curve are beyond the limits of the segment being analyzed.]
- Superelevation of horizontal curve, if the segment is located on a horizontal curve.
- Grade (percent), considering each grade as a straight grade from PVI to PVI (i.e., ignoring the presence of vertical curves).
- Driveway density (driveways per mi).
- Presence or absence of a passing lane to increase passing opportunities.
- Presence or absence of a short four-lane section to increase passing opportunities.
- Presence or absence of a two-way left-turn lane.
- Roadside hazard rating.

For each intersection, the following geometric and traffic control features must be quantified for both the existing and proposed roadways:

- Number of intersection legs (3 or 4).
- Type of traffic control (minor-road STOP, all-way STOP, minor-road YIELD control, or signal).
- Intersection skew angle (degrees departure from 90 degrees, with a + or - sign indicating the direction of the departure).
- Number of major-road approaches with intersection left-turn lanes (0, 1, or 2).
- Number of major-road approaches with intersection right-turn lanes (0, 1, or 2).
- Number of intersection quadrants with deficient intersection sight distance (0, 1, 2, 3, or 4).

The values of these geometric and traffic control parameters for roadway segments and intersections, for both the existing and proposed designs, will be determined from the CAD system, from existing data files for those designs, or from data supplied by the user.

Step 4—Determine the ADTs for each roadway segment and the major- and minor-road ADTs for each intersection during each year of the before period for which observed accident history data are available and for each year of the after period for which the expected safety performance is to be predicted.

For each roadway segment and for the major- and minor-road approaches to each intersection, ADT data are needed for each year of the before-and-after periods. Ideally, these ADT data will already be available in a file or they will be entered by the user. If ADTs are available for every roadway segment, the major-road ADTs for intersection approaches can be determined without additional data being supplied by the user. If the ADTs on the two major-road legs of an intersection differ, the average of the two ADT values should be used for the intersection. For a three-leg intersection, the user should enter the ADT of the minor-road leg. For a four-leg intersection, the user should enter the average of the ADTs for the two minor-road legs.

In many cases, it is expected that ADT data will not be available for all years of the before or after period. In that case, the analyst should interpolate or extrapolate as appropriate to obtain an estimate of ADT for each year of the evaluation period. If the analyst does not do this, the following default rules are applied within the accident prediction algorithm to estimate the ADTs for years for which the data are not available. If ADT data are available for only a single year of the before period, that same value is assumed to apply to all years of the before period. If two or more years of ADT data for the before period are available, the ADTs for intervening years is computed by interpolation. ADTs for years before the first year for which data are available are assumed to be equal to the ADT for that first year; ADTs for years after the last year for which data are available should be assumed to be equal to the last year (i.e., no extrapolation is used by the algorithm). The same approach should be used to determine ADT data for the after period.

Step 5—Apply the accident prediction algorithm to each of the individual roadway segments and intersections that make up the existing roadway.

The accident prediction algorithm should be applied to each individual roadway segment and intersection that makes up the existing roadway. This step is equivalent to steps 5 through 12 of the algorithm applied when site-specific accident data are not available, as presented earlier in this section of the report. The accident prediction algorithm is applied to roadway segments using equation (13), which incorporates the base model in equation (6), the calibration factor for roadway segments, and the roadway segment AMFs. The accident prediction algorithm for intersections is applied using equation (14), which incorporates the base models in equations (8), (10), or (12), depending on the type of intersection being evaluated, the calibration factor for intersections, and the intersection AMFs. The accident prediction algorithm is applied to each roadway segment or intersection individually for each year of the before period, using the appropriate ADT(s) for that year, and the results for that segment or intersection are summed over all the years to obtain a total predicted accident frequency for the before period. The total predicted accident frequency is allocated to two severity levels (fatal or injury accidents and PDO accidents) based on default proportions from table 1 in the main text of this report.

The predicted accident frequencies obtained from this process are designated N_{rs} for a roadway segment, N_{i1} for a three-leg STOP-controlled intersection, N_{i2} for a four-leg STOP-controlled intersection, and N_{i3} for a four-leg signalized intersection. Each of these predicted frequencies has associated with it predicted frequencies by severity level that sum to the predicted total accident frequency.

Step 6—Aggregate roadway segments to join together complete horizontal curve and tangent sections.

The individual roadway segments are often so short, and have so few predicted and observed accidents, that it would not be meaningful to apply the EB procedure to each individual roadway segment. Therefore, aggregation of short roadway segments into longer segments, which will be called *EB analysis units*, is desirable.

The first stage of this process is to joint together complete horizontal curve and tangent sections of roadway. Proceed in geographical order from one end of the project (e.g., in order of increasing centerline stations) and join roadway segments together leaving breaks between segments only at the following locations:

- Intersections.
- Other points at which the ADT changes by 20 percent or more.
- Beginnings and ends of horizontal curves.
- Beginnings and ends of grades of 5 percent or more that fall with a tangent roadway section.
- Beginnings and ends of passing lanes.
- Beginnings and ends of short, four-lane sections.
- Beginnings and ends of two-way left-turn lanes.

As the segments are aggregated, their predicted accident frequencies (total and for each severity level) should be added together.

Step 6 deals only with the aggregation of roadway segments. For the present, each individual intersection serves as a separate EB analysis unit.

Step 7—Determine the observed accident history during the before period for each of the aggregated roadway segments from step 6 and for each intersection. If accident locations are not available in sufficient detail to identify the individual accident history for each aggregated roadway section and each intersection, aggregate roadway sections and/or intersections further, as necessary, until EB analysis units are obtained for which observed accident frequencies can be determined.

The observed accident history for the before period is obtained from one of two sources: (1) an available accident data file available that can be queried on-line; or (2) key entry of accident data for individual EB analysis units by the IHSDM user. If an accident history file is used, the user will have supply data on the correspondence between the accident location system used in the file (e.g., mileposts) and the stationing system used for the project. This can consist, for example, of the milepost corresponding to the end of the project with the lowest station and an indication of whether mileposts and stations increase in the same direction or in opposite directions. Then, the accident prediction algorithm can calculate the milepost limits of each aggregated roadway segment and the milepost of each intersection and retrieve accident data for each. Intersection-related accidents with 76 m (250 ft) of the intersection should be attributed to that intersection. Non-intersection-related accidents and intersection-related accidents that are located more than 76 m (250 ft) from an intersection should be attributed to the roadway segment within which the accident falls. It should be noted that there are very few intersection-related accidents more than 76 m (250 ft) from an intersection on rural two-lane highways. Intersection-related and non-intersection-related accidents can be distinguished in accident data from most states by the investigating officer's assessment of whether a given accident was intersection related. The available accident file should be used to determine both total observed accidents and accidents by severity level.

If no accident file is available, the accident prediction algorithm will prompt the user to supply total accidents and accidents by severity level for each roadway segment (identified by a range of centerline stations) and each intersection (identified by station and, if possible, by the name of the intersecting minor road). For a large project, this data entry may be tedious, so it will be advantageous to have a accident file whenever possible.

If accident locations are not available in sufficient detail to identify the individual accident history for each aggregated roadway section and each intersection, it will be necessary to aggregate roadway sections and/or intersections further, as necessary, until EB analysis units are obtained for which observed accident frequencies can be determined. The IHSDM user will be asked to identify situations in which further aggregation is necessary because of limitations in the availability or precision of accident locations. Roadway segments and intersections and intersections of different types can be joined together, if necessary, if the accident histories of roadway segments and intersections, or the accident histories of closely-spaced intersections, cannot be distinguished. If the available accident data consist only of totals at the project level, then the entire project can be aggregated into one EB analysis unit, although this should be done only if necessary.

The observed accident frequencies for the before period for a given analysis unit are designated O_{rs} for a roadway segment, O_{i1} for a three-leg STOP-controlled intersection, O_{i2} for a four-leg STOP-controlled intersection, and O_{i3} for a four-leg signalized intersection. Each of these observed frequencies has associated with it observed frequencies by severity level that sum to the observed total accident frequency.

Step 8—Aggregate roadway segments and/or intersection further, if necessary, into larger EB analysis units to ensure a minimum accident sample size for each analysis unit.

It is desirable for an EB analysis unit (i.e., an aggregated roadway segment, an intersection or intersections of a specific type, a combination of roadway segments and intersections, or a combination of intersections of different types) to have at least a minimum number of predicted accidents for the EB procedure to be applied as intended. An individual roadway segment or intersection can constitute an EB analysis unit (i.e., can be evaluated by itself in the EB procedure) if observed accident history data are available for the analysis unit and the accident frequency predicted by the model for the before period is equal to at least 1/k, where k is the overdispersion parameter of the relevant base model. Table 23 shows the desirable minimum numbers of predicted accidents for application of the EB procedure to roadway segments and each type of intersection.

The predicted injury accident frequencies for EB analysis units after any further aggregation in step 7 should be reviewed to determine if there are any analysis units for which the predicted injury accident frequency is less than 1/k, based on table 23. If roadway segments and intersections or intersections of different types are mixed together, use the value of largest appropriate value of 1/k from table 23 for purposes of step 8 (a more sophisticated procedure for dealing with disparate k values will be used in step 9). Any EB analysis unit for which the predicted accident frequency is less than 1/k is a candidate for further aggregation.

Since, an EB analysis unit can combine roadway segments and intersections of different types, its total predicted accident frequency is the sum of the predicted accident frequencies of its constituent segments and intersections:

$$N = N_{rs} + N_{i1} + N_{i2} + N_{i3} \qquad \textbf{(39)}$$

where:

N_{i1} = predicted accident frequency for three-leg STOP-controlled intersections;
N_{i2} = predicted accident frequency for four-leg STOP-controlled intersections; and
N_{i3} = predicted accident frequency for four-leg signalized intersections.

Similarly, the total observed accident frequency for an EB analysis unit is the sum of O_{rs}, O_{i1}, O_{i2}, and O_{i3}.

The aggregation process is conducted as follows:

Step 8A—Select as candidates for further aggregation all analysis units consisting exclusively of tangent roadway segments with grades less than 5 percent for which the predicted injury accident frequency is less than 1/k.

Step 8B—For each of the candidate analysis units, compute the ratio of the predicted total accident frequency to the observed total accident frequency [i.e., N/O, or $(N_{rs}+N_{i1}+N_{i2}+N_{i3})/(O_{rs}+O_{i1}+O_{i2}+O_{i3})$].

Step 8C—Select the candidate with the lowest predicted accident frequency. If no more candidates remain, go to step 8F.

Step 8D—Identify and select another candidate whose ratio of predicted to total accident frequency is closest to that of the candidate selected in step 8C. The two selected candidate do not necessarily need to be geographically adjacent. If there are no remaining candidates other than the candidate selected in step 8C, then select the analysis unit of the same type (i.e., a tangent roadway segment with grades less than 5 percent) with predicted injury accident frequency greater or equal to 1/k whose ratio of predicted to total accident frequency is closest to that of the selected candidate. If there are no eligible analysis units, including even those with predicted injury accident frequency greater than 1/k, proceed to step 8F.

Step 8E—Combine the analysis unit selected in step 8C with the analysis unit selected in step 8D, add together their predicted and observed accident frequencies (i.e., sum the values of N_{rs}, N_{i1}, N_{i2}, N_{i3}, O_{rs}, O_{i1}, O_{i2}, and O_{i3}, for total accidents and by accident severity level), and recompute the ratio of predicted and observed total accident frequency. If the predicted injury accident frequency for the combined section is greater than 1/k, then go to step 8F. If the predicted injury accident frequency is still less than 1/k, then go back to step 8D and identify another section to combine.

Step 8F—Repeat steps 8A through 8E for each of the following types of analysis units, in turn:

- Tangent segments with grades greater than or equal to 5 percent.
- Horizontal curve segments.
- Tangent segments with grades less than 5 percent in passing lanes.
- Tangent segments with grades greater than or equal to 5 percent in passing lanes.
- Horizontal curve segments in passing lanes.
- Tangent segments with grades less than 5 percent in short four-lane sections.
- Tangent segments with grades greater than or equal to 5 percent in short four-lane sections.
- Horizontal curve segments in short four-lane sections.
- Tangent segments with grades less than 5 percent in two-way left-turn lanes.
- Tangent segments with grades greater than or equal to 5 percent in two-way left-turn lanes.
- Horizontal curve segments in two-way left-turn lanes.
- Three-leg STOP-controlled intersections.
- Four-leg STOP-controlled intersections.
- Four-leg signalized intersections.

Step 8G—If analysis units with predicted injury accident frequency less than 1/k remain, repeat steps 8A through 8E joining together analysis units of disparate types in the following order of descending priority:

- Join tangent roadway segments of any type to other tangent roadway segments of any type.
- Join horizontal curve segments of any type to horizontal curve segments of any type.
- Join three-leg STOP-controlled intersections to other three-leg STOP-controlled intersections.
- Join four-leg STOP-controlled intersections to other four-leg STOP-controlled intersections.
- Join four-leg signalized intersections to other four-leg signalized intersections.
- Joint tangent roadway segments with horizontal curves.
- Join three-leg STOP-controlled intersections to four-leg STOP-controlled intersections.
- Join signalized intersection with STOP-controlled intersections.
- Join roadway segments and intersections.

The aggregation process can be stopped whenever no analysis units with predicted injury accident frequency less than 1/k remain or whenever the entire project has been joined into a single analysis unit.

Step 9—For each EB analysis unit, as joined in Step 8, apply the EB procedure by computing the expected accident frequency for the before period as a weighted average of the predicted and observed accident frequencies.

The next step is to apply the EB procedure by computing a weighted average of the predicted and observed accident frequencies for the before period. For any EB analysis unit that is composed of entirely of roadway segments or composed entirely of a single intersection type, compute the weight to be placed on predicted accident frequency using equation (33) and then compute the expected accident frequency using equation (32).

If an EB analysis unit combines roadway segments and intersections, or more than one type of intersection, two additional factors must be accounted for. First, the overdispersion parameter, k, in the denominator of equation (33) is no longer uniquely defined, because two or more base models with differing overdispersion parameters must be considered. Second, it can no longer be assumed, as is normally done, that the expected numbers of accidents for the roadway segments and intersections, or for the different types of intersections are statistically correlated with one another. Rather, an estimate of expected accidents should be computed based on the assumption that the different entities are statistically independent (ñ=0) and on the alternative assumption that they are perfectly correlated (ñ=1). The expected accident frequency is then estimated as the average of the estimates for ñ=0 and ñ=1. This approach is implemented with equations (34) through (38).

Step 9 is applied both to predicted and observed total accident frequencies and to predicted and observed accident frequencies by accident severity level. Since these computations are independent, the expected accident frequencies by severity level may not sum to the expected total accident frequency. A correction is made as follows so that the expected accident frequencies for the individual severity levels do sum to the expected total accident frequency:

$$E_{fi/corr} = E_{tot} \left(\frac{E_{fi}}{E_{fi} + E_{pdo}} \right) \quad \textbf{(40)}$$

where:

$$E_{pdo/corr} = E_{tot} \left(\frac{E_{pdo}}{E_{fi} + E_{pdo}} \right) \quad \textbf{(41)}$$

$E_{fi/corr}$	=	expected accident frequency for fatal and injury accidents (corrected);
E_{pdo}	=	expected accident frequency for property-damage-only accidents (corrected);
E_{tot}	=	expected accident frequency for total accidents as estimated with equations (34) to (38);
E_{fi}	=	expected accident frequency for fatal and injury accidents as estimated with equations (34) to (38); and
E_{pdo}	=	expected accident frequency for property-damage-only accidents as estimated with equations (34) to (38).

Step 10—For each EB analysis unit, disaggregate the expected total accident experience during the before period back to the original roadway segments and intersections identified in step 2.

If an EB analysis unit, consists of more than one roadway segment or intersection, the expected accident frequency (N_{xx}) for the EB analysis unit should be disaggegated back to the original roadway segments and intersections in proportion to their original predicted accident frequencies. In other words:

$$E_p = E_{au} \left(\frac{N_p}{N_{au}} \right) \quad \textbf{(42)}$$

where:

E_p = expected accident frequency for an individual homogeneous roadway segment or intersection;

E_{au} = expected accident frequency for an EB analysis unit combining two or more roadway segments or intersections;

N_p = original predicted accident frequency for the individual homogeneous roadway segment or intersection from step 5;

N_{au} = original predicted accident frequency for all roadway segments and intersections that make up the analysis unit, as aggregated in steps 6 and 8.

Repeat this process for each of the original roadway segments and intersections, for total accidents and for each severity level.

Step 11—Apply ADT growth factors and/or AMFs for geometric changes to convert the expected accident frequency for the before period to an expected accident frequency for the proposed project during the specified future time period.

At the conclusion of step 10, E_p represents the expected accident frequency for a given roadway segment or intersection during the before period. To obtain an estimate of expected accident frequency in a future period (the *after* period), the estimate must be corrected for (1) any difference in the duration of the before-and-after periods; (2) any growth or decline in ADTs between the before-and-after periods; and (3) any changes in geometric design or traffic control features between the before-and-after periods that affect the values of the AMFs for the roadway segment or intersection. The expected accident frequency for a roadway segment or intersection in the after period can be estimated as:

$$E_f = E_p\ (N_{bf} / N_{bp})\ (AMF_{1f} / AMF_{1p})\ (AMF_{2f} / AMF_{2p})\ ...\ (AMF_{nf} / AMF_{np}) \qquad \textbf{(43)}$$

where:

E_f = expected accident frequency during the future time period for which accidents are being forecast for the analysis segment or intersection in question;

E_p = expected accident frequency for the past time period for which accident history data were available;

N_{bf} = number of accidents forecast by the base model using the future ADT data, the specified nominal values for geometric parameters, and—in the case of an analysis segment—the actual length of the analysis segment;

N_{bp} = number of accidents forecast by the base model using the past ADT data, the specified nominal values for geometric parameters, and—in the case of an analysis segment—the actual length of the analysis segment;

AMF_{xf} = value of the x^{th} AMF for the geometric conditions planned for the future (i.e., proposed) design; and

AMF_{xp} = value of the x^{th} AMF for the geometric conditions for the past (i.e., existing) design.

Because of the form of the base model for roadway segments, the ratio N_{bf} / N_{bp} is the same as the ratio of the traffic volumes, ADT_f / ADT_p. However, for intersections, the ratio N_{bf} / N_{bp} must be evaluated explicitly with the base models because the intersection base models incorporate separate major- and minor-road ADT terms with differing coefficients. In applying equation (43), the values of N_{bp}, N_{bf}, AMF_{xp}, and AMF_{xf} should be based on the average ADTs during the entire before or after period, respectively.

Equation (43) is applied to total accident frequency. The expected future accident frequencies by severity level should also be determined by multiplying the expected accident frequency from the before period for each severity level by the ratio E_f/E_p.

In the case of minor changes in roadway alignment (i.e., flattening a horizontal curve), the length of an analysis segment may change from the past to the future time period. In this case, the length of the analysis segment for the existing condition is used to determine N_{bp} and the modified length of the analysis segment for the planned condition is used to determine N_{bf}. This implicitly incorporates the assumption that, if the length of the analysis segment is changed, its base accident rate (per million-vehicle miles) remains constant and, therefore, the accident frequency increases or decreases in proportion to length. Of the course, the AMF ratios that also appear in equation (43) will account for any change in geometrics (i.e., reduction in radius of curvature) that accompany a change in length.

Step 12—Summarize and present the predictions in useful formats for the IHSDM user.

In the final step, the predicted accident frequencies are summarized and presented in useful formats for IHSDM users. The data presented include, at a minimum:

- Accident frequencies for the project as a whole including:
    - -- Total accident frequency.
    - -- Accident frequency by severity level.
    - -- Accident frequency by accident type.

The accident severity and accident type distribution for each roadway segment and intersection in the specified future time period is based on the default accident severity and accident type distributions presented in tables 1 and 2.

- Accident frequencies for individual roadway segments and intersections, expressed as accident rates per mi per year or million veh-mi for roadway segments and accident rates per million entering vehicles for intersections, so that accident “hot spots” that might be corrected through design improvements are evident.

Estimated accident frequencies could also be broken down by individual years of the evaluation period. However, this is not normally done because the combined estimates across all years of the evaluation period are generally of greatest interest to safety analysts. Predicted accident frequencies for a multiyear period are likely to be more accurate than predicted accident frequencies for any particular year.

7. CONCLUSIONS, RECOMMENDATIONS, AND FUTURE ENHANCEMENTS

This section presents the conclusions concerning the accident prediction algorithm developed in this report and presents recommendations for possible future enhancements of the algorithm.

Conclusions

The primary conclusion of this report is that an accident prediction algorithm has been developed and that this algorithm appears to be a useful tool for predicting the safety performance of rural two-lane highways. The primary strengths of the algorithm are as follows:

- The algorithm makes quantitative estimates of accident frequency and of the accident severity and accident type distributions for any two-lane highway section or project.
- The algorithm has been developed with a modular structure that combines base models and AMFs. The base models serve as scale factors to assure that the magnitude of the predicted accident frequency is appropriate, while the AMFs assure that the predicted accident frequency is sensitive to site-specific geometric and traffic control features.
- The use of AMFs that are separate from the base models assures that the effects of individual geometric design and traffic control features are not dependent upon inappropriate regression coefficients that are too large, too small, or in the wrong direction. Each AMF has been developed by a panel of experts to represent the best information currently available on the safety effects of that particular geometric design or traffic control feature.
- The modular structure makes the algorithm easy to update as better information, including new research results, become available.
- A calibration procedure is provided to allow individual highway agencies to adapt the algorithm to the safety conditions present on their rural two-lane highway system. The calibration procedure allows IHSDM users to adjust the predicted accident frequencies for agency-to-agency and State-to-State differences in factors such as accident reporting thresholds, accident reporting practices, animal populations, driver populations, and climates.
- A procedure based on the EB method allows users to combine accident predictions obtained from the algorithm with observed site-specific accident history data.

The major weakness of the algorithm is that it incorporates the effects on safety of most, but not all, geometric and traffic control features of interest to highway agencies. The algorithm incorporates only those features whose effects were considered by a panel of experts to be well established in quantitative terms. Geometric and traffic control features that are poorly understood, or not understood at all, have necessarily been omitted. The model generally treats the effects of individual geometric design and traffic control features as independent of one another and ignores potential interactions between them. It is likely that such interactions exist and, ideally, they should be accounted for in the accident prediction algorithm. However, such interactions are poorly understood and none could be quantified by the expert panels that participated in the development of the algorithm. It is the assessment of the expert panels that the base models and AMFs presented in this report represent the current state of knowledge about safety on rural two-lane highways and cannot be improved without further research. The next section of report discusses potential areas to which future research might be directed to improve the model.

FHWA plans to incorporate the accident prediction algorithm for rural two-lane highways presented in this report in software for implementation as part of the IHSDM. A stand-alone version of the software may also be available for use independent of a CAD system.

Future Enhancement of the Accident Prediction Algorithm

It is recommended that future enhancements be made to the accident prediction algorithm as further research is completed and that forthcoming research on rural two-lane highways be structured so that results are obtained in a form that can be directly implemented in the accident prediction algorithm. It is also recommended that a program of additional research be undertaken with the specific goal of filling gaps in the accident prediction algorithm and expanding its scope. Specific areas for future enhancement of the accident prediction algorithm are discussed below.

Base Models

The base models for roadway sections and for three- and four-leg STOP-controlled intersections appear to be well established and there is no immediate need for work to improve them. By contrast, the base model for four-leg signalized intersections is based on a small sample size (only 49 intersections split between two different States). The resulting models were not as satisfactory as desired, and it was difficult to choose among the available candidates. Signalized intersections are relatively rare on rural two-lane highways, so the limitations of the base model for signalized intersections do not overly limit the utility of the algorithm, but it would be desirable to assemble a larger database on signalized intersections on rural two-lane highways for the purpose of developing an improved base model. It would also be desirable to develop a base model for three-leg signalized intersections on rural two-lane highways, which are outside the scope of the current accident prediction algorithm.

A minor drawback of the base model for roadway segments is that it was based on sites with a mix of shoulder types, but the shoulder type itself was not a statistically significant factor in the model. This is understandable given the small effect of shoulder type on safety based on the expert panel's assessment shown in table 3. However, for consistency, if the roadway segment base model should be updated in the future, it would be desirable to (1) include an effect of shoulder type in the base model if found to be statistically significant; or (2) omit the shoulder type factor from the model and develop the model solely with data for sites with paved shoulders.

Accident Modification Factors

The accident prediction algorithm omits AMFs for several geometric design and traffic control features which the accident prediction algorithm should desirable address. Other AMFs that are included in the model could be improved through further research. These improvement needs are described below:

Roadway Segments

One of the greatest limitations of the roadway segment algorithm is the lack of an AMFs for bridge width. The expert panel on roadway segments strongly desired to include a bridge width factor in the algorithm because narrow bridges are known to be associated with accident concentrations on two-lane highways. However, the panel found that even the best study of the relationship between bridge width and safety had a major flaw that limited its use in the algorithm. The flaw was that the study included only bridges that had experienced one or more accidents during the study period.[41] Omission of sites which have experienced no accidents is a known source of bias in accident research. It is recommended that a well designed study of the relationship between bridge width and accidents on rural two-lane highways be undertaken and that its results, if found to be satisfactory, be incorporated in the accident prediction algorithm.

The roadway segment algorithm lacks an AMF to account for the effect on safety of vertical curve design and stopping sight distance. This effect has never been satisfactorily quantified, and recent research suggests that the safety effect of limited stopping sight distance at a crest vertical curve is relatively small.[42] However, should this effect—even if small—be reliably quantified in the future, it would be desirable to include it in the accident prediction algorithm.

It would be desirable to improve the representation of driveway effects in the accident prediction algorithm for roadway effects. The algorithm currently bases driveway effects on the driveway density (driveways per mi). The AMF for driveway density, based on a regression equation, indicates that there is greater sensitivity of safety to driveway density at lower ADT than at higher ADT. This appears to be the opposite of what might be expected. Furthermore, it would be more desirable to develop a method to quantify the safety effect of each individual driveway, but such a method does not currently exist. Such an approach would also require the user to supply detailed data on individual driveway locations and types (e.g., commercial vs. residential) and driveway traffic volumes which are not necessarily available to all users.

The effects of passing lanes and shoulder width are treated as independent in the accident prediction algorithm, but there may in fact be an interaction between them. In the research that established the safety effects of passing lanes, some of the sites at which passing lanes were installed may also have had full shoulders provided as part of the same project. Other sites may have had a portion of the shoulder converted to the passing lane. If further research quantifies the separate effects of passing lanes and shoulder widths, or the interactions between them, it would be desirable to incorporate these effects in the accident prediction algorithm.

The effect of center two-way left-turn lanes currently included in the accident prediction algorithm is clearly an oversimplification of a much more complex effect. Many evaluations of two-way left-turn lanes have been conducted, but little of this work is specific to rural two-lane highways. Further research to improve the AMF for two-way left-turn lanes would be desirable.

The roadside design AMF incorporated in the model is based on a qualitative roadside hazard rating system (a subjective 1 to 7 rating scale), rather than addressing the explicit effects of specific roadside design features. It would be desirable to provide the capability for individual roadside design features to be evaluated explicitly in situations where detailed data on roadside design features are available. Research is currently underway as part of the NCHRP program which may lead to such a capability.[22]

Intersections

Two undesirable omissions in the accident prediction algorithm for at-grade intersections are the lack of effects for roadside design and driveways in the vicinity of an intersection. The accident prediction algorithm for roadway segments includes the effect of both roadside design and driveways over the entire length of a study section or project, but the intersection algorithm places no special weight on restrictive roadside design at an intersection or driveways located near an intersection. The omission of roadside design issues at intersections may be a minor limitation, because run-off-the-road accidents are generally understood to be a roadway segment, rather than an intersection, problem. For example, table 2 shows that the proportion of intersection-related single-vehicle run-off-the-road accidents is small. However, poor roadside design in the vicinity of an intersection could increase the severity of multiple-vehicle accidents in which one or more of the involved vehicles leaves the traveled way. The lack of an effect for driveways near intersections is an omission of greater concern, although it is not as serious an omission on rural two-lane highways as it would be on urban or suburban arterials. Driveways near intersections are known to be a safety-related access management concern, but the expert panel found that the safety effects of driveways near intersections have not been well quantified. Research on this issue should be encouraged and should address, at a minimum, the type of driveway (e.g., commercial vs. residential), the distance from the intersection to the driveway, and the presence or absence of access control measures that restrict turning maneuvers (e.g., medians or turn prohibitions).

It would be desirable to improve the AMFs for left- and right-turn lanes at intersections. The expert panel found substantial past research on the effects of left- and right-turn lanes, but no definitive results. The AMFs included in the accident prediction algorithm relied heavily upon expert judgment in interpreting the available results and additional research to improve these AMFs would be desirable. FHWA has such research underway, and this research may lead to an update of the accident prediction algorithm.

A final concern with the accident prediction algorithm for intersections is that it does not address before-and-after evaluations of improvement projects in which an existing STOP-controlled intersection is signalized. It would be desirable to develop a specific AMF for such projects based on well-designed before-and-after studies.

Accident Prediction Algorithms for Other Facility Types

The accident prediction algorithm presented in this report applies only to rural two-lane highways. Rural two-lane highways were selected for the initial development of the IHSDM because existing rural two-lane highways have a wide range of variation in the quality of their design features and, therefore, present substantial opportunities for improvement of safety. Most freeways, by contrast, have been built much more recently and are more consistent in their design features.

It would be desirable to expand the accident prediction algorithm to other facility types, following the general approach developed for the accident prediction algorithm for rural two-lane highways presented in this report. It is recommended that the scope of the accident prediction algorithm be expanded in the following priority order: rural multilane highways, urban and suburban arterials, and, finally, freeways.

8. REFERENCES

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APPENDIX A

EXPERT PANEL MEMBERSHIP

The following individuals participated in the expert panels that established the accident modification factors:

EXPERT PANEL ON ROADWAY SECTIONS

Forrest M. Council	University of North Carolina
Douglas W. Harwood	Midwest Research Institute
Ezra Hauer	University of Toronto
Timothy R. Neuman	CH2M/Hill
Andrew Vogt	Pragmatics, Inc.
Charles V. Zegeer	University of North Carolina

EXPERT PANEL ON AT-GRADE INTERSECTIONS

Forrest M. Council	University of North Carolina
Kay Fitzpatrick	Texas A&M University
Douglas W. Harwood	Midwest Research Institute
Ezra Hauer	University of Toronto
Warren E. Hughes	Bellomo-McGee, Inc.
Patrick T. McCoy	University of Nebraska
Timothy R. Neuman	CH2M/Hill
Andrew Vogt	Pragmatics, Inc.

INTERNATIONAL RESOURCE PERSONS

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Joe G. Bared	Federal Highway Administration
Michael Dimiauta	A/E Group, Inc.
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Gregory L. Giering	A/E Group, Inc.
Michael S. Griffith	Federal Highway Administration
Raymond A. Krammes	Federal Highway Administration
Jeffrey F. Paniati	Federal Highway Administration
Justin G. True	Federal Highway Administration
Davey L. Warren	Federal Highway Administration

These expert panels were central to the approach used in developing the accident prediction algorithm, and the contribution of these panel members to the results presented here is gratefully acknowledged.

APPENDIX B

DEVELOPMENT OF BASE MODELS

The base models for the accident prediction algorithm were developed as part of the preparation of two FHWA reports, *Accident Models for Two-Lane Rural Roads: Segments and Intersections*, and *Accident Models for Rural Intersections: 4-Lane by 2-Lane Stop-Controlled and 2-Lane by 2-Lane Signalized*.[3,5] This appendix describes the data base development, the base model for roadway segments, and the base models for at-grade intersections.

Data Base Development

The base models were developed with geometric design, traffic control, traffic volume, and accident data on roadway sections and intersections on rural two-lane highways in California, Michigan, Minnesota, and Washington. These data were obtained from the FHWA Highway Safety Information System (HSIS). The geometric design data in the HSIS files, from databases maintained by the States identified above, were supplemented with additional data obtained by Vogt and Bared from field measurements and photolog review. These efforts are described more fully in two FHWA reports and a published paper.[3,4,5]

Base Model for Roadway Segments

In the modeling of roadway segment accidents, the dependent variable included five years of accident data (1985-1989) for 619 rural two-lane roadway segments in Minnesota and 3 years of accident data (1993-1995) for 712 roadway segments in Washington. The model development excluded roadway segments within 76 m (250 ft) of an at-grade intersection and excluded the (relatively few) accidents that occurred more than 76 m (250 ft) from an intersections but were identified by the investigating officer as related to an intersection.

The independent variables representing geometric, design traffic control. and traffic volume used in modeling included:

- Exposure (million vehicle-miles of travel).
- State in which the roadway section is located (Minnesota/Washington).
- Lane width.
- Shoulder width.
- Roadside hazard rating.
- Driveway density.

- Horizontal curvature.
- Grade rate for crest vertical curves.
- Percent grade for straight grades.

All of these independent variables were found to have a statistically significant relationship to roadway section accidents.

The base model for roadway segments was developed from the HSIS roadway segment data for rural two-lane highways in Minnesota and Washington. The base model is presented below:

$$N_{br} = EXPO \exp(0.6409 + 0.1388\,STATE - 0.0846\,LW - 0.0591\,SW + 0.0668\,RHR + 0.0084\,DD) \left(\Sigma WH_i \exp(0.0450\,DEG_i)\right) \left(\Sigma WV_i \exp(0.4652\,V_i)\right) \left(\Sigma WG_i \exp(0.1048\,GR_i)\right) \qquad \textbf{(42)}$$

where: N_{br} = predicted number of total accidents per year on a particular roadway segment;

EXPO = exposure in million vehicle-miles of travel per year = (ADT)(365)(L)(10^{-6});

ADT = average daily traffic volume (veh/day) on roadway segment;

L = length of roadway segment (mi);

STATE = location of roadway segment (0 in Minnesota, 1 in Washington);

LW = lane width (ft); average lane width if the two directions of travel differ;

SW = shoulder width (ft); average shoulder width if the two directions of travel differ;

RHR = roadside hazard rating; this measure takes integer values from 1 to 7 and represents the average level of hazard in the roadside environment along the roadway segment. (For the development of the roadside hazard rating, see Zegeer et al.; for definitions of individual rating levels, see Appendix D.);[6]

DD = driveway density (driveways per mi) on the roadway segment;

WH_i = weight factor for the i^{th} horizontal curve in the roadway segment; the proportion of the total roadway segment length represented by the portion of the i^{th} horizontal curve that lies within the segment. (The weights, WH_i, must sum to 1.0.);

DEG_i = degree of curvature for the i^{th} horizontal curve in the roadway segment (degrees per 100 ft);

WV_j = weight factor for the j^{th} crest vertical curve in the roadway segment; the proportion of the total roadway segment length represented by the portion of the j^{th} crest vertical curve that lies within the segment. (The weights, WV_j, must sum to 1.0.);

V_j = crest vertical curve grade rate for the j^{th} crest vertical curve within the roadway segment in percent change in grade per 31 m (100 ft) = $|g_{j2} - g_{j1}|/l_j$;

g_{j1}, g_{j2} = roadway grades at the beginning and end of the j^{th} vertical curve (percent);

l_j = length of the j^{th} vertical curve (in hundreds of feet);

WG_k = weight factor for the k^{th} straight grade segment; the proportion of the total roadway segment length represented by the portion of the k^{th} straight grade segment that lies within the segment. (The weights, WG_k, must sum to 1.0.); and

GR_k = absolute value of grade for the k^{th} straight grade on the segment (percent).

The model was developed with extended negative binomial regression analysis. This extension of the standard negative binomial regression analysis technique was devised by Miaou.(3) In negative binomial models for roadway segments, the mean number of accidents in a specified time period is typically represented in the form:

$$EXPO \exp(aX + bY + ...)$$

which is equivalent to:

$$EXPO \exp(aX) \exp(bY) ...$$

where EXPO is a measure of exposure, X and Y are measures of roadway segment characteristics, and a and b are appropriate regression coefficients.

The extended negative binomial regression analysis technique devised by Miaou replaces some of the factors exp(aX) by expressions of the form:

$$w_1 \exp(aX_1) + w_2 \exp(aX_2) + ... + w_m \exp(aX_m)$$

where X_1, X_2, ..., X_m are local variables along the roadway segment, characterizing for subsegments what X attempts to measure for the entire segment. For example, X_1, ..., X_m might represent the degree of curvature for individual horizontal curves while X is the average degree of curvature for the roadway segment as a whole. Thus, X is a composite variable, while the X_is represent variation within the segment. Such variation occurs for many variables used in accident modeling of roadway segments, even on supposedly homogeneous segments. Notable examples are degree of curvature, grade, and change of grade per unit length. The variable w_i is the proportion of the segment length to which the value X_i applies. (The default value for X_i is assumed to be zero if there is no horizontal or vertical curve or if the grade is level, and an artificial subsegment with this value is added, if necessary, so that the weights, w_i, always sum to 1.0.)[3]

The extended negative binomial regression model decomposes the roadway segment into subsegments within which the roadway characteristic measured by X is constant. If this is done for two or more variables (e.g., X, Y, ...), the method assumes that the variables are independent of one another, so that the value Y_j occurs with a particular value of X_i. Although such independence cannot be assured, the extended negative binomial regression model attempts to capture the effect of variation within a segment in an additive manner consistent with the basic form of the model.[3]

Table 29 summarizes the model presented in equation (42) with the coefficient, standard deviation, and significance level (p) for each independent variable and the overdispersion parameter (k). The goodness-of-fit measures for the roadway segment base model include R^2, the traditional measure of the percentage of variation in accident frequency explained by the independent variables in the model, as well as R_k^2, defined as:

$$R_k^2 = 1 - \frac{k}{k_{max}} \qquad \textbf{(43)}$$

Where:

k = the overdispersion parameter for the regression model; and
k_{max} = the overdispersion parameter in a model with no covariables (the so-called "zero model").

This latter measure of goodness of fit, R_k^2, has been proposed by Miaou.[9] For the roadway segment base model in equation (42), the values of the goodness-of-fit measures are $R^2 = 0.6547$ and $R_k^2 = 0.8291$.

Table 29. Model Parameters and Goodness of Fit for Equation (42).

	Independent Variable									
Parameter	Intercept	State	LW	SW	RHR	DD	DEG	V	GR	Overdispersion parameter (k)
Coefficient	0.6409	0.1388	-0.0846	-0.0591	0.0668	0.0084	0.0450	0.4652	0.1048	0.3056
Standard deviation	0.5008	0.0659	0.0425	0.0114	0.0211	0.0026	0.0078	0.1260	0.0287	0.0331
Significance level (p)	0.2006	0.0351	0.0465	0.0001	0.0015	0.0011	0.0001	0.0002	0.0003	0.0001

Note: The values of the goodness-of-fit measures are $R^2 = 0.6547$ and $R_k^2 = 0.8291$

Table 30 presents descriptive statistics for the variables in the roadway segment model.

Table 30. Descriptive Statistics for Roadway Segments Used in Modeling.

Variable	Mean	Standard deviation	Minimum	25th percentile	Median	75th percentile	Maximum
MINNESOTA (619 roadway segments)							
ADT (veh/day)	2,402	1,937	208	1,176	1,866	2,900	15,162
L (mi)	1.14	1.30	0.10	0.26	0.66	1.50	8.24
LW (ft)	11.54	0.67	10.00	11.00	12.00	12.00	12.00
SW (ft)	7.08	2.44	0.00	6.00	8.00	8.00	12.00
RHR	2.14	0.98	1.00	1.00	2.00	3.00	6.00
DD (mi^{-1})	6.58	10.25	0.00	0.89	3.73	7.68	100.00
DEG (degrees/100 ft)	0.51	0.95	0.00	0.00	0.08	0.62	7.50
V (percent/100 ft)	0.066	0.092	0.00	0.007	0.037	0.086	0.888
GR (percent)	0.38	0.52	0.00	0.10	0.24	0.45	4.46
WASHINGTON (712 roadway segments)							
ADT (veh/day)	3,352	3,199	159	1,261	2,239	4,455	17,766
L (mi)	0.75	0.83	0.10	0.27	0.554	0.948	13.23
LW (ft)	11.37	0.56	9.00	11.00	11.00	12.00	12.00
SW (ft)	5.01	2.35	0.00	3.00	5.00	7.00	10.00
RHR	3.67	1.57	1.00	2.00	3.00	6.00	7.00
DD (mi^{-1})	10.12	12.41	0.00	2.07	6.12	13.61	85.07
DEG (degrees/100 ft)	1.03	2.13	0.00	0.00	0.32	1.31	30.55
V (percent/100 ft	0.068	0.127	0.000	0.000	0.026	0.083	1.997
GR (percent)	0.92	1.17	0.00	0.20	0.49	1.13	6.92

Conversion: 1 ft = 0.305 m; 1 mi = 1.61 km

The exposure variable, EXPO, was treated as a scale factor in the development of the model. Therefore, even though multiple years of accident data were used in developing the model, the expected annual accident frequency can be determined from the model if EXPO is determined as (ADT)(365)(L)(10^{-6}).

Other variables investigated in the development of the roadway segment base models included posted speed limit, truck percentage, and intersection density (i.e., number of intersections per mile). Posted speed limit was found to be negatively correlated with accident frequency, while truck percentage and intersection density were positively correlated with accident frequency. However, none of these three variables was statistically significant in regression models that included the variables listed above. Grade change per unit length of roadway was also considered for sag vertical curves and for all vertical curves (i.e. both sags and crests), but the version of the variable for crest vertical curves (V_j) had the greatest statistical significance and was, therefore, retained in the final model. Two weather-related variables were also investigated in the modeling for the Minnesota roadway segments—number of rain days and number of snow days per year. These variables were based on the climate district in which each roadway segment was located. These variables were found to be negatively correlated with accident frequency. Because these variables were not sufficiently local (i.e., they represented climate districts rather than the climate of individual roadway segments) and because they were only marginally statistically significant, they were not collected for the Washington data and were omitted from the final model.

Base Models for At-Grade Intersections

Base models have been developed for three types of at-grade intersections on rural two-lane highways. These are:

- Three-leg intersections with STOP control on the minor-road approach.
- Four-leg intersections with STOP control on the minor-road approach.
- Four-leg signalized intersections.

Models for each of these intersection types were developed using three different modeling approaches, described below. The available database used in modeling included 5 years of accident data (1985-89) for selected STOP-controlled intersections in Minnesota and 3 years of accident data (1993-1995) for selected signalized intersections in California and Michigan. Following the description of the modeling approaches, the models developed for each intersection type are presented.

The available accident data for at-grade intersections generally included accidents within 76 m (250 ft) of the intersection on the major road, typically a State highway. For minor roads that are also State highways, data were generally available for all accidents that occur within 76 m (250 ft) of the intersection. For minor roads that are not State highways, accidents on the minor road that are classified as intersection-related are typically assigned the milepost of the intersection on the major road and are therefore included in the available data. In Michigan, all minor-road accidents within 31 m (100 ft) of the intersection and, in California, all accidents within 76 m (250 ft) of the intersection are included in the available data. In Minnesota, all accidents that occurred on minor-road approaches and were identified as related to the intersection are included in the available data.

Modeling Approaches

Three different modeling approaches were employed to develop candidate base models for each intersection type. The approaches are:

- The dependent variable (number of accidents in a 5-year period) included only accidents that occurred within the curbline limits of a particular intersection or within 76 m (250 ft) of that intersection and were identified by the investigating officer as being related to the intersection.

- The dependent variable included only accidents that occurred within the curbline limits of a particular intersection or within 76 m (250 ft) of that intersection and were of accident types that are generally related to intersection operations.

- The dependent variable included all accidents that occurred within the curbline limits of a particular intersection or within 76 m (250 ft) of that intersection, whether those accidents were related to the intersection or not. However, the modeling technique used an offset to the observed accident frequency applied iteratively to account for the frequency of roadway section accidents that would be expected to occur on 152 m (500 ft) of the major roadway (76 m or 250 ft on either side of the intersection), as predicted by equation (37).

Each of these modeling techniques is described in greater detail in the following discussion. A fourth modeling technique that was considered but not used is explained at the end of this appendix.

The research reports by Vogt and Bared from the FHWA project that developed the base models and the related paper include only the models based on accidents that were identified by the investigating officer as being intersection related.[3,4,5] The models based on accident types that are generally related to intersection operations and on the iterative offset approach were developed as part of the same project but are documented only in this report.

Limitation of the Analysis to Intersection-Related Accidents

The first modeling approach used to develop base models for at-grade intersections was to limit the dependent variable to those accidents that occurred within the curbline limits of the intersection or within 76 m (250 ft) of the intersection on any approach and were identified by the investigating officer as related to the intersection. This definition is generally reasonable because the vast majority of intersection-related accidents on a rural two-lane highway would be expected to occur within 76 m (250 ft) of the intersection to which they are related. For example, traffic queues that extend more than 76 m (250 ft) from an intersection and which might lead to rear-end collisions remote from the intersection are much less common in rural than in urban areas.

Not all accidents within 76 m (250 ft) of an intersection would be expected to be related to the operation of that intersection. For example, collisions between motor vehicles and animals and collisions involving vehicles entering or leaving a driveway may occur within 76 m (250 ft) of an intersection but may have no particular relationship to the presence of that intersection. In other words, some accidents would be expected to occur on the roadway near the intersection whether the intersection were present or not. the first approach to modeling distinguishes between intersection-related and non-intersection-related accidents based on judgments made by the investigating officer or by an accident coder and recorded in the State's computerized accident record system. Nominally, such judgments make exactly the distinction that is desired, but there is concern that investigating officers and accident coders may not make such judgments consistently.

Once the dependent variable was defined, as described above, predictive models were developed using extended negative binomial regression in a manner similar to the development of the roadway segment model in equation (42).

Limitation of the Analysis to Selected Accident Types

The second modeling approach used was to limit the dependent variable to accidents that occurred within the curbline limits of the intersection and are of accident types that are generally related to intersection operations. The accident types classified as related to the intersection were:

- Rear-end collisions.
- Right-angle collisions.
- Left-turn collisions.
- Right-turn collisions.
- Sideswipe, same-direction collisions.

All other accidents types were classified as unrelated to the intersection.

The list of accident types classified as intersection-related and non-intersection-related were established through two special studies that involved a review of hard copy police accident reports and classification of accidents as intersection-related or non-intersection-related based on the judgment of safety experts.

This approach to modeling eliminates the concern raised in the first modeling approach about the potential misjudgment in classifying accidents made by investigating officers or accident coders. However, the approach based on classification of particular accident types as intersection-related has a similar concern since not every accident of the types identified above is actually related to an intersection and some accidents of other types are, in fact, intersection-related. For example, the proposed classification scheme based on accident types would classify all reported right-turn accidents within 76 m (250 ft) of the intersection as intersection-related, including accidents that are, in fact, related to turning movements at nearby driveways. The effectiveness of this classification could be improved by an agency by utilizing other fields of the accident record such as a driveway involvement indicator, if available, in classifying accidents.

Once the dependent variable was defined, as described above, predictive models were developed using extended negative binomial regression in a manner similar to the development of the roadway segment model in equation (42).

Use of All Accidents with an Offset for Expected Roadway-Segment Accidents

The third modeling technique uses as the dependent variable all accidents that occur within the curbline limits of a particular intersection and all accidents that occur within 76 m (250 ft) of that intersection, regardless of the accident type and regardless of the judgment made by the investigating officer or accident coder concerning the relationship of the accident to the intersection. Since this dependent variable includes some accidents that are clearly unrelated to the intersection, the expected frequency of roadway segment accidents, as predicted by equation (42), is used as an offset factor in the model development. This modeling used an iterative technique that makes successive approximations to the model coefficients.

The iterative offset modeling approach was performed as follows. Let:

$$Z_i = \exp(k_i + a_i X + b_i Y + \ldots) \qquad \textbf{(44)}$$

denote an equation with integer subscript i estimating the mean number of intersection-related accidents per unit time, Z_i, in terms of intersection characteristics (X, Y, ...). Let N denote the mean number of non-intersection-related accidents on a 152 m (500 ft) section of roadway containing the intersection (i.e., 76 m or 250 ft on either side of the intersection), as predicted by a roadway section model such as equation (42), as applied to Minnesota (STATE=0).

Let A be the mean number of accidents of all kinds per unit time within 76 m (250 ft) of the intersection. Let:

$$OFFSET_i = ln\left(\frac{N + Z_i}{Z_i}\right) \quad \textbf{(45)}$$

The modeling approach used is iterative. Equation (44) implies that the following relationship must be valid for iteration i+1:

$$Z_{i+1} = \exp(k_{i+1} + a_{i+1}X + b_{i+1}Y + ...) \quad \textbf{(46)}$$

A negative binomial model for A is sought of the form:

$$A = \exp(OFFSET_i + k_{i+1} + a_{i+1}X + b_{i+1}Y + ...) \quad \textbf{(47)}$$

which can be expressed as:

$$A = \exp(OFFSET_i)\ \exp(k_{i+1} + a_{i+1}X + b_{i+1}Y) \quad \textbf{(48)}$$

$$A = \exp(OFFSET_i)\ Z_{i+1} \quad \textbf{(49)}$$

$$A = \frac{N + Z_i}{Z_i} Z_{i+1} \quad \textbf{(50)}$$

The initial negative binomial model for intersection-related accidents is represented by an equation for Z_0 in the form of equation (44). The offset technique represented by equations (47) through (50) is applied repeatedly to obtain a sequence of new models for Z_1, Z_2, Z_3, ... This process is continued until the coefficients k_n, a_n, b_n, ... cease to change appreciably; i.e., until Z_{n+1}

is approximately the same as Z_n. The appropriate model for the mean number of intersection-related accidents is then:

$$Z = \exp(k_n + a_n X + b_n Y + \ldots) \tag{51}$$

In principle, $A = N + Z$, or $Z = A - N$. Thus from, the model for roadway segment accidents used to estimate N and the initial model for intersection-related accidents Z_0, the offset technique yields a final model for intersection-related accidents, Z. This final model depends on the choice of the model for N, but should not generally depend on the initial model for Z_0. Thus, initial estimates or starting values of the regression coefficients k_0, a_0, b_0, ... can be selected through engineering judgment or an alternative preliminary model.

Models Developed

The following discussion presents the candidate base models developed for three-leg STOP-controlled intersections, four-leg STOP-controlled intersections, and four-leg signalized intersections.

Three-Leg STOP-Controlled Intersections

Candidate base models were developed for three-leg intersections with STOP control on the minor-road approach. The dependent variables used in these models have been described above in the discussion of modeling techniques. All of the models for three-leg STOP-controlled intersections used a data sets of approximately 382 intersections in Minnesota including 5 years of accident data (1985-1989). There were minor variations in sample size from one model to the next because of small amounts of missing data. The candidate independent variables considered in predicting accidents at three-leg STOP-controlled intersections were:

- Average daily traffic volume on the major road.
- Average daily traffic volume on the minor road.
- Intersection skew angle.
- Posted speed limit on major road.
- Presence of a right-turn lane on the major road.
- Number of driveways on the major road within 76 m (250 ft) of the intersection.
- Grade rate for crest vertical curves on the major road within 76 m (250 ft) of the intersection.
- Radius of horizontal curves on the major road within 76 m (250 ft) of the intersection.
- Roadside hazard rating along the major road within 76 m (250 ft) of the intersection.

Table 31 presents descriptive statistics for these variables.

A candidate model for three-leg STOP-controlled intersections developed using the intersection-related accident definition based on the investigating officer's assessment of each accident is:

$$N_{bi} = \exp(-12.99 + 0.805\,ln\ ADT_1 + 0.504\,ln\ ADT_2 + 0.290\,VCI + 0.034\,HI \\ + 0.029\,SPDI + 0.173\,RHRI + 0.27\,RT + 0.0045\,SKEW_3) \quad \textbf{(52)}$$

Table 31. Descriptive Statistics for 382 Three-Leg STOP Controlled Intersections in Minnesota Used in Modeling.

Variable	Mean	Standard deviation	Minimum	25th percentile	Median	75th percentile	Maximum
ADT_1 (veh/day)	3,718	3,725	201	1,239	2,333	4,627	19,413
ADT_2 (veh/day)	408	531	5	103	237	478	4206
ln ADT_1	7.81	0.91	5.30	7.12	7.75	8.44	9.87
ln ADT_2	5.40	1.14	1.51	4.64	5.47	6.17	8.34
VCI (percent/100 ft)	0.142	0.300	0	0	0	0	4.39
HI (degrees/100 ft)	1.22	2.52	0	0	0	2	29
SPDI (mph)	52.8	4.6	22.5	52.5	55	55	55
RHRI	2.10	0.88	1	1	2	3	5
RT	0.42	0.49	0	0	0	1	1
$SKEW_3$ (degrees)	-0.67	24.98	-90	0	0	0	85.1
ND_1	1.24	1.44	0	0	1	2	9

Conversion: 1 ft = 0.305 m

where: N_{bi} = predicted number of total accidents per year at a particular intersection and within 76 m (250 ft) in either direction along the major road;

ADT_1 = average daily traffic volume (veh/day) on the major road; if the ADTs differ between the major-road legs, they should be averaged;

ADT_2 = average daily traffic volume (veh/day) on the minor road;

VI = crest vertical curve grade rate on the major road within 76 m (250 ft) of the intersection = (1/m) ΣV_i for all crest vertical curves wholly or partly within 76 m (250 ft) of the intersection;

m = number of crest vertical curves wholly or partly within 76 m (250 ft) of the intersection;

HI = horizontal curvature change rate on the major road within 76 m (250 ft) of the intersection = (1/n) ÓDEG_i for all horizontal curves wholly or partly within 76 m (250 ft) of the intersection;

n = number of horizontal curves within 76 m (250 ft) of the intersection;

SPDI = posted speed limit on the major road (mi/h);

RHRI = roadside hazard rating within 76 m (250 ft) of the intersection on the major road [see description of the variable RHR in Equation (42)];

RT = presence of right-turn lane on the major road (0 = no right-turn lane present; 1 = right-turn lane present); and

$SKEW_3$ = intersection angle (degrees) minus 90 for the angle between the major-road leg in the direction of increasing stations and a leg to the right; 90 minus intersection angle (degrees) for the angle between the major-road leg in the direction of increasing stations and a leg to the left.

Equation (52) includes all of the candidate independent variables except the number of driveways which was dropped because its coefficient was not statistically significant (p=0.5405) and because coefficient had a negative sign, which is opposite to the direction expected. Table 32 summarizes the model parameters and goodness of fit for the model in equation (52). Goodness of fit for models like equation (52) is measured by R^2 and R_k^2, as noted earlier for equation (42), as well as by R_{PD}^2. This last goodness-of-fit measure, R_{PD}^2, has been proposed by Fridstrøm for use with negative binomial models.[43] A description of this goodness-of-fit measure is also provided by Vogt and Bared.[3]

Table 32. Model Parameters and Goodness of Fit for Equation (52).

	Independent variable									Over-dispersion parameter (k)
Parameter	Intercept	*ln* ADT_1	*ln* ADT_2	VCI	HI	SPDI	RHRI	RT	$SKEW_3$	
Coefficient	! 12.99	0.805	0.504	0.290	0.034	0.029	0.173	0.27	0.0045	0.481
Standard deviation	1.15	0.064	0.071	0.294	0.033	0.018	0.068	0.140	0.0032	0.100
Significance level	0.000	0.0001	0.0001	0.323	0.3004	0.107	0.0108	0.0561	0.1578	0.0001

Note: The values of the goodness-of-fit measures are R^2 = 0.4409, R_k^2 = 0.7805, and R_{PD}^2 = 0.6279

The model in equation (52) was reevaluated including only those variables that were statistically significant with a significance level (p) of 0.150 or less. This model is presented below:

$$N_{bi} = \exp(-11.28 + 0.79\,ln\ ADT_1 + 0.49\,ln\ ADT_2 + 0.19\,RHRI + 0.28\,RT) \qquad \textbf{(53)}$$

Table 33 summarizes the model parameters and goodness of fit for the model in equation (53).

A candidate model developed for three-leg STOP-controlled intersections using only those accident types generally related to intersection operations is:

$$\begin{aligned} N_{bi} = \exp(&-12.82 + 1.001\,ln\ ADT_1 + 0.406\,ln\ ADT_2 + 0.22\,RHRI \\ &+ 0.33\,RT + 0.0040\,SKEW_3) \end{aligned} \qquad \textbf{(54)}$$

Equation (54) includes all of the candidate independent variables except speed limit (p=0.41), number of driveways (p=0.56), horizontal curvature (p=0.62), and vertical curvature (p=0.40) which were dropped because their coefficients were not statistically significant. Table 34 summarizes the model parameters and goodness of fit for the model in equation (54).

Table 33. Model Parameters and Goodness of Fit for Equation (53).

	Independent variable					Over-dispersion
Parameter	Intercept	*ln* ADT_1	*ln* ADT_2	RHRI	RT	parameter (k)
Coefficient	−11.28	0.79	0.49	0.19	0.28	0.54
Standard deviation	0.063	0.062	0.068	0.067	0.14	0.102
Significance level (p)	0.0001	0.0001	0.0001	0.0035	0.0402	0.0001

Note: The values of the goodness-of-fit measures are $R^2 = 0.3955$, $R_K^2 = 0.7546$, and $R_{PD}^2 = 0.6109$.

Table 34. Model Parameters and Goodness of Fit for Equation (54).

Parameter	Independent variable: Intercept	*ln* ADT_1	*ln* ADT_2	RHRI	RT	$SKEW_3$	Over-dispersion parameter (k)
Coefficient	−12.82	1.001	0.406	0.22	0.33	0.0040	0.46
Standard deviation	0.73	0.072	0.073	0.073	0.15	0.0029	0.106
Significance level	0.0001	0.0001	0.0001	0.0024	0.032	0.17	0.0001

Note: The values of the goodness-of-fit measures are $R^2 = 0.4181$, $R_k^2 = 0.8070$, and $R_{PD}^2 = 0.7228$.

The model in equation (54) was reevaluated including only those variables that were statistically significant with a significance level of 0.150 or less. This model is presented below:

$$N_{bi} = \exp(-13.01 + 1.015\,ln\ ADT_1 + 0.42\,ln\ ADT_2 + 0.23\,RHRI + 0.29\,RT) \qquad \textbf{(55)}$$

Table 35 summarizes the model parameters and goodness of fit for the model in equation (55).

Table 35. Model Parameters and Goodness of Fit for Equation (55).

Parameter	Independent variable: Intercept	*ln* ADT_1	*ln* ADT_2	RHRI	RT	Over-dispersion parameter (k)
Coefficient	−13.01	1.015	0.42	0.23	0.29	0.49
Standard deviation	0.72	0.072	0.071	0.073	0.14	0.108
Significance level (p)	0.0001	0.0001	0.0001	0.0020	0.041	0.0001

Note: The values of the goodness-of-fit measures are $R^2 = 0.4008$, $R_k^2 = 0.7953$, and $R_{PD}^2 = 0.7162$

A candidate model for three-leg STOP-controlled intersections developed with the iterative offset technique is:

$$\begin{aligned} N_{bi} = \exp(-12.40 + 0.74\,ln\ ADT_1 + 0.53\,ln\ ADT_2 + 0.36\,VCI + 0.028\,SPDI \\ + 0.14\,RHRI + 0.0063\,SKEW_3) \end{aligned} \qquad \textbf{(56)}$$

Equation (56) includes all of the candidate independent variables except number of driveways (p=0.35), presence of right-turn lane (p=0.47), and horizontal curvature (p=0.50) which were dropped because their coefficients were not statistically significant. Table 36 summarizes the model parameters and goodness of fit for the model in equation (56). It should be noted that the goodness-of-fit measure, R_{PD}^2, is not directly applicable to models developed with the iterative offset technique and, therefore, is not presented in Table 36. Moreover, R^2 must be interpreted with caution since it measures the goodness-of-fit for the combined roadway segment and intersection model for all accidents with 76 m (250 ft) of the intersection. Since most of these accidents are intersection-related, R^2 at least roughly measures the goodness of fit for intersection-related accidents

Table 36. Model Parameters and Goodness of Fit for Equation (56).

Parameter	Independent variable: Intercept	*ln* ADT_1	*ln* ADT_2	VCI	SPDI	RHRI	$SKEW_3$	Over-dispersion parameter (k)
Coefficient	−12.40	0.74	0.53	0.36	0.028	0.14	0.0063	0.52
Standard deviation	0.93	0.063	0.064	0.28	0.013	0.070	0.0029	0.090
Significance level (p)	0.0001	0.0001	0.0001	0.20	0.033	0.053	0.032	0.0001

Note: The values of the goodness-of-fit measures are R^2 = 0.4163, and R_k^2 = 0.5809.

The model in equation (56) was reevaluated including only those variables that were statistically significant with a significance level of 0.150 or less. This model is presented below:

$$N_{bi} = \exp(-12.25 + 0.75\,ln\ ADT_1 + 0.52\,ln\ ADT_2 + 0.026\,SPDI + 0.15\,RHRI + 0.0059\,SKEW_3) \qquad \textbf{(57)}$$

Table 37 summarizes the model parameters and goodness of fit for the model in equation (57).

A decision was made to use in the accident prediction algorithm the models containing only those independent variables that are statistically significant at a significance level of 0.15 or less, like equations (53), (55), and (57).

Table 37. Model Parameters and Goodness of Fit for Equation (57).

	Independent Variable						Over-dispersion parameter
Parameter	Intercept	*ln* ADT_1	*ln* ADT_2	SPDI	RHRI	$SKEW_3$	(k)
Coefficient	−12.25	0.75	0.52	0.026	0.15	0.0059	0.52
Standard deviation	0.92	0.062	0.064	0.013	0.065	0.0029	0.089
Significance level (p)	0.0001	0.0001	0.0001	0.046	0.019	0.043	0.0001

Note: The value of goodness-of-fit measures are R^2 = 0.4069 and R_k^2 = 0.5765.

Four-Leg STOP-Controlled Intersections

Candidate base models were developed for four-leg intersections with STOP control on the minor-road approaches. The dependent variables used in these models have been described above in the discussion of modeling techniques. All of the models for four-leg STOP-controlled intersections used a data sets of approximately 324 intersections in Minnesota including 5 years of accident data (1985-1989). There were minor variations in sample size from one model to the next because of small amounts of missing data. The candidate independent variables considered in predicting accidents at four-leg STOP-controlled intersections were:

- Average daily traffic volume on the major road.
- Average daily traffic volume on the minor road.
- Intersection skew angle.
- Posted speed limit on major road.
- Presence of right-turn lanes on the major road.
- Number of driveways within 76 m (250 ft) of the intersection.
- Grade rate for crest vertical curves within 76 m (250 ft) of the intersection.
- Radius of horizontal curves within 76 m (250 ft) of the intersection.
- Roadside hazard rating.

Table 38 presents descriptive statistics for these variables.

A candidate model for four-leg STOP-controlled intersections developed using the intersection-related accident definition is:

$$N_{bi} = \exp(-10.43 + 0.603\, ln\ ADT_1 + 0.609\, ln\ ADT_2 + 0.29\, VCI + 0.045\, HI + 0.019\, SPDI + 0.12\, ND_1 - .0049\, SKEW_4) \tag{58}$$

where:

ND_1 = number of driveways on the major road with 76 m (250 ft) of the intersection; and

$SKEW_4$ = intersection angle (degrees) expressed as one-half of the angle to the right minus one-half of the angle to the left for the angles between the major-road leg in the direction of increasing stations and the right and left legs, respectively.

Table 38. Descriptive Statistics for 324 Four-Leg STOP-Controlled Intersections in Minnesota.

Variable	Mean	Standard deviation	Minimum	25th percentile	Median	75th percentile	Maximum
ADT_1 (veh/day)	2,216	1,966	174	972	1,739	2,611	14,611
ADT_2 (veh/day)	304	383	7	105	191	365	3,414
ln ADT_1	7.42	0.75	5.16	6.88	7.46	7.87	9.59
ln ADT_2	5.25	0.97	1.93	4.65	5.25	5.90	8.14
VCI (percent/100 ft)	0.146	0.280	0	0	0.023	0.207	2.942
HI (degrees/100 ft)	0.46	1.08	0	0	0	0.25	8.00
SPDI (mi/h)	54.0	3.3	30	55	55	55	55
ND_1	0.61	1.14	0	0	0	1	6
$SKEW_4$ (degrees)	!0.14	18.34	-60.00	-0.44	0.00	0.58	75.00

Conversion: 1 ft = 0.305 m; 1 mi = 1.61 km

Equation (58) includes all of the candidate independent variables except roadside hazard rating (p=0.28) and presence of right-turn lanes (p=0.66) which were not statistically significant. Table 39 summarizes the model parameters and goodness of fit for the model in equation (58).

Table 39. Model Parameters and Goodness of Fit for Equation (58).

	Independent variable								Over-dispersion parameter (k)
Parameter	Intercept	$lnADT_1$	$lnADT_2$	VCI	HI	SPDI	ND_1	$SKEW_4$	
Coefficient	!10.43	0.603	0.609	0.29	0.045	0.019	0.12	!0.0049	0.205
Standard deviation	1.32	0.084	0.069	0.26	0.047	0.018	0.05	0.0033	0.065
Significance level	0.000	0.000	0.000	0.26	0.34	0.29	0.01	0.13	0.0016

Note: The values of goodness-of-fit are R^2 = 0.5944, R_k^2 = 0.8336, and R_{PD}^2 = 0.7364

The model in equation (58) was reevaluated including only those variables that were statistically significant with a significance level of 0.150 or less. This model is presented below:

$$N_{bi} = \exp(-9.34 + 0.60\,ln\ ADT_1 + 0.61\,ln\ ADT_2 + 0.13\,ND_1 - .0054\,SKEW_4) \quad \textbf{(59)}$$

Table 40 summarizes the model parameters and goodness of fit for the model in equation (59).

Table 40. Model Parameters and Goodness of Fit for Equation (59).

Parameter	Independent variable					Over-dispersion parameter (k)
	Intercept	*ln* ADT_1	*ln* ADT_2	ND_1	$SKEW_4$	
Coefficient	−9.34	0.601	0.61	0.13	−0.0054	0.24
Standard deviation	0.72	0.078	0.069	0.039	0.0034	0.071
Significance level (p)	0.0001	0.0001	0.0001	0.0009	0.108	0.0008

Note: The values of goodness-of-fit measures are $R^2 = 0.5662$, $R_k^2 = 0.8081$, and $R_{PD}^2 = 0.7326$

A candidate model developed using only those accident types generally related to intersection operations is:

$$N_{bi} = \exp(-9.40 + 0.55\,ln\ ADT_1 + 0.65\,ln\ ADT_2 + 0.31\,VCI + 0.14\,ND_1 - 0.0049\,SKEW_4) \quad \textbf{(60)}$$

Equation (60) includes all of the candidate independent variables except speed limit (p=0.25 to 0.50), presence of right-turn lane (p=0.59), and horizontal curvature (p=0.48) which were not statistically significant, and roadside hazard rating (p=0.12) which was marginally statistically significant but whose coefficient was negative, which is opposite to the direction expected. Table 41 summarizes the model parameters and goodness of fit for the model in equation (60).

The model in equation (60) was reevaluated including only those variables that were statistically significant with a significance level of 0.150 or less. This model is presented below:

$$N_{bi} = \exp(-9.30 + 0.53\,ln\ ADT_1 + 0.67\,ln\ ADT_2 + 0.15\,ND_1 - .0057\,SKEW_4) \quad \textbf{(61)}$$

Table 41. Model Parameters and Goodness of Fit for Equation (60).

Parameter	Independent variable						Over-dispersion parameter (k)
	Intercept	*ln* ADT_1	*ln* ADT_2	VCI	ND_1	$SKEW_4$	
Coefficient	−9.40	0.55	0.65	0.31	0.14	−0.0049	0.253
Standard deviation	0.77	0.085	0.073	0.27	0.05	0.0038	0.079
Significance level (p)	0.0001	0.0001	0.0001	0.25	0.004	0.1997	0.0014

Note: The values of goodness-of-fit measures are $R^2 = 0.5495$, $R_k^2 = 0.8131$, and $R_{PD}^2 = 0.7183$

Table 42. Model Parameters and Goodness of Fit for Equation (61).

Parameter	Independent variable					Over-dispersion parameter (k)
	Intercept	*ln* ADT_1	*ln* ADT_2	ND_1	$SKEW_4$	
Coefficient	−9.30	0.53	0.67	0.15	−0.0057	0.293
Standard deviation	0.78	0.085	0.074	0.043	0.0039	0.086
Significance level (p)	0.0001	0.0001	0.0001	0.0004	0.14	0.0007

Note: The values of goodness-of-fit measures are $R^2 = 0.5020$, $R_k^2 = 0.7835$, and $R_{PD}^2 = 0.7047$

Table 42 summarizes the model parameters and goodness of fit for the model in equation (61).

A candidate model for four-leg STOP-controlled intersections developed with the iterative offset technique is:

$$N_{bi} = \exp(-11.25 + 0.69\,ln\ ADT_1 + 0.56\,ln\ ADT_2 + 0.033\,SPDI + 0.11\,ND_1 - 0.21\,RT - .0059\,SKEW_4) \quad \textbf{(62)}$$

Equation (62) includes all of the candidate independent variables except horizontal curvature (p=0.51) and vertical curvature (p=0.30 to 0.41) which were not statistically significant, and roadside hazard rating (p=0.11) which was marginally statistically significant but whose coefficient was negative, which is opposite to the direction expected. Table 43 summarizes the model parameters and goodness of fit for the model in Equation (62). No value of R_{PD}^2 is computed for equation (62) because this goodness-of-fit measure is not directly applicable to iterative offset technique.

Table 43. Model Parameters and Goodness of Fit for Equation (62).

Parameter	Independent variable							Over-dispersion parameter (k)
	Intercept	*ln* ADT_1	*ln* ADT_2	SPDI	ND_1	RT	$SKEW_4$	
Coefficient	!11.25	0.69	0.56	0.033	0.107	!0.21	!0.0059	0.203
Standard deviation	1.59	0.080	0.073	0.021	0.054	0.12	0.0031	0.065
Significance level	0.0001	0.0001	0.0001	0.11	0.048	0.082	0.057	0.0018

Note: The values of goodness-of-fit measures are $R^2 = 0.5563$, and $R_k^2 = 0.7984$

All of the variables in equation (62) are statistically significant at the 0.15 significance level. Therefore, no alternative version of equation (62) was developed.

Four-Leg Signalized Intersections

Candidate base models were developed for four-leg signalized intersections. The dependent variables used in these models were those described above for the intersection-related and selected-accident-type modeling techniques. All of the models for four-leg signalized intersections used a data sets of 49 intersections, 18 in California and 31 in Michigan, with 3 years of accident data (1993-1995) for each intersection. The candidate independent variables considered in predicting accidents at four-leg signalized intersections on rural two-lane highways were:

- Average daily traffic volume on the major road.
- Average daily traffic volume on the minor road.
- Presence of protected left-turn signal phase on the major road.
- Grade rate for all vertical curves (including both crests and sags) within 244 m (800 ft) of the intersection along the major and minor roads.
- Percentage of minor-road traffic turning left at the signal for the morning and evening peak hours combined.
- Percentage of trucks (vehicles with more than four wheels) in the traffic stream entering the intersection from all approaches for morning and evening peak hours combined.
- Number of driveways within 76 m (250 ft) of the intersection on the major-road leg.

Table 44 presents descriptive statistics for these variables.

Table 44. Descriptive Statistics for 49 Four-Leg Signalized Intersections in California and Michigan Used in Modeling.

Variable	Mean	Standard deviation	Minimum	25th percentile	Median	75th percentile	Maximum
ADT_1 (veh/day)	10,491	4,331	4,917	7,568	8,900	13,133	25,133
ADT_2 (veh/day)	4,367	2,369	940	2,800	3,670	5,080	12,478
ln ADT_1	9.18	0.39	8.50	8.93	9.09	9.48	10.13
ln ADT_2	8.26	0.49	6.85	7.93	8.21	8.53	9.43
SUMLADT	17.44	0.65	16.34	16.94	17.49	17.78	19.14
$PCTLEFT_2$ (percent)	28.4	15.1	2.5	19.0	25.7	35.7	75.70
VEICOM (percent/100 ft))	1.88	1.87	0.00	0.50	1.43	2.54	8.13
PTRUCK (percent)	9.0	6.7	2.7	5.0	7.7	11.2	45.4
ND_1	3.00	3.00	0.00	0.00	3.00	4.00	15.00
PROTLT[a]	—	—	—	—	—	—	—

Conversion: 1 ft = 0.305 m

[a] 43 percent of the four-leg signalized intersections have protected left-turn signal phases and 57 percent do not.

Negative binomial models were developed for predicting accident experience at four-leg signalized intersections on rural two-lane highways using the modeling technique based on the investigating officer's identification of intersection-related accidents. A candidate model developed using this approach is:

$$
\begin{aligned}
N_{bi} = \exp(&-6.12 + 0.46\,SUMLADT - 0.61\,PROTLT - 0.013\,PCTLEFT_2 \\
&+ 0.12\,VEICOM + 0.030\,PTRUCK)
\end{aligned}
\qquad \textbf{(63)}
$$

where:

SUMLADT = *ln* ADT_1 + *ln* ADT_2;

PROTLT = presence of protected left-turn signal phase on one or more major-road approaches; = 1 if present; 0 = if not present;

$PCTLEFT_2$ = percentage of minor-road left-turning traffic at the signal for the morning and evening peak hours combined;

VEICOM	=	grade rate for all vertical curves (crests and sags) any portion of which is within 244 m (800 ft) of the intersection averaged for the major- and minor- road legs of the intersection;
PTRUCK	=	percentage of trucks (vehicles with more than four wheels) entering the intersection for morning and evening peak hours combined;

The variable PROTLT indicates the presence of either a fully protected left-turn signal phase or a protected-permitted phase.

An alternative to equation (63) using the major- and minor-road traffic volumes separately:

$$N_{bi} = \exp(-6.95 + 0.62\,ln\ ADT_1 + 0.39\,ln\ ADT_2 - 0.68\,PROTLT - 0.014\,PCTLEFT_2 + 0.13\,VEICOM + 0.032\,PTRUCK) \quad \textbf{(64)}$$

Tables 45 and 46 summarize the model parameters and goodness of fit for equations (63) and (64), respectively.

Negative binomial models were developed for predicting accident experience at four-leg signalized intersections on rural two-lane highways using the modeling technique based on the accident types generally considered to be intersection-related. A model using the sum of the major- and minor-road traffic volume variables is:

$$N_{bi} = \exp(-4.96 + 0.39\,SUMLADT - 0.38\,PROTLT - 0.015\,PCTLEFT_2 + 0.103\,VEICOM + 0.027\,PTRUCK) \quad \textbf{(65)}$$

An alternative form of equation (65) using separate variables for the major- and minor-road traffic volumes is:

$$N_{bi} = \exp(-6.084 + 0.60\,ln\ ADT_1 + 0.29\,ln\ ADT_2 - 0.47\,PROTLT - 0.017\,PCTLEFT_2 + 0.11\,VEICOM + 0.029\,PTRUCK) \quad \textbf{(66)}$$

Negative binomial models comparable to equations (65) and (66) including an independent variable representing the number of driveways within 76 m (250 ft) of the intersection were also developed. A model using the sum of the major- and minor-road traffic volume variables is:

$$N_{bi} = \exp(-4.11 + 0.33\,\mathrm{SUMLADT} - 0.30\,\mathrm{PROTLT} - 0.016\,\mathrm{PCTLEFT}_2 + 0.100\,\mathrm{VEICOM} + 0.023\,\mathrm{PTRUCK} + 0.035\,\mathrm{ND}_1) \quad \mathbf{(67)}$$

Table 45. Model Parameters and Goodness of Fit for Equation (63).

Parameter	Independent variable: Intercept	SUMLADT	PROTLT	$PCTLEFT_2$	VEICOM	PTRUCK	Over-dispersion parameter (k)
Coefficient	!6.12	0.46	!0.611	!0.013	0.12	0.030	0.12
Standard deviation	2.60	0.15	0.151	0.0048	0.051	0.014	0.032
Significance level (p)	0.018	0.0017	0.0001	.0052	0.014	0.033	0.0002

Note: The values of goodness-of-fit measures are $R^2 = 0.5208$, $R_k^2 = 0.6414$, and $R_{PD}^2 = 0.2550$

Table 46. Model Parameters and Goodness of Fit for Equation (64).

Parameter	Independent variable: Intercept	*ln* ADT_1	*ln* ADT_2	PROTLT	$PCTLEFT_2$	VEICOM	PTRUCK	Over-dispersion parameter (k)
Coefficient	!6.95	0.62	0.39	!0.68	!0.014	0.13	0.032	0.12
Standard deviation	2.79	0.25	0.17	0.18	0.0047	0.045	0.014	0.032
Significance level	0.013	0.013	0.023	0.0002	.0023	0.0039	0.028	0.0003

Note: The values of goodness-of-fit measures are $R^2 = 0.5053$, $R_k^2 = 0.6490$, and $R_{PD}^2 = 0.2362$

where:

ND_1 = number of driveways with 76 m (250 ft) of the intersection on the major road.

An alternate form of equation (67) using separate variables for the major- and minor-road traffic volumes is:

$$\begin{aligned} N_{bi} = \exp(&-5.46 + 0.60\,ln\ ADT_1 + 0.20\,ln\ ADT_2 \\ &- 0.40\,PROTLT - 0.018\,PCTLEFT_2 + 0.11\,VEICOM \\ &+ 0.026\,PTRUCK + 0.041\,ND_1) \end{aligned} \qquad \textbf{(68)}$$

Tables 47 through 50 summarize the model parameters and goodness of fit for equations (65) through (68), respectively.

A negative binomial model for predicting accident experience at four-leg signalized intersections on rural two-lane highways was also developed using the iterative offset technique. This model uses a different functional form for the major and minor road ADT variables than was used in the preceding models. This model is:

$$N_{bi} = ADT_1^{0.307}\ \exp(0.0000376\ ADT_1)\ ADT_2^{0.461}\ LTLN_1\ RTLN_1 \qquad \textbf{(69)}$$

where:

$LTLN_1$ = factor for number of major-road left-turn lanes present at the intersection ($LTLN_1$ = 1.000 for zero left-turn lanes, 0.934 for one left-turn lane, and 0.737 for two left-turn lanes.); and

$RTLN_1$ = factor for number of major-road right-turn lanes present at the intersection ($RTLN_1$ = 1.000 for zero right-turn lanes, 0.832 for one right-turn lane, and 0.871 for two right-turn lanes).

The value of the overdispersion parameter for this model is 0.26.

Table 47. Model Parameters and Goodness of Fit for Equation (65).

Parameter	Independent variable: Intercept	SUMLADT	PROTLT	$PCTLEFT_2$	VEICOM	PTRUCK	Over-dispersion parameter (k)
Coefficient	−4.96	0.39	−0.38	−0.015	0.103	0.027	0.14
Standard deviation	3.078	0.18	0.17	0.006	0.042	0.013	0.039
Significance level (p)	0.107	0.0309	0.022	0.0101	0.013	0.040	0.0005

Note: The values of goodness-of-fit measures are $R^2 = 0.3913$, $R_k^2 = 0.5382$, and $R_{PD}^2 = 0.1770$

Table 48. Model Parameters and Goodness of Fit for Equation (66).

Parameter	Independent variable: Intercept	*ln* ADT_1	*ln* ADT_2	PROTLT	$PCTLEFT_2$	VEICOM	PTRUCK	Over-dispersion parameter (k)
Coefficient	−6.084	0.60	0.29	−0.47	−0.017	0.113	0.029	0.13
Standard deviation	3.39	0.28	0.20	0.20	0.0057	0.037	0.013	0.039
Significance level	0.072	0.037	0.14	0.019	.0036	0.002	0.028	0.0008

Note: The values of goodness-of-fit measures are $R^2 = 0.3650$, $R_k^2 = 0.5521$, and $R_{PD}^2 = 0.1816$

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Table 49. Model Parameters and Goodness of Fit for Equation (67).

Parameter	Intercept	SUMLADT	PROTLT	$PCTLEFT_2$	VEICOM	PTRUCK	ND_1	Over-dispersion parameter (k)
	Independent variable							
Coefficient	!4.11	0.332	!0.302	!0.016	0.100	0.023	0.035	0.12
Standard deviation	2.95	0.172	0.175	0.0055	0.038	0.012	0.027	0.037
Significance level (p)	0.163	0.053	0.083	0.0038	0.009	0.055	0.199	0.0011

Note: The values of goodness-of-fit measures are $R^2 = 0.4563$, $R_k^2 = 0.2834$, and $R_{PD}^2 = 0.1822$

Table 50. Model Parameters and Goodness of Fit for Equation (68).

Parameter	Intercept	*ln* ADT_1	*ln* ADT_2	PROTLT	$PCTLEFT_2$	VEICOM	PTRUCK	ND_1	Over-dispersion parameter (k)
	Independent variable								
Coefficient	-5.46	0.599	0.202	-0.404	-0.018	0.111	0.026	0.098	0.11
Standard	3.19	0.279	0.192	0.188	0.005	0.033	0.012	0.041	0.040
Significance level	0.087	0.032	0.293	0.032	0.0005	0.0006	0.028	0.025	0.0043

Note: The values of goodness-of-fit measures are $R^2 = 0.4327$, $R_k^2 = 0.6094$, and $R_{PD}^2 = 0.1816$

Assessment of Alternative Base Models

A careful examination and comparison was made of the alternative base models presented above to select the most suitable models for use in the accident prediction algorithm. The final selections were made based on a sensitivity analysis and an assessment of the combined roadway segment and intersection models to predict total accidents within 76 m (250 ft) of the set of study intersections as a whole, including both intersection-related and non-intersection-related accidents.

Equation (42) was the only candidate roadway segment model and that model was found to be satisfactory for use as the base model for roadway segments. This model also appears as equation (5) in the main text of the report.

Six candidate models were developed for three-leg STOP-controlled intersections. Of these six models, the models developed using the officer's assessment of intersection-related accidents were found, together with the roadway segment model, to provide the best estimate of total accidents with in 76 m (250 ft) of the 382 study intersections. In choosing between equations (52) and (53), preference was given to equation (53) which includes only those variables found to be statistically significant with a significance level (p) of 0.015 or less. Therefore, equation (53) was selected as the base model for three-leg STOP-controlled intersections and it also appears as equation (7) in the main text of this report.

Similar logic was used to select equation (59) as the base model for four-leg STOP-controlled intersections. Equation (59) also appears as equation (9) in the main text of this report.

The assessment of the candidate models for four-leg signalized intersections was difficult because all seven candidate models were based on a relatively small sample size (49 intersections) and none of the models was found to be completely satisfactory. Equation (69) was appealing because it contains the same independent variables for which AMFs have been provided in the accident prediction algorithm. However, the functional form selected for equation (69) is such that the predicted accident frequency begins to decrease when the major-road ADT exceeds approximately 10,000 veh/day. This does not appear reasonable, so equation (69) was eliminated from consideration. The best of the remaining candidate models was equation (68), which also appears in the main text of the report as equation (11).

Another Alternative Approach for Future Consideration

The accident prediction algorithm for two-lane highways is based on the concept that accidents in the vicinity of an intersection can be predicted by separate models for intersection-related accidents and roadway-segment-related (or non-intersection-related) accidents. The models used to predict roadway segment and intersection accidents are independent; this is based on the assumption that there is a certain "background" level of roadway segment accidents that would occur on the major-road legs of the intersection whether the intersection was present or not. The accident prediction algorithm does not consider the possibility that the presence of the intersection could affect the frequency, severity, or collision types for accidents that are classified as non-intersection-related.

An alternative concept for considering the interaction described above was proposed during the development of the accident prediction algorithm. While this approach was, in the end, judged to have sufficient conceptual problems to be unworkable, this alternative approach is documented here for future reference in case a workable variation can be developed in the future.

The proposed concept was that a 152 m (500 ft) roadway segment along the major road (76 m or 250 ft on either side of the intersection) operates as an intersection approach during the time when traffic is present on the minor-road approach and operates as a roadway segment the rest of the time. Thus, if one could estimate the proportions of time when the segment operated as a roadway segment and as an intersection, its safety performance could be predicted as:

$$N_t = (1-p)\ N_{rs} + (p)\ N_{int} \tag{70}$$

where:

N_t = Total predicted accident frequency for a 152 m (500 ft) roadway segment containing an intersection; and

p = proportion of time during which the roadway segment operates as an intersection ($0 \le p \le 1$).

Values for N_{rs} and N_{int} would could be determined from the accident prediction algorithm as shown in Equations (13) and (14). Furthermore, it was proposed that the proportion p could be estimated as follows:

$$p = \frac{t_{minor} ADT_2}{86{,}400},\ p \le 1.0 \tag{71}$$

where:

t_{minor} = average time each minor-road vehicle is present at or near the intersection (sec/veh); and

ADT_2 = average daily traffic volume on the minor road (veh/day).

The average time each minor-road vehicle is present at or near the intersection, t_{minor}, could be estimated as some representatives value, such as 10 to 20 sec. The constant value of 86,400 in the denominator of equation (66) represents the number of seconds in a 24-hour day.

The approach represented by equations (70) and (71) has substantial conceptual appeal because it incorporates a direct proportionality to the minor-road ADT. The approach could be strengthened conceptually by accounting for traffic turning from the major road onto the minor road and by adjusting for time periods when more than one minor-road vehicle is present at the intersection at the same time. Furthermore, the concept might also be adapted to the prediction of driveway accidents by assuming very low values of ADT_2 for driveways. This could allow the accident prediction algorithm to address driveway accidents much more explicitly than is permitted by the driveway density AMF shown in equation (22).

The approach represented by equations (70) and (71) has conceptual disadvantages as well. First, it is based on a presumption that when a minor-road vehicle is present at the intersection there is no risk of an accident occurring elsewhere on the adjacent 152 m (500 ft) roadway segment. This presumption appears unrealistic. There must be some risk of an accident elsewhere on the major road when a minor-road vehicle is present at the intersection. Indeed, there is simply no evidence as to whether the presence of a minor-road vehicle at an intersection affects the risk of accidents elsewhere on the major road at all. Second, the form of equation (71) is such that, depending on the value of t_{minor} that is selected, there is some level at which p becomes equal to 1.0. At and above that ADT level, equation (70) implies that there would be no roadway-section-related accidents within 152 m (500 ft) of the intersection. Such a proposition is also unrealistic.

On balance, this approach to intersection accident prediction discussed above was found to be unworkable and was abandoned. The concept has been presented here to document that it was considered and to suggest an alternative that might be adapted for use in the future.

APPENDIX C

CALIBRATION PROCEDURE TO ADAPT THE ACCIDENT PREDICTION ALGORITHM TO THE DATA OF A PARTICULAR HIGHWAY AGENCY

This appendix presents a calibration procedure for use with the accident prediction algorithm. The purpose of the calibration procedure is to allow potential users of the algorithm to scale the accident predictions to be suitable for the roads under the jurisdiction of their agency.

The base models that form a key element of the accident prediction algorithm are based on data from several States. The roadway section base models were based on data from Minnesota and Washington, but are implemented in the accident prediction algorithm for Minnesota conditions (i.e., STATE=0 in Equation (42). The base models for STOP-controlled intersections are based on Minnesota data, and the base models for signalized intersections are based on combined data from California and Michigan. The development of these base models has been documented in appendix B of this report.

The calibration procedure provides a method for highway agencies other than those identified above to adapt the accident prediction algorithm to their own safety conditions. Because safety conditions change over time, even a state whose data were used in the development of the base models should consider applying the calibration procedure every 2 or 3 years.

The accident prediction algorithm applies to rural two-lane highways. Safety conditions and the resulting accident rates and distributions of accident severities and accident types, on rural two-lane highways, vary substantially from one highway agency to another. Some of these variations are due to geometric design factors, such as differences in the distributions of lane and shoulder width, and differences in terrain, which lead to differences in horizontal and vertical alignment. These geometric design factors are accounted for by the accident prediction algorithm and should not require calibration to allow the algorithm to be used by different agencies.

By contrast, there are several factors that lead to safety differences between highway agencies in different geographical areas that are not directly accounted for by the accident prediction algorithm. These include:

- Differences in climate (i.e., exposure to wet pavement and snow-and-ice-covered pavement conditions).

- Differences in animal populations that lead to higher frequencies of collision with animals in some States than in others.

- Differences in driver populations and trip purposes (i.e., commuter vs. commercial travel vs. recreational travel).

- Accident reporting thresholds established by State law (i.e., minimum property damage threshold that requires reporting of an accident).

- Accident investigation practices (i.e., some police agencies are much more diligent about investigating and reporting property-damage-only collisions than others).

The calibration procedure is intended to account for these differences and provide accident predictions that are comparable to the estimates that a highway agency would obtain from its own accident records system.

The tables for the calibration procedure are presented in this appendix using conventional units, because all of the underlying equations and data are in conventional units. A metric conversion chart is included for the convenience of those who wish to implement the calibration procedure in metric units.

How Does the Calibration Procedure Affect the Accident Predictions?

The calibration procedure is implemented by a highway agency by determining the value of calibration factors for roadway sections and at-grade intersections from comparison of their own data to estimates from the accident prediction algorithm. As shown in section 3 of this report, the calibration factors are incorporated in the accident prediction algorithm in the following fashion for roadway segments and at-grade intersections, respectively:

$$N_{rs} = N_{br} \, C_r \, (AMF_{1r} \, AMF_{2r} \, ... \, AMF_{nr}) \qquad \textbf{(72)}$$

$$N_{int} = N_{bi} \, C_i \, (AMF_{1i} \, AMF_{2i} \, ... \, AMF_{ni}) \qquad \textbf{(73)}$$

where:

C_r = calibration factor for roadway sections developed for use by a particular highway agency; and
C_i = calibration factor for at-grade intersections developed for use by a particular highway agency.

The procedures for estimating values of C_r and C_i for a particular highway agency are described later in this section.

The calibration procedure also permits a highway agency to modify the basic accident severity distribution for rural two-lane highways presented in table 1 and the basic accident type distribution for rural two-lane highways presented in table 2 based on their own data.

Who Should Perform the Calibration Procedure?

It is recommended that one division or office within a specific State highway agency should calibrate the accident prediction algorithm for subsequent use by all other highway agency employees and their contractors throughout the State. Because the calibration procedure requires the use of historical accident, roadway, and traffic data for State highways, it is further recommended that the calibration be performed by a division or office of the highway agency that has the following:

- Unrestricted access to the State traffic, accident, and roadway record system(s).
- Capability to generate estimates of roadway mileage and accident experience for various ADT levels.
- Expertise in highway safety analysis procedures.

It is not necessary to calibrate these models to different regions (or districts) within the State, although that certainly can be done if the highway agency wishes by stratifying mileage and accidents by region (or district) and applying the recommended calibration procedures for each region (or district).

Furthermore, since the accident analysis module is only applicable to rural two-lane highways at this time, the models currently cannot be calibrated for different urban areas within the State. As the models for multilane and urban roads are developed, calibration procedures that apply to individual urban areas may also be desirable.

Should the Accident Prediction Algorithm Be Used Without Calibration?

It is possible for a highway agency to use the accident prediction algorithm without calibration, but this is not recommended. Using the accident prediction algorithm without calibration requires the user to accept the assumption that for their agency $C_r = 1.0$, $C_i = 1.0$, and the accident severity and accident type distributions for two-lane highways are those shown in tables 1 and 2, respectively. These assumptions are unlikely to be correct for any highway agency and are unlikely to remain correct over time. Even a minimal calibration effort (referred to later in this appendix as level 1 calibration) is likely to produce far more satisfactory results than using the algorithm without calibration.

How Often Should the Accident Prediction Algorithm Be Calibrated?

The recommended calibration procedure uses the three most recent years of accident data for the highway agency's rural two-lane highway system. Recalibration every year is not necessary because it is unlikely that adding a new year of data and dropping the oldest of the three years used for calibration in the previous year will change the calibration factors substantially. Recalibration every 2 or 3 years is recommended instead.

Recommended Calibration Procedures

It is neither necessary nor recommended for users of the accident prediction algorithm to generate new accident prediction models using accident, roadway, and traffic data from their State and appropriate statistical analysis procedures. It is definitely not recommended for users to change the accident modification factors in the accident prediction algorithm. These AMFs are based on a compilation of the best available research, and they will be updated by FHWA as new research becomes available. Rather, the recommended calibration procedure estimates the values of calibration factors that adapt the outputs from the accident prediction algorithm to the safety conditions experienced by an individual highway agency.

Calibrating the Roadway Segment Accident Prediction Algorithm

Optional levels are available to calibrate the roadway segment accident prediction algorithm. The levels vary in terms of the effort required and the availability of the following:

- Type of data elements maintained within the existing traffic records systems.
- Existence, quality and coverage of roadway, and traffic files.
- Availability and quality of accident data.

- Skills of personnel who will perform the calibration.

- Level of effort and personnel resources that a State is willing to commit to calibration.

The minimum data requirements and anticipated effort for each calibration process level are specified in table 51. Level 1 is deemed the minimum and, compared to the other level, the easier calibration to perform. Level 2 requires more effort, but with a corresponding gain in applicability of the accident prediction algorithm to a particular State's accident experience. It is strongly recommended that the models NOT be used without calibration. Attempting to calibrate using procedures less stringent than the level 1 procedure is strongly discouraged.

Table 51. Minimum Requirements for Calibration Levels 1 and 2

Calibration process*	Minimum requirements	Anticipated effort for calibration
Level 1	The State must have the ability to: (1) Stratify all two-lane rural roads by ADT; and (2) Identify all non-intersection related accidents reported on those two lane rural roads.	Minimal
Level 2	Level 1 requirements + the State must have the ability to: (3) Stratify all two-lane rural roads by ADT, shoulder width and lane width.	Moderate

* to be selected by the State.

While table 51 describes the minimum data requirements for each calibration level and shows how the calibration levels relate to each other, table 52 identifies the data elements that must be present within the State's record systems to complete the calibration level. The procedures to perform levels 1 and 2 are described on the following pages.

The basic procedure to be followed in each level involves six steps shown in figure 7. The only difference in the two levels is that, in level 2, step 1 will develop estimates of miles of roadway by lane and shoulder width, as well as by curve and grade.

Road System to be Used in Calibration Process

It is important to recognize that there are differences in State highway agency record systems. Some States, such as California, include only roads under State jurisdiction in their record systems. Others, such as Minnesota, include local roads that are maintained by counties and municipalities. Calibration can be performed using whatever rural two-lane

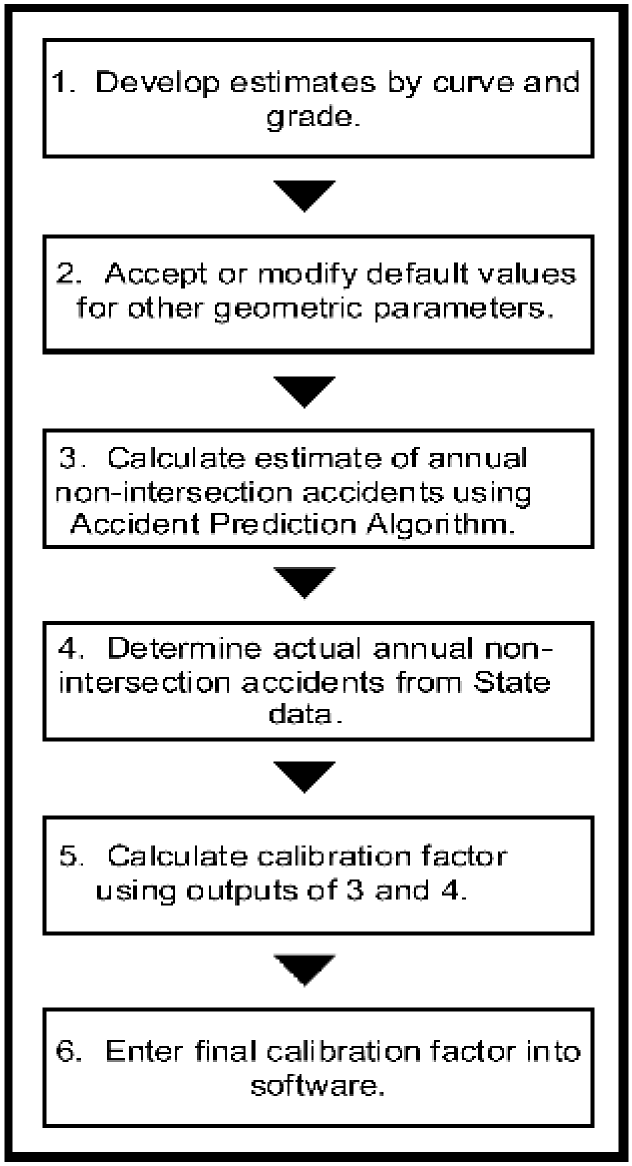

Figure 7. Flow Diagram of Calibration Process.

Table 52. Data Needs for Calibration Levels 1 and 2.

	Type of data	Data element	Level 1	Level 2A
Required	Accident records	Relationship of accidents to intersections or junctions	x	x
	Traffic volume files	ADT	x	x
	Roadway inventory files	Lane width Shoulder width		x x
Desirable	Alignment inventory files	Horizontal curve data; Grade and vertical curve data	x x	x x
	Access point	Driveway	x	x

road system the user wishes to apply the accident prediction algorithm to as long as the data are accurate. Specifically, the following should exist:

- A reliable inventory of paved roads.
- Reliable counts of ADT and accidents that can be assigned to specific sections of a highway.
- Ability to stratify sections of highways into different highway types (i.e., the ability to distinguish rural two-lane highways from their highway types).

It should be noted that the base models used in the accident prediction algorithm were developed for State primary routes that generally had 88-km/h (55-mi/h) speed limits.

If the State has accurate data for county highways and higher order routes but not for municipal highways, then the calibration process should not use municipal or township highways. Rather, data for only the county highways and higher order routes should be used. For example, Utah has data for 20,600 km (12,800 mi) of paved roadways under State jurisdiction and another 51,000 km (32,000 mi) of roadways not under State jurisdiction. A complete roadway inventory to which accidents can be linked is available for the 20,600 km (12,800 mi) system of state roadways, but the other 51,000 km (32,000 mi) have only “zone” records, that do not permit accident records to be linked to specific roadway sections. Consequently, any calibration of the accident prediction model by Utah should use only the 20,600 km (12,800 mi) of roadway under State jurisdiction.

Only paved two-lane highways should be considered in the calibration process.

Level 1 Calibration Process for Roadway Segment Accident Prediction Using ADT Only

Step 1. Develop Estimates of Paved Rural Two-Lane Highway Mileage by Curve and Grade. The State will first need to estimate the following for all paved rural, two-lane highways in the State for each of five ADT groups:

- Number of miles of tangent roadway.
- Number of miles of roadway on horizontal curves.
- Average degree of curvature for horizontal curves.
- Number of miles of level roadway.
- Number of miles of roadway on grade.
- Average grade percent for roadway on grade.

For States with Alignment (Curve and Grade) Inventory Files

If the State has curve and grade inventory files for paved rural two-lane highways, then it will be possible to calculate the necessary alignment data to perform the calibration. Using their horizontal curve inventory data, the State should first calculate the number of miles of tangent roadway, the number of miles of curved roadway, and the average degree of curve for horizontal curves by ADT interval, using the format shown in table 53. Table 53, and subsequent tables in this section, are presented to illustrate the format in which the calibration calculations should be made; the blank spaces in the tables may be filled in by users of the calibration process. Using its vertical alignment (grade) inventory data files, the State should then calculate the number of miles of two-lane rural roads that are not on grade (e.g., level), the number of miles with non-zero grades, an average percent grade for the miles with non-zero grades, and an overall average percent grade as shown in table 54.

Table 53. Estimate Mileage by ADT Level and Horizontal Alignment.

ADT Interval	Number of Tangent Miles	Number of Curved Miles	Average Degree of Curvature for Curved Miles (D)	Average Radius of Horizontal Curve[a]	Average Length of Curve
< 1000					
1,001 - 3,000					
3,001 - 5,000					
5,001 - 10,000					
> 10,000					

[a] Calculated as 5729.58 / (D).

Table 54. Estimate Mileage by ADT Level and Vertical Alignment.

ADT Interval	Number of Level Miles (M_l)	Number of Miles on Grade (M_g)	Average Percent Grade for Miles on Grade (P_g)	Average Percent Grade[a]
< 1000				
1,001 - 3,000				
3,001 - 5,000				
5,001 - 10,000				
> 10,000				

[a] Calculated as $[(M_l) * (0) + (M_g) * (P_g)] / [M_l + M_g]$.

For States without Alignment (Curve and Grade) Inventory Files

For States without curve or grade files, they can use an estimation procedure that will calculate “default” values for curve and grade mileages and average degree of curve and percent grade values based on the percent of rural two-lane miles that fall into each of the three terrain groups—flat, rolling, and mountainous. Thus, the State needs to estimate the mileage by ADT interval and the percentage of mileage in flat, rolling, and mountainous terrains, which could be derived from Highway Performance Monitoring System (HPMS) data. Then, the State will need to determine the mileages by ADT intervals and the percentages of level, rolling, and mountainous terrain.

Default values for percent of tangent and curved miles and the average degree of curve for the curved miles were based on analyses of States in the HSIS database which had both a curve file and terrain information. Based on these analyses, default values for proportion of two-lane rural miles that are curved and the average degree of curve for these miles are as follows:

- Flat: percent of non-tangent miles = 19 percent; average degree of curvature= 2E.
- Rolling: percent of non-tangent miles = 24 percent; average degree of curvature = 4E.
- Mountainous: percent of non-tangent miles = 38 percent; average degree of curvature = 8E.

Similarly, information on default values for percent of non-flat miles and average grade for non-flat mileage from HSIS States were used to calculate the following default values:

- Flat: percent of non-flat miles = 87 percent; average grade = 1.5 percent.
- Rolling: percent of non-flat miles = 91 percent; average grade = 2.0 percent.
- Mountainous: percent of non-flat miles = 97 percent; average grade = 3.7 percent.

Tables 55 and 56 illustrate the data that are required.

Table 55. Estimate Mileage by ADT Interval.

ADT Interval	Estimated miles of paved, two-lane rural highways
< 1,000	
1,000 - 3,000	
3,001 - 5,000	
5,001-10,000	
> 10,000	

Table 56. Estimate Proportion of Mileage by Terrain.

Terrain	Proportion of paved, two-lane rural highways (percentage)
Flat	
Rolling	
Mountainous	
Total	100%

These values in tables 55 and 56 will then be used by the calibration software to produce estimates of the following:

- Number of miles of tangent roadway.
- Number of miles of roadway on horizontal curves.
- Average degree of curvature for horizontal curves.
- Average radius of horizontal curves.
- Average length of horizontal curves.
- Number of miles if level roadway.
- Number of miles of roadway on grade.
- Average percent grade for roadway on grade.
- Statewide average percent grade.

Step 2. Accept or Modify Default Values for Other Geometric Parameters. In addition to the values of curvature and grade developed in the proceeding step, the calibration procedure requires other geometric parameters. Using data from a sample of States during the calibration procedure development, it has been found reasonable to use the following default values for these parameters:

- Shoulder type = paved.

- Driveway density = three driveways/km (five driveways/mi).
- Passing lane = not present.
- Short four-lane section = not present.
- Two-way left turn lanes = not present.
- Roadside hazard rating = 3.
- For horizontal curves:
  no spiral transition present; and
  superelevation is not deficient (e.g., AMF = 1.0).

For level 1 calibration (which does not require data to be input for lane width and shoulder width), the following default values were estimated based on average values for two-lane highways obtained from eight HSIS States and are recommended for use:

	DEFAULT VALUES FOR LEVEL 1 CALIBRATION		
ADT Interval (Vehicles/day)	ADT (vehicles/day)	Lane Width (ft)	Shoulder Width (ft)
< 1000	400	11	3.5
1,000 - 3,000	1800	11.5	5
3,001 - 5,000	3900	12	5
5,001 - 10,000	6900	12	6.25
> 10,000	13800	12	6.5

The State may accept these defaults as presented and proceed onward. However, if a State has a better estimate for a value [e.g., if average driveway density is known to be equal to four driveways/km (seven driveways/mi), rather than three driveways/km (five driveways/mi)], or if the predominant two-lane roadway design used in a particular State differs from these assumed defaults (e.g., if the predominant rural two-lane highway shoulder type is unpaved, rather than paved), then the user should feel free to modify the defaults.

Step 3. Calculate Estimate of Annual Non-Intersection Accidents Using the Accident Prediction Algorithm. Using the roadway segment accident prediction algorithm, calculate the estimated annual number of non-intersection accidents for tangents and horizontal curves. Then, sum the total. Table 57 illustrates how the predicted number of accidents per year can be determined by the calibration software.

Table 57. Calculate the Predicted Annual Number of Non-Intersection Accidents as a Function of ADT.

ADT (veh/day)	Assumed mean of ADT interval*	Mileage of rural two lane highways			Predicted number of non-intersection accident per year**		
		Tangent	Curve	Total	Tangent	Curve	Total
< 1,000	400						
1,000-3,000	1,800						
3,001-5,000	3,800						
5,001-10,000	7,000						
>10,000	13,500						

* Generally the average is less than the midpoint of the ADT interval.

** Determined with the uncalibrated accident prediction algorithm for roadway segments.

Step 4. Determine Actual Annual Non-Intersection Accidents from State Data. Using data from the last 3 years, determine the actual number of non-intersection accidents per year that were reported on the rural two-lane highways. First, determine the number of total accidents on the selected rural two-lane highways. Then, deduct all accidents on those rural two-lane highways that were identified by the investigating officer as being "at intersection" or "intersection-related."

With respect to this criterion, the State must determine the most appropriate field(s) to establish intersection-relatedness. for example, a State may have a field for "type of event location" or "site location" or "relationship to junction" on its accident report form that includes categories such as intersection, junction area, non-junction area, driveway access, and alley access. Some States have an explicit field for "relation to intersection" with categories for yes or no. The decision on which field to use is left to the user. It is important to note that driveway accidents are NOT considered intersection accidents and should NOT be excluded from the calibration data for roadway segments. Driveway accidents were included in the data set from which the roadway segment base model was developed and, therefore, driveway accidents should not be excluded with intersection accidents.

Step 5. Calculate Calibration Factor Using Outputs of Steps 3 and 4. Calculate the calibration factor (C_r) as the ratio of the total number of reported non-intersection accidents (from step 3) to the total number of predicted non-intersection accidents (from step 2).

Step 6. Enter Final Calibration Factor into Software. The preceding five steps will produce a calibration factor based on the user's inputs. At this point, the user will usually accept the calibration factor as calculated. However, if the resulting factor appears unreasonable (perhaps because of data problems of which the user is aware), it may be modified before entry into the system. Once accepted by the user, the value of the calibration factor (C_r) should be entered into a file of default input values for the accident prediction algorithm. In subsequent applications of the accident prediction algorithm, this value of C_r will be used in applying equation (72). As discussed earlier, the user can, and should, update the calibration factor every 2 or 3 years.

Level 2 Calibration Process for Roadway Segment Accident Prediction Using ADT, Lane Width, and Shoulder Width

Step 1. Develop Estimates of Mileage by Curve and Grade and Lane and Shoulder Width. The State will first need to stratify the mileage of their paved rural, two-lane highways in the State by the following factors and associated levels:

- ADT
  - < 1000 veh/day
  - 1,001 - 3,000 veh/day
  - 3,001 - 5,000 veh/day
  - 5,001 - 10,000 veh/day
  - >10,000 veh/day

- Lane Width
  - < 2.9 m (9.5 ft)
  - 9.5-10.5 2.9-3.2 m (9.5-10.5 ft)
  - 3.2-3.5 m (10.5-11.5 ft)
  - > 3.5 m (11.5 ft)

- Shoulder Width
  - < 0.9 m (3 ft)
  - 0.9-1.5 m (3-5 ft)
  - 1.5-2.1 m (5-7 ft)
  - > 2.1 m (7 ft)

- Horizontal Alignment
  - tangent
  - curve

For States with Alignment (Curve and Grade) Inventory Files

If the State has curve and grade inventory files, then it will be possible to calculate the necessary alignment data to perform the calibration. Using their horizontal curve inventory data, they should first estimate the number of miles of tangent roadway, the number of miles of curved roadway and the average degree of curve for horizontal curves for each unique combination of

ADT interval, lane width and shoulder width. Table 58 illustrates this for a portion of all possible combinations.

Using their vertical alignment (grade) inventory data files, the State should then calculate the number of miles of two-lane rural roads that are not on grade (e.g., level), the number of miles with non-zero grades, an average percent grade for the miles with non-zero grades, and an overall average percent grade as shown in table 59.

For simplification purposes, it can be assumed that the average percent grade computed for an ADT interval is equally applicable across lane widths and shoulder widths within that interval. By way of an example, consider the case where 1.9 percent is computed as the average percent grade for all two-lane rural roads within the <1000 ADT interval. The 1.9 percent grade can be assumed for all lane and shoulder width combinations having ADTs less than 1,000 veh/day, as illustrated in table 60.

For States without Alignment (Curve and Grade) Inventory Files

For States without curve or grade files, they can use an estimation procedure that will calculate “default” values for curve and grade mileages and average degree of curve and percent grade values based on the percent of rural two-lane miles that fall into each of the three terrain groups—flat, rolling, and mountainous. Thus, the State needs to estimate the mileage by ADT interval, lane width and shoulder width, and the percentage of mileage in flat, rolling, and mountainous terrains, which could be derived from HPMS data. Then, the State will need to enter the mileages by ADT intervals and the flat, rolling, and mountainous percentages into the calibration software. Tables 61 and 62 illustrate the data that are required.

Default values for percent of tangent and curved miles and the average degree of curve for the curved miles were based on analyses of States in the HSIS database which had both a curve file and terrain information. Based on these analyses, default values for proportion of two-lane rural miles that are curved and the average degree of curve for these miles are as follows:

- Flat: percent of non-tangent miles = 19 percent; average degree of curvature = 2E.
- Rolling: percent of non-tangent miles = 24 percent; average degree of curvature = 4E.
- Mountainous: percent of non-tangent miles = 38 percent; average degree of curvature = 8E.

Table 58. Develop Estimates Required for Alignment Component of the Procedure.

ADT interval	Lane width (ft)	Shoulder width (ft)	Number of miles on tangent	Number of miles on horizontal curves	Average degree of curvature (D)	Average radius of horizontal curve[a] (ft)	Average length of curve (mi)
< 1000	< 9.5	< 3					
		3 to 5					
		5 to 7					
		> 7					
	9.5 to 10.5	< 3					
		3 to 5					
		5 to 7					
		> 7					
	10.5 to 11.5	< 3					
		3 to 5					
		5 to 7					
		> 7					
	> 11.5	< 3					
		3 to 5					
		5 to 7					
		> 7					
1,001 to 3,000	< 9.5	< 3					
		3 to 5					
		5 to 7					
		> 7					
	9.5 to 10.5	< 3					
		3 to 5					
		5 to 7					
		> 7					
	10.5 to 11.5	< 3					
		3 to 5					
		5 to 7					
		> 7					
	> 11.5	< 3					
		3 to 5					
		5 to 7					
		> 7					

[a] Calculated as 5729.58 / (D).

Table 59. Estimate Mileage by ADT Level and Vertical Alignment.

ADT interval	Number of miles of level roadway (M_l)	Number of miles on grade (M_g)	Average percent grade for miles of roadway on grade (P_g)	Average percent grade[a]
< 1000				
1,001-3,000				
3,001-5,000				
5,001-10,000				
> 10,000				

[a] Calculated as $[(M_l) * (0) + (M_g) * (P_g)] / [M_l + M_g]$.

Table 60. Illustration of How Average Percent Grade Can Be Applied Across Lane and Shoulder Width Combinations.

AADT (Vehicles per day)	Average Percent Grade (%)															
	Lane Width															
	< 9.5 ft				9.5 to 10.5 ft				10.5 to 11.5 ft				> 11.5 ft			
	Shoulder Width (ft)															
	< 3	3-5	5-7	> 7	< 3	3-5	5-7	> 7	< 3	3-5	5-7	> 7	< 3	3-5	5-7	> 7
< 1,000	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9

Table 61. Estimate Proportion of Mileage by Terrain.

Terrain	Proportion of paved, two-lane rural highways (percentage)
Flat	
Rolling	
Mountainous	
Total	100%

Table 62. Estimate Mileage by ADT Interval, Lane and Shoulder Widths.

AADT (Vehicles per day)	Mileage of paved, two-lane rural roads															
	Lane Width <9.5 ft				9.5<Lane Width<10.5ft				10.5<LaneWidth<11.5 ft				Lane Width > 11.5 ft			
	Shoulder Width (ft)															
	<3	3-5	5-7	>7	<3	3-5	5-7	>7	<3	3-5	5-7	>7	<3	3-5	5-7	>7
< 1,000																
1,001 - 3,000																
3,001 - 5,000																
5,001 - 10,000																
> 10,000																

1 ft = 0.305 m

Similarly, information on default values for percent of non-flat miles and average grade for non-flat mileage from HSIS States were used to calculate the following default values:

- Flat: percent of non-flat miles = 87 percent; average grade = 1.5 percent.
- Rolling: percent of non-flat miles = 91 percent; average grade = 2.0 percent.
- Mountainous: percent of non-flat miles = 97 percent; average grade = 3.7 percent.

These values in tables 61 and 62 will then be used by the calibration software to produce estimates of the following:

- Number of tangent miles.
- Number of curved miles.
- Average degree of curvature for horizontal curves.
- Average radius of horizontal curve.
- Average length of curve.
- Number of level miles.
- Number of miles on grade.
- Average percent grade for miles on grade.
- Statewide average percent grade.

Step 2. Accept or Modify Default Values for Other Geometric Parameters. Next, the State will need to develop values for other input parameters of the roadway segment accident prediction algorithm. Values for average percent grade, average length of curve, and average radius of curve will have been developed in step 1. For the purposes of calibration, it has been found to be reasonable to assume the following default values:

- Shoulder type = paved.
- Driveway density = three driveways per km (five driveways per mile).
- Passing lane = not present.
- Short four-lane section = not present.
- Two-way left turn lanes = not present.
- Roadside hazard rating = 3.
- For horizontal curves:
  no spiral transition present; and
  superelevation is not deficient (e.g., AMF = 1.0).

Step 3. Calculate Estimate of Annual Non-Intersection Accidents Using the Accident Prediction Algorithm. Calculate the predicted annual number of non-intersection related accidents for tangents and curves using the roadway segment accident prediction algorithm. Then, sum the total. Table 63 illustrates the procedure. the predicted number of total non-intersection accident rates will then be the sum of the number predicted in each line of the table (i.e., sum all calculated values in the final table column for all combinations of ADT, lane width, shoulder width, curve, and grade).

Step 4. Determine Actual Annual Non-Intersection Accidents from State Data. Determine the number of non-intersection accidents on rural two-lane highways using 3 years of accident data. This is the same as step 3 from the level 1 calibration process, which was previously described.

Step 5. Calculate Calibration Factor Using Outputs of Steps 3 and 4. Calculate the calibration factor (C_r) as the ratio total number of reported non-intersection accidents on rural two-lane highways (from step 4) to the predicted total number of non-intersection accidents on rural two-lane highways (from step 3).

Step 6. Enter Calibration Factor into Software. As in level 1, the preceding five steps will produce a calibration factor based on the user's inputs. At this point, the user will usually accept the calibration factor as calculated. However, if the resulting factor appears unreasonable (perhaps because of data problems of which the user is aware), it may be modified before entry into the system. Once accepted by the user, the value of the calibration factor (C_r) should be entered into a file of default input values for the accident prediction algorithm. In subsequent applications of the accident prediction algorithm, this value of C_r will be used in applying equation (72). As discussed earlier, the user can, and should, update the calibration factor every 2 or 3 years.

Table 63. Predicting Total Non-Intersection Accidents as a Function of ADT, Lane Width, and Shoulder Width.

ADT (veh per day)	Lane width (ft)	Shoulder width (ft)	Calculated mean ADT for each combination of ADT, lane width, and shoulder width	Mileage of rural two lane highways			Predicted number of non-intersection accidents per year**		
				Tangent	Curve	Total	Tangent	Curve	Total
< 1,000	< 9.5	< 3							
		3.1-5.0							
		5.1-7.0							
		> 7.0							
	9.5-10.5	< 3.0							
		3.1-5.0							
		5.1-7.0							
		> 7.0							
	10.5-11.5	< 3.0							
		3.1-5.0							
		5.1-7.0							
		> 7.0							
	> 11.5	< 3.0							
		3.1-5.0							
		5.1-7.0							
		> 7.0							
Continue Table for Other AADT Intervals									
Predicted total number of non-intersection accidents =									

1 ft = 0.305 m 1 mi = 1.61 km

* To be calculated by accident prediction algorithm, although the State will have to enter the mean ADT values for each interval.

** Determined with the uncalibrated accident prediction algorithm for roadway segments.

Calibrating the At-Grade Intersection Accident Prediction Algorithm

A calibration procedure for at-grade intersections has also been developed. Several alternative methodologies were developed, applied, and evaluated using California, Minnesota, and Maine data obtained from FHWA's HSIS. The objective was to develop a calibration process that produces intuitively logical, accurate, reliable, and consistent calibration factors. Based on the results of the research, the following calibration process is recommended.

Step 1. Identify Intersection Sites. For each of the following intersection types, identify a sample of intersections for which both intersecting roads are paved rural two-lane highways:

- Three-leg, STOP-controlled intersections.
- Four-leg, STOP-controlled intersections.

- Four-leg, signalized intersections.

Separate values of the calibration factor (C_i) are obtained for each intersection type.

It should be recognized that the current IHSDM Accident Analysis Module does not currently contain prediction algorithms for other types of rural road intersections, such as (1) five-leg intersections; (2) traffic circles/rotaries; (3) three-leg signalized intersections; (4) intersections of two-lane roads with multi-lane roads; (5) uncontrolled or YIELD-controlled intersections; (6) intersections involving one-way streets; or (7) intersections (which are technically considered ramp terminals in the AASHTO Green Book) formed by the junction of freeway ramps and two-lane rural roads. In general, these types of intersections constitute a small percentage of all two-lane rural road intersections. There was insufficient data to develop reliable accident prediction algorithms for these types of intersections. As times goes on, more research may be conducted to develop additional accident prediction algorithms, which can then be integrated into the IHSDM.

The desirable minimum sample sizes are suggested in table 64.

Table 64. Minimum Sample Sizes by Type of Intersection.

Type of Intersection	Suggested Minimum Sample Size
Three-leg STOP-controlled intersections	100
Four-leg STOP-controlled intersections	100
Four-leg signalized intersections	25

The suggested minimum sample sizes are somewhat arbitrary in that they do not have a statistical basis. Clearly, more intersections should produce a more accurate and reliable calibration factor. However, more data collection translates into greater costs to the highway agency. It is believed that the minimum sample sizes presented in table above represent a reasonable compromise between accuracy and cost.

States should identify the sample of their two-lane rural road intersections based on selected characteristics for each of the three intersection types (e.g., three-leg STOP-controlled, four-leg STOP-controlled, and four-leg signalized intersections). The most important characteristic is ADT

It should be understood that there are other parameters used in the accident prediction algorithm such as intersection skew angle, number of major approaches with left-turn lanes, and number of major approaches with right-turn lanes, among others. Moreover, data on several of these variables may even be available within the State's intersection inventory file. However, because the research has found that the ADTs had the strongest predictive relationship with accidents compared to the other variables, it was deemed that the sampling strategy should be based only on the ADTs.

For States with complete and accurate intersection inventory files, it may be possible to automate the sample identification process. For States without intersection inventory files, it will not be possible to sample in proportion to known intersection distributions. Hence, a different sampling procedure applies. The two procedures are discussed separately in the following sections.

Sample Selection for States with Intersection Inventory Files

If a State maintains an accurate and comprehensive intersection inventory file which includes attribute data about the ADTs on the mainline and the cross road, then the following process is applicable.

For all three- and four-leg STOP-controlled intersections, stratify intersections by ADT on the major road. Table 65 illustrates how this can be accomplished for the three- and four-leg STOP-controlled intersections. As shown, the State will calculate the proportions of, say, three-leg STOP-controlled intersections which fall into each major-road ADT group (see “Relative Percentage”). The needed sample of 100 intersections will be divided into groups using these proportions (i.e., see “Desired Sample”).

For four-leg signalized intersections, both the major- and minor-road ADTs have a significant effect on accidents. Hence, for four-leg signalized intersections, sampling should consider both the major-road ADT and the minor-road ADT. Given that there are expected to be relatively few rural signalized intersections in a State, the decision was made to combine major- and minor-road ADTs into a single factor, which can be subsequently used for sampling stratification. The factor is the sum of the average major-road ADT and the average minor-road ADT. Table 66 presents a numerical example of how this might be done for four-leg signalized intersections.

It is important to note that States can do much more with respect to selecting an appropriate sample of intersections. For example, additional factors can be considered and integrated into the sampling stratification strategy. In some States, there are distinct differences in climate, topography, and/or accident reporting practices such that the accident experience in certain parts of the State may significantly differ from other parts of the States. For example, a State’s western portion may be dominated by mountains while the eastern portion is primarily flat plains. In addition, there may be distinct differences in geometric design, maintenance practices, and traffic control as a function of highway district (or region). For example, District A may be “better” with respect to maintenance and traffic control devices than District Z, which may help to explain differences in average accident rates between the two districts. In any event, the State has the ability to

Table 65. Example of Desired Sample Stratification of Three- and Four-Leg STOP-Controlled Intersections Based on Major Road ADT.

		Intersections of two-lane rural roads		
Intersection type	ADT on major road (vehicles/day)	Number	Relative percentage	Desired sample
Three-leg STOP-controlled intersections	< 1,000	1,095	18.3%	18
	1,001 - 3,000	1,443	24.1%	24
	3,001 - 5,000	1,641	27.4%	27
	5,001 - 10,000	887	14.8%	15
	10,001 - 15,000	574	9.6%	10
	> 15,000	347	5.8%	6
	Total	5,987	100.0%	100
Four-leg STOP-controlled intersections	< 1,000	874	21.0%	21
	1,001 - 3,000	777	18.6%	19
	3,001 - 5,000	1,219	29.2%	29
	5,001 - 10,000	489	11.7%	12
	10,001 - 15,000	544	13.0%	13
	> 15,000	267	6.4%	6
	Total	4,170	100.0%	100

Table 66. Example of Desired Sample Stratification of Four-Leg Signalized Intersections Based on Major Road ADT.

		Intersections of two-lane rural roads		
Four-leg, signalized intersections	Sum of major- and minor-road ADTs	Number	Relative percentage	Desired sample
	< 15,000	49	36.0%	9
	$ 15,000	87	64.0%	16
	Total	136	100.0%	25

integrate additional factors such as regional area, district, or terrain into the sampling strategy (provided that they have data for these factors in their intersection inventories).

By way of an example, consider a situation in which the State wants to sample intersections based on ADT and highway district. Table 67 illustrates how highway districts can be considered in the development of a sampling strategy for three-leg STOP-controlled intersections only in a State with 3 districts. The results of this table can be compared with the top portion of table 65 to illustrate how the desired stratification scheme for the sample can be further refined.

Table 67. Example of Desired Sample Stratification of Three-Leg STOP-Controlled Intersections Based on Major-Road ADT and Highway District.

ADT on major roads	State highway district	Intersections of two two-lane rural roads		
		Number	% within State	Desired sample
< 1,000	District 1	620	10.4%	10
	District 2	339	5.7%	6
	District 3	136	2.3%	2
	Subtotal	**1,095**	**18.3%**	**18**
1,001 - 3,000	District 1	435	7.3%	7
	District 2	674	11.3%	11
	District 3	334	5.6%	6
	Subtotal	**1,443**	**24.1%**	**24**
3,001 - 5,000	District 1	592	9.9%	10
	District 2	527	8.8%	9
	District 3	522	8.7%	9
	Subtotal	**1,641**	**27.4%**	**27**
5,001 - 10,000	District 1	363	6.1%	6
	District 2	446	7.4%	7
	District 3	78	1.3%	1
	Subtotal	**887**	**14.8%**	**15**
10,001 - 15,000	District 1	185	3.1%	3
	District 2	334	5.6%	6
	District 3	55	0.9%	1
	Subtotal	**574**	**9.6%**	**10**
> 15,000	District 1	58	1.0%	1
	District 2	289	4.8%	5
	District 3	0	0.0%	0
	Subtotal	**347**	**5.8%**	**6**
Total for entire State		**5,987**		**100**

After the sampling stratification is established, then the specific intersections within each of the cells must be identified. A random selection process should be employed. Within this context, the term random means that all intersection sites of the same type (e.g., either three-leg STOP-controlled intersections, four-leg STOP-controlled intersections, or four-leg signalized intersections) must have the same chance of being selected.

As an example, using the sampling stratification shown in table 65, the names of all three-leg STOP-controlled intersections with major-road ADT less than 1,000 veh/day could be put into a hat and the names of 18 intersections could be drawn at random. The same procedure would be followed for each of the other ADT levels, and then repeated again for four-leg STOP-controlled and signalized intersections. The same "random draw" could be accomplished with a computerized sample by assigning a number to each three-leg STOP-controlled intersection with major-road ADT less than 1,000 veh/day and determining a ratio by dividing the total number of intersections in the file by 18. The first intersection selected would be the intersection whose number is closest to that ratio, the second selected is the intersection whose number is closest to twice that ratio, etc. For example, if there were 90 total intersection from which 18 were to be selected every fifth intersection should be chosen (i.e., 90/18=5).

Sample Selection for States Without Intersection Inventory Files

For States without comprehensive or accurate intersection inventories, information about the existing distribution of intersections within a State may not be known. Hence, the task of identifying existing distributions on which to base the sampling strategy may be limited or even non-existent. Moreover, it may not be possible to identify every intersection of two-lane rural roads, although one expects traffic signal inventory data to be available to do so for signalized intersections.

When a comprehensive list of intersections is not available or cannot be created, then it will be necessary to identify and develop a sample of two-lane rural road segments. [It is assumed that the State has the capability to identify, sort and develop a random sample of all two-lane rural road segments.] Using an approach similar to one previously described for States with intersection inventories, a sample of 100 two-lane rural road segments should be identified based on either (1) ADT alone; or (2) ADT plus additional factors such as highway district that the State deems necessary. (Thus, using the results shown in table 65 for three- and four-leg STOP-controlled intersections in the lowest ADT groups, the State should sample approximately 20 intersections with ADTs less than 1,000 veh/day.)

The data collection teams should be dispatched to those two-lane rural road segments with instructions to collect data at the first three-leg STOP-controlled intersection and the first four-leg STOP-controlled intersection encountered on each segment. In this manner, data can be gathered on a sample of 100 intersections for two of the three intersection types.

As was noted earlier, it is reasonable to assume that traffic signal inventory information, even if in hard copy format, will be available to identify the locations of four-leg signalized intersections at which both the major and minor legs are two-lane rural roads. If the information is not adequate (e.g., cannot determine whether the intersecting roads are two-lane rural roads), then it may be necessary to use the random selection process described for three- and four-leg STOP-controlled intersections above. Based on the intersection inventory data from three HSIS States, there are relatively few four-leg signalized intersections of two-lane rural roads. Thus, the potential exists that the data collection team will have to drive long distances before encountering their first four-leg signalized intersection at which both legs are two-lane rural roads.

Additional Constraints with Respect to Final Selection of the Sample

It is important to recognize that regardless of how the sample is identified, the State should ensure that each selected site experienced no significant changes in geometry, traffic control, roadside improvements, or other factors during the 3-year period corresponding the accident data (which will be discussed in steps 4 and 5). In addition, it will be necessary to exclude intersections that have undergone changes since the end of that 3-year period because measurements of skew angle and intersection sight distance taken today may not be representative of conditions that existed during the 3-year period. By way of an example, assume that accident data from 1995, 1996, and 1997 are going to be used in the calibration. To be included in the sample, the intersection should not have undergone any major changes since January 1, 1995.

If an intersection is known to have had experienced a major change during the study period, then it should be excluded from the sample. Major changes at intersection sites include the following:

- Installation of traffic signal control.
- Widening to provide more approach lanes and/or turn lanes.
- Cchange from a two-way STOP control to an all-way STOP control.
- Changes in intersection geometry (e.g., alignment or cross-section).

Step 2. Collect Data at Selected Intersections. Collect the following information for each intersection in the selected sample:

- Average ADT of major road.
- Average ADT of minor road.
- Intersection skew angle.
- Number of quadrants with deficient sight distance.
- Number of major-road approaches with left turn lanes.
- Number of major-road approaches with right turn lanes.
- Type of traffic control applies (e. g., minor-road or all-way STOP control).

Several of these variables are described below in more detail.

Average ADT of Major Road and **Average ADT of Minor Road** pertain to the bi-directional (i.e., two-way) ADT on the mainline and cross road, respectively. For intersections where the ADT on one leg of the major road differs from the ADT of the other leg of the major road, the average of the two values should be used. Similarly, if the ADT on one leg of a cross road at a four-leg intersection differs from the ADT on the other leg of the crossroad, then the average of the two values should be used.

Most importantly, the ADT should correspond to the accident data. For example, if the calibration is to be performed using accident data from 1995, 1996, and 1997, then the ADTs to be used in the accident prediction algorithm should be the average of the 1995, 1996, and 1997 ADTs for the major and minor roads for the selected intersection. When data for selected years are missing or not available, then the State should use the mid-year if available, or the last year if the mid-year is not available.

The skew angle is described in the main text of this report. The skew angle is the deviation of the intersection angle from the nominal or base angle of 90 degrees (i.e., a right angle). For a four-leg intersection where the angles of the intersection legs to the left and the right of the major road differ, they are averaged. For example, if one leg forms a 60 degree angle and the other intersects at 90 degrees, then the average would be 15 degrees [e.g., *(60-90)* / 2].

Step 3. Execute Accident Prediction Algorithm. The data collected in step 2 will be entered into tables. Data for each intersection within each of the three intersection types will be entered in a row of the table by itself. The accident prediction algorithm will then be used to calculate the predicted number of annual intersection accidents for each intersection. The total number of predicted accidents for each of the three types of intersections will be obtained by summing the calculated predictions for all intersections of that type.

Step 4. Tally Reported Intersection Accidents. Determine the number of intersection-related accidents reported at these intersections over a period of 3 calendar years. An intersection-related accident is an accident that occurs at the intersection itself or an accident that occurs on an intersection approach with 76 m (250 ft) of the intersection and is related to the presence of the intersection. The State should exercise its judgment to develop the criteria that best apply this definition to their available data. Within each intersection type, the 3-year totals for intersection-related accidents will be divided by three to obtain annual counts. These annual counts will then be used in determining the calibration factor.

Step 5. Calculate Calibration Factors. For each intersection type, calculate the calibration factor (C_i) as the total number of reported annual intersection accidents (from step 4) divided by the total number of predicted annual intersection accidents (from step 3). As with the earlier roadway segment calibration, the user will then have the option of accepting the calculated factors or modifying them. Once accepted by the user, the values of the calibration factors for each intersection type should be entered into a file of default input values for the accident prediction algorithm. In subsequent applications of the accident prediction algorithm, these values of C_i will be used in applying equation (73). Again, it is suggested that the calibration factors be updated every 2 to 3 years. The same sample of intersections can be used at 2- to 3-year intervals for this purpose.

Calibration of Accident Severity Distribution

It is recommended that users of the accident prediction algorithm replace the default accident severity distribution shown in table 1 with values specifically applicable to the rural two-lane highways under a particular agency's jurisdiction. In step 3 of the preceding calibration procedures for roadway segments and step 4 of the procedures for intersections, the user will have identified appropriate files of accidents for roadway segments and intersections. It is recommended that these accidents be used in determining the accident severity distributions to update table 1.

Calibration of Accident Type Distribution

The default accident type distribution shown in table 2 can be calibrated by a user to obtain values specifically applicable to the rural two-lane highways under a particular agency's jurisdiction using a procedure that is entirely analogous to the procedure for accident severity distributions described above. The accident type distributions for both roadway segments and at-grade intersections in table 2 should be calibrated in this manner using the same accident data used to update table 1. It should be noted that the accident type distribution for roadway segments influences the AMFs for lane width, shoulder width, and shoulder type presented in section 4 of this report.

Calibration Factors for Subareas within a State

The calibration procedures presented earlier in this appendix for the roadway segment and intersection accident prediction algorithms will result in statewide calibration factors. If the State deems it necessary, then they can develop calibration factors for subareas in the State. For example, States may feel that it is necessary to developing unique sets of calibration factors for subareas within the State stratified by the following:

- Geographic area (e.g., upstate vs. downstate, plains vs. mountains, etc.).
- Terrain (e.g., flat areas, rolling areas, or mountainous areas).
- Highway district or region.

The calibration procedures previously described can be applied to develop multiple sets of calibration factors within the same State. If it is possible to stratify the State's mileage of all two-lane rural highways into the associated categories or levels that define the subareas, then sets of calibration factors can be developed for each unique category or level. Of course, the effort required will also increase. As more cells are added to the stratification matrix, sample sizes for individual cells will decrease. If the sample size is too small, then the resulting calibration factors may not be reliable. It is recommended that calibration factors only be used for the following absolute minimum sample sizes:

Type of intersection	Absolute minimum sample size (number of intersections)
STOP-controlled intersections	50
Signalized intersections	25

Local Calibration Factors

In addition to the calibration process described in this appendix, which can be thought of as a "global" calibration of the algorithms, another form of calibration is possible. Specifically, predictions made with the algorithm [including the calibration factors (C_r and C_i] can be further "calibrated" to existing local conditions by means of the EB procedure, which is described in section 6 of this report. It is important to recognize that for projects in which existing accident histories can be used, weights are assigned to the outputs from the accident prediction algorithm and the site-specific accident history within the EB procedure. Thus, a greater degree of importance can be assigned to site-specific accident histories than to the algorithm-generated prediction.

For alternatives primarily on new alignment, the EB procedure does not apply because the site-specific accident history of the old alignment is not necessarily representative of conditions on the new alignment. Thus, such projects should be analyzed without the EB procedure, but using the applicable calibration factors, C_r and C_i. If the analyst is concerned that safety conditions in a particular local area may differ substantially from the conditions represented by the "global" calibration factors, a special calibration study for a local area (possibly with a reduced sample size of roadway segments and/or intersections) can be performed using the procedures presented in this appendix. This might be particularly suitable for smaller areas with distinct populations or climate conditions.

APPENDIX D

DEFINITIONS OF ROADSIDE HAZARD RATINGS USED WITH THE ACCIDENT PREDICTION ALGORITHM

The accident prediction algorithm uses a roadside hazard rating system developed by Zegeer, et al. to characterize the accident potential for roadside designs found on two-lane highways.(6) Roadside hazard is ranked on a seven-point categorical scale from 1 (best) to 7 (worst). The seven categories of roadside hazard rating are defined as follows:

Rating = 1

- Wide clear zones greater than or equal to 9 m (30 ft) from the pavement edgeline.
- Sideslope flatter than 1:4.
- Recoverable.

Rating = 2

- Clear zone between 6 and 7.5 m (20 and 25 ft) from pavement edgeline.
- Sideslope about 1:4.
- Recoverable.

Rating = 3

- Clear zone about 3 m (10 ft) from pavement edgeline.
- Sideslope about 1:3 or 1:4.
- Rough roadside surface.
- Marginally recoverable.

Rating = 4

- Clear zone between 1.5 and 3 m (5 to 10 ft) from pavement edgeline.
- Sideslope about 1:3 or 1:4.
- May have guardrail (1.5 to 2 m [5 to 6.5 ft] from pavement edgeline).
- May have exposed trees, poles, or other objects (about 3 m or 10 ft from pavement edgeline).

- Marginally forgiving, but increased chance of a reportable roadside collision.

Rating = 5

- Clear zone between 1.5 and 3 m (5 to 10 ft) from pavement edgeline.
- Sideslope about 1:3.
- May have guardrail (0 to 1.5 m [0 to 5 ft] from pavement edgeline).
- May have rigid obstacles or embankment within 2 to 3 m (6.5 to 10 ft) of pavement edgeline.
- Virtually non-recoverable.

Rating = 6

- Clear zone less than or equal to 1.5 m (5 ft).
- Sideslope about 1:2.
- No guardrail.
- Exposed rigid obstacles within 0 to 2 m (0 to 6.5 ft) of the pavement edgeline.
- Non-recoverable.

Rating = 7

- Clear zone less than or equal to 1.5 m (5 ft).
- Sideslope 1:2 or steeper.
- Cliff or vertical rock cut.
- No guardrail.
- Non-recoverable with high likelihood of severe injuries from roadside collision.

Figures 8 through 14 present photographs illustrating the seven roadside hazard rating categories.

Figure 8. Typical Roadway with Roadside Hazard Rating Equal to 1.

Figure 9. Typical Roadway with Roadside Hazard Rating Equal to 2.

Figure 10. Typical Roadway with Roadside Hazard Rating Equal to 3.

Figure 11. Typical Roadway with Roadside Hazard Rating Equal to 4.

Figure 12. Typical Roadway with Roadside Hazard Rating Equal to 5.

Figure 13. Typical Roadway with Roadside Hazard Rating Equal to 6.

Figure 14. Typical Roadway with Roadside Hazard Rating Equal to 7.