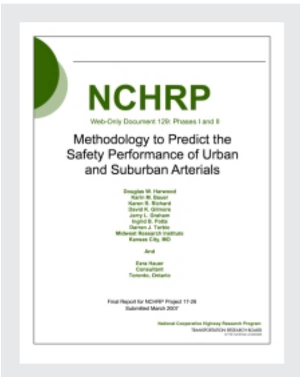

Methodology to Predict the Safety Performance of Urban and Suburban Arterials

DETAILS

0 pages | null | PAPERBACK
ISBN 978-0-309-42945-0 | DOI 10.17226/23084

AUTHORS

ACKNOWLEDGMENT

This work was sponsored by the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program (NCHRP), which is administered by the Transportation Research Board (TRB) of the National Academies.

THE NATIONAL ACADEMIES

Advisers to the Nation on Science, Engineering, and Medicine

The **National Academy of Sciences** is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. On the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Ralph J. Cicerone is president of the National Academy of Sciences.

The **National Academy of Engineering** was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. Charles M. Vest is president of the National Academy of Engineering.

The **Institute of Medicine** was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, on its own initiative, to identify issues of medical care, research, and education. Dr. Harvey V. Fineberg is president of the Institute of Medicine.

The **National Research Council** was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both the Academies and the Institute of Medicine. Dr. Ralph J. Cicerone and Dr. Charles M. Vest are chair and vice chair, respectively, of the National Research Council.

The **Transportation Research Board** is one of six major divisions of the National Research Council. The mission of the Transportation Research Board is to provide leadership in transportation innovation and progress through research and information exchange, conducted within a setting that is objective, interdisciplinary, and multimodal. The Board's varied activities annually engage about 7,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation. **www.TRB.org**

www.national-academies.org

PREFACE

This Final Report for Phases I and II is submitted by Midwest Research Institute (MRI) in accordance with the contractual requirements of NCHRP Project 17-26, *Methodology to Predict the Safety Performance of Urban and Suburban Arterials*. This report was prepared by Mr. Douglas W. Harwood, Ms. Karin M. Bauer, Ms. Karen R. Richard, Mr. David K. Gilmore, Mr. Jerry L. Graham, Ms. Ingrid B. Potts, and Dr. Darren J. Torbic of Midwest Research Institute, and Dr. Ezra Hauer.

This report covers the original scope of the research which addresses the preparation of a draft chapter on urban and suburban arterials for the forthcoming *Highway Safety Manual* (HSM). A recent addition to the project scope of work involves the development of a safety prediction methodology for collisions involving pedestrians on urban and suburban arterials. That work is currently underway in Phase III of the project and will be reported separately. The draft HSM chapter presented in Appendix B of this report will be revised when the pedestrian safety methodology is available.

Sincerely,

Midwest Research Institute

Douglas W. Harwood
Principal Traffic Engineer

Approved:

Robert G. Barton, Ph.D.
Director
Engineering Division

March 2007

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

BACKGROUND

The Transportation Research Board (TRB) and the American Association of State Highway and Transportation Officials (AASHTO) have begun a major initiative to develop a *Highway Safety Manual* (HSM). The HSM began from recognition of the need that, for safety to receive proper consideration in the highway project development process, analysts needed tools to make quantitative statements about the safety effects of proposed projects or design alternatives.

Most of the potential impacts of highway projects other than safety (e.g., traffic operations, air quality, noise, wetlands, construction cost) are addressed in quantitative fashion during the design process. However, while the accident history of the site may be documented quantitatively, most safety assessments of proposed projects are qualitative in nature and do not include explicit estimates of the likely safety effects of the project. Even when an attempt to develop quantitative estimates of safety effects is made, the accuracy of those estimates is limited by the lack of reliable tools for preparing such estimates. It is a concern that, when accurate traffic operational and environmental assessments are available, these factors may receive more weight in the decision making process, to the detriment of safety.

The HSM initiative grew out of the recognition that safety analysts need methodologies to predict safety similar to the quantitative methodologies that traffic operations engineers have available to assess capacity and level of service from specific traffic operational service measures using the *Highway Capacity Manual* (HCM) (*1*). The HSM initiative began with a conference session at the TRB annual meeting in January 1999 to explore the role of safety in the HCM. The consensus of the speakers at that session was that the HCM was already complex enough without trying to address safety and that a separate stand-alone document was needed. A workshop sponsored by TRB and FHWA was then held in Irvine, California, in December 1999 to consider the need for an HSM and develop a scope for a potential first edition. The Irvine workshop led to two specific initiatives: the formation of a TRB Joint Subcommittee on the Development of the Highway Safety Manual and the conduct of NCHRP Project 17-18(4) to prepare a detailed outline, a strategic plan, and a prototype chapter for the HSM. The TRB Joint Subcommittee has now become the TRB Task Force on Development of the Highway Safety Manual and may, in the future, be recognized as a full TRB Committee. AASHTO has formed a Joint Task Force to review and approve the HSM for publication.

The HSM will be targeted primarily to those on the front line of daily decision making in state highway agencies, and in local agencies including cities, counties, and metropolitan planning organizations (MPOs). These include analysts evaluating the potential effects of proposed improvement projects and those conducting planning, design, or operational studies.

The TRB Task Force has reviewed and approved a draft outline for the HSM, working from recommendations presented in *Development of a Highway Safety Manual* (*2*). The key components of the HSM will be:

LIST OF FIGURES

LIST OF TABLES

- Part I—Introduction and Fundamentals
- Part II—Knowledge
- Part III—Predictive Methods
- Part IV—Safety Management of a Roadway System
- Part V—Safety Evaluation

Thus, the HSM, as it has evolved, will be broader than simply providing quantitative safety prediction methodologies, but the methodologies in HSM Part III will still be the centerpiece of the HSM.

HSM Part III will consist of chapters that provide safety prediction methodologies for specific types of highway facilities. The TRB Task Force plans that the first edition of the HSM should include safety prediction methodologies for rural two-lane highways, rural multilane highways, and urban and suburban arterials. These methodologies would address the safety performance of both roadway segments and at-grade intersections on these facility types. A prototype chapter on rural two-lane highways for use in HSM Part III has already been developed to illustrate the potential format and scope for such chapters (*2*). This report presents research results which document the development of a draft HSM chapter presenting a safety prediction methodology for urban and suburban arterials. This is the first safety prediction methodology developed specifically for the HSM.

RESEARCH OBJECTIVE AND SCOPE

The objective of the research is to develop a methodology that predicts the safety performance of the various elements (e.g., lane width, shoulder width, use of curbs) considered in planning of nonlimited-access urban and suburban arterials. This methodology will be suitable for incorporation as a chapter in Part III of the HSM.

The scope of the research includes definition of the structure and content of the safety prediction methodology, obtaining the data best suited to development of the methodology, analyzing those data to develop valid statistical models of specific aspects of safety performance of urban and suburban arterials, fitting those models together into a comprehensive methodology, testing, and validating that methodology, and preparing a chapter that presents the model to users.

The urban and suburban arterials to be considered in the research potentially include all highway facilities in urban areas, other than freeways, that serve the primary function of serving through traffic movements. Highways which provide access to adjacent land as a primary or substantial function will not be considered. Although mobility is the primary function of an arterial, and the access function is less important than on collector or local roads, direct access to arterial facilities will play an important role in defining their safety performance. Thus, the relationship to safety of driveways, median openings, and median treatments on arterials will be a key consideration in the research.

ORGANIZATION OF THIS REPORT

This report presents the results of the literature review, the survey of potential HSM users, the recommended structure of the safety prediction methodology, and the recommended work plan for the methodology development.

The remainder of the report is organized as follows. Chapter 2 presents information related to the safety performance of urban and suburban arterials, including the results of the literature review and an HSM user survey. Chapter 3 summarizes the recommended structure for the safety prediction methodology. Chapter 4 presents the development of the project database. Chapter 5 documents the development of the base models and adjustment factors used in the HSM methodology, while Chapter 6 documents the accident modification factors (AMFs) used in the methodology. The HSM methodology is summarized in Chapter 7. Chapter 8 presents the results of a validation study in which the safety prediction methodology was applied to sites in jurisdictions other than those used in its development. Chapter 9 presents the conclusions and recommendations of the research.

Appendix A presents the results of the survey of state and local highway agencies, MPOs, and TRB Task Force members. Appendix B presents a draft of HSM Chapter 10 which incorporates the safety prediction methodology.

CHAPTER 2. SAFETY PERFORMANCE OF URBAN AND SUBURBAN ARTERIALS

This section of the report summarizes the safety performance of urban and suburban arterials by presenting the results of the literature review, and a survey of potential HSM users.

LITERATURE REVIEW

The results of the literature review presented here include the safety effects of lane width, shoulder width, horizontal curves, access features, and intersections on urban and suburban arterials. The literature focuses on results that have been reported, but does not address the many potentially relevant issues that have not been previously investigated. The literature review presents the results of each relevant study, but is not intended to provide a detailed critique of the methodology for each study. Many of the studies in the literature, particularly older studies, have methodological problems that should discourage reliance on their results, especially if results of better designed research are available. Methodological concerns of the this type were addressed in the assessment of the literature for the development of accident modification factors (AMFs) that is presented in Chapter 6 of this report.

Lane Width

Lane widths on urban and suburban arterials typically vary from 3.0 to 3.6 m (10 to 12 ft) (*3*). Lane widths of 3.0 m (10 ft) are sometimes used in highly restricted areas having little or no truck traffic. Lane widths of 3.3 m (11 ft) are used quite extensively for urban arterial street design, and lane widths of 3.6 m (12 ft) are desirable on higher speed, free-flowing, principal arterials.

A key aspect in the selection of the appropriate lane width on an urban or suburban arterial is safety. There are many reasons that might lead designers to consider the use of lanes narrower than 3.6 m (12 ft) on urban and suburban arterials. These include the desire to reduce pedestrian crossing distances; the desire to provide space for other roadway features such as medians, bicycle lanes, and curb parking; and the desire to provide space for roadside features such as sidewalks and clear zones and to minimize interference with existing roadside development. The benefits of these other features are substantial, and the decision to use narrower lanes might be an easy one, were it not for concern that provision of lanes narrower than 3.6 m (12 ft) would be accompanied by a reduction in safety. Therefore, an understanding of the relationship between lane width and safety is central to design decision making concerning urban and suburban arterials.

In a recent review of lane width and safety issues, Hauer (*4*) has indicated that there are two principal aspects to the potential link between lane width and safety:

- Wider lanes increase the average separation between vehicles moving in adjacent lanes and, therefore, may provide a wider buffer to accommodate small random deviations from their intended paths.
- Wider lanes may provide more room for corrective maneuvers by drivers in near-accident circumstances.

The possible safety effects of each of these potential linkages are discussed below. The following discussion is adapted from Hauer (*4*), but includes additional insights.

Hauer (*4*) states that the first possible linkage between lane width and safety is that, as lane width increases, so does the average separation between vehicles moving in adjacent lanes, in either the same direction or in opposite directions. This may provide a wider buffer to accommodate the small random deviations of vehicles from their intended paths. The research team for the current research believes that a wider buffer should also accommodate larger deviations that may occur if drivers are inattentive. However, Hauer (*4*) considers that it is also likely that wider lanes also induce faster travel speeds which may increase accident risk and is likely to increase accident severity. The 1985 edition of the HCM (*5*) suggested that wider lanes on multilane highways also increase capacity and, therefore, reduce following distances; however, HCM editions since 1985 (*1,6*) have indicated that wider lanes on multilane highways increase free-flow speeds, but do not increase capacity and, therefore, do not reduce vehicle headways.

The second possible linkage of lane width and safety noted by Hauer (*4*) is that wider lanes may provide more room for drivers to make corrective maneuvers in near-accident situations related to driver inattention. Specifically, the research team notes that, where lanes are wider, vehicles have the opportunity to travel at a greater separation from the edge of the traveled way and, thus, may be less likely to leave the roadway and run onto the roadside. Hauer (*4*) states that, for the driver of a vehicle in a narrow lane, a moment's inattention may result in the vehicle entering an adjacent lane or the roadside. Thus, the research team notes that wider lanes would be expected to increase the likelihood that inattentive drivers will recover before leaving their lane or entering the roadside. However, the likelihood of a reportable accident resulting from a roadside encroachment is largely a function of the design quality of the roadside. Where a wide paved shoulder is present, a roadside encroachment may have virtually no consequence; where there is a pavement edge drop-off, a poor shoulder, a curb at the edge of the traveled way, or steep slopes or substantial objects exist beyond the edge of the traveled way, a reportable accident, and possibly a severe accident, may result. This indicates that the safety effects of lane width may be difficult to quantify independently of shoulder and roadside design factors.

Hauer (*4*) indicates that the linkages described above are sufficiently complex that they do not indicate conceptually what the relationship between lane width and safety should be. This review of the literature begins with a discussion of the relationship between lane width and safety on urban and suburban arterials for motor vehicles, followed by discussions of safety-related lane width issues concerning pedestrians and bicyclists.

Motor Vehicle Collisions

The relationship between lane width and safety has been studied extensively in the rural environment. An expert panel (*7*) recently reviewed the literature on safety for lane widths on rural two-lane highways for the FHWA Interactive Highway Safety Design Model (IHSDM). This panel concluded that the most credible studies of lane width on rural two-lane highways were those by Griffin and Mak (*8*) for low-volume roads and by Zegeer et al. (*9*) for higher volume roads. Figure 1 presents the AMFs developed by the IHSDM expert panel (*7*) based on these past studies. The nominal or base condition for any AMF corresponds to an AMF value of 1.0. In the case of lane width, as shown in Figure 1, the nominal or base condition is a lane width of 3.6 m (12 ft). An AMF value greater than 1.0 represents a condition for which more accidents are expected than for the nominal or base condition. An AMF value less than 1.0 represents a condition for which fewer accidents are expected than for the nominal or base condition. An alternative name for AMF is crash reduction factor (CRF), although CRFs are typically expressed as percentage reductions in accident frequency rather than as multiplicative factors. Another expert panel in a later research study (*10*) concluded that the AMFs for rural two-lane highways shown in Figure 1 are also the best available estimates for rural multilane highways.

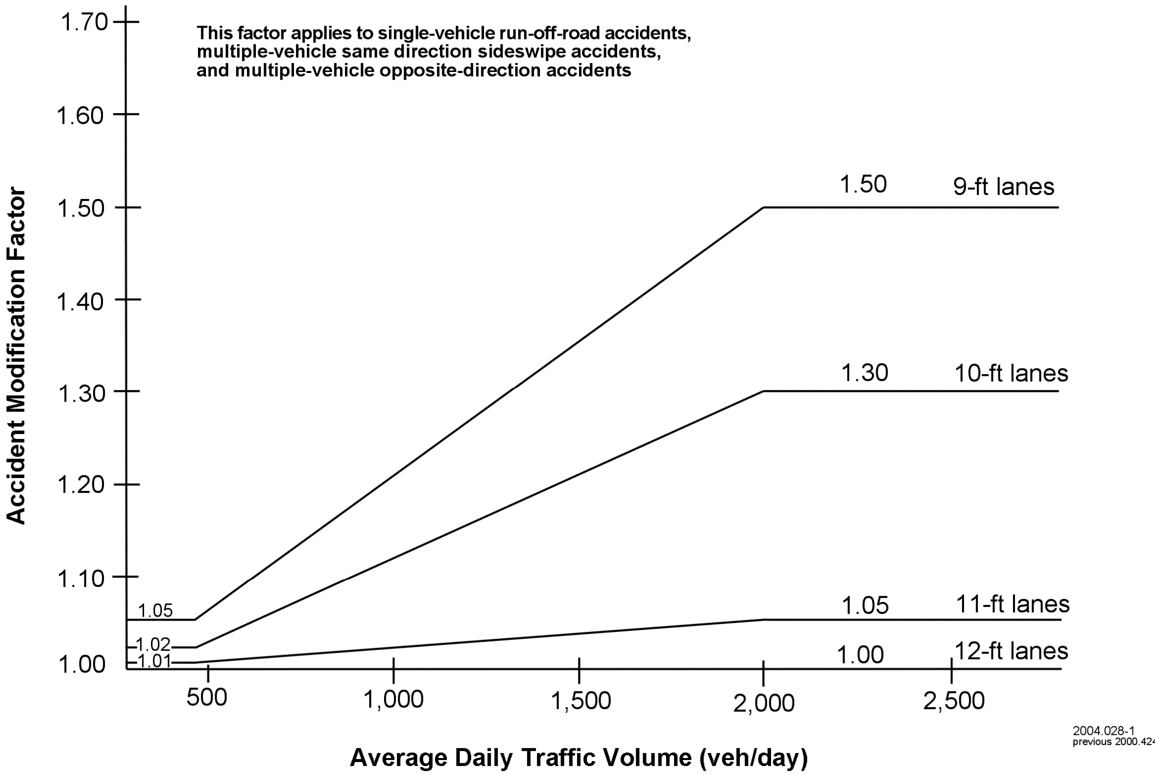

Figure 1. Lane width AMF for two-lane highways from IHSDM crash prediction module (7).

Only a few studies have researched the relationship between lane width and safety in the urban environment. Hauer (*11*) developed six statistical models to predict the nonintersection accident frequency of urban four-lane undivided roads. Separate models were developed for "off-the-road" and "on-the-road" property-damage-only (PDO), injury, and total accidents. From the three statistical models for "off-the-road" accidents, Hauer concluded that if accident frequency is influenced by lane width, it is not discernable. From the three statistical models for "on-the-road" accidents, lane width was found to be associated with PDO accidents but not with injury accidents. In the PDO model, wider lanes result in larger accident frequencies. However, Hauer notes that the relationship is weak, and lane width is only included in the model because of the traditional interest in this variable.

In 2001, Strathman et al. (*12*) analyzed design attributes and crash frequencies on the Oregon state highway system. The analysis differentiated the crash frequencies according to functional classification (freeway vs. nonfreeway) and location (urban vs. nonurban). Strathman et al. did not find any relationship between lane width and accident frequency.

Hadi et al. (*13*) developed negative binomial regression equations to estimate the safety effects of various cross-sectional elements for a number of different highway types. Hadi et al. found significant relationships between lane width and crash rates for undivided highways and urban freeways. Hadi considered both total and midblock crash rates, including crashes for each roadway type considered. In general, widening lane widths up to 3.6 m (12 ft) and 4.0 m (13 ft) would be expected to decrease crash rates on two-lane urban roads and four-lane urban undivided roadways, respectively. Figure 2 illustrates the potential benefits of lane widening for four roadway classes, including four-lane urban undivided highways.

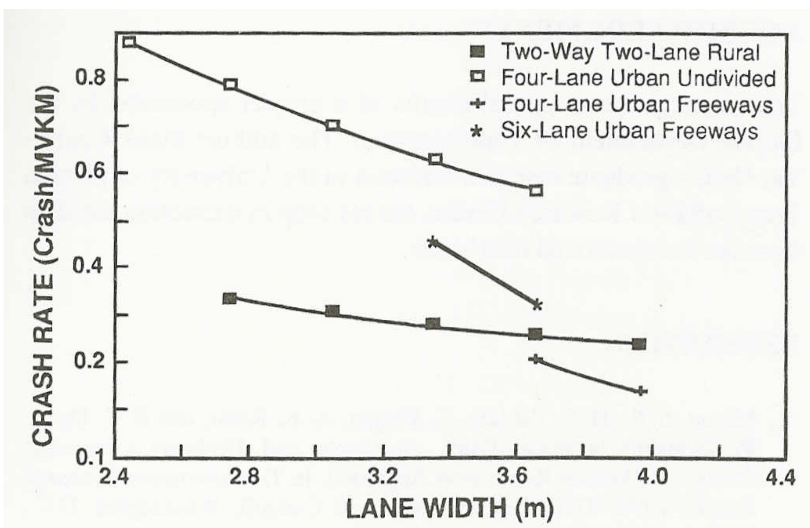

Figure 2. Effect of lane width on midblock crash rates (13).

Harwood (*14*) conducted research to determine the effectiveness of various alternative strategies for reallocating the use of street width on urban arterials without changing the total curb-to-curb width. The research addressed urban arterial streets with curb-and-gutter cross sections and speeds of 72 km/h (45 mph) or less. Harwood indicated that the preferred lane width for urban arterial streets under most circumstances is 3.3 or 3.6 m (11 or 12 ft). However, constraints on street widening do not always permit the use of lanes that wide, and under some situations, traffic operational benefits, traffic safety benefits, or both can be obtained from the use of narrower lanes.

Commercial buses operate frequently on many urban and suburban arterials. Traffic accidents involving buses result in about 35,000 injuries annually in the United States, as noted by Zegeer et al. (*15*). Zegeer et al. described bus and motor vehicle accident characteristics and recommended several highway improvements to reduce the number and severity of bus-related highway crashes. One of the recommendations concerned lane widths along urban streets. Zegeer et al. note that a primary transit bus accident type involves sideswipe collisions between buses and other motor vehicles. Because of the wider dimensions on buses, it is important that lane widths be adequate to minimize the chance for sideswipe collisions. With narrower lanes, the potential for sideswipe accidents increases. Therefore, Zegeer et al. recommended that along major arterials where buses and other large trucks are likely to operate, consideration should be given to providing lane widths of 3.6 m (12 ft), where possible, or at least 3.3 m (11 ft). This will increase the lateral spacing between buses and other motor vehicles, reducing the potential for accidents.

Recent research by Potts et al. (*16*) using the same database assembled for this report investigated the relationship between lane width and safety for midblock roadway segments and for intersection approaches on urban and suburban arterials. The research by Potts et al. found no indication of a consistent relationship between lane width and safety; except for a possible indication that accident frequencies may be higher on four-lane undivided arterials with 2.7 to 3.0 m (9 to 10 ft) lanes than on four-lane undivided arterials with 3.3 to 3.6 m (11 to 12 ft) lanes. This finding is in general agreement with the relationship developed by Hadi et al. (*13*) shown in Figure 2.

Pedestrian Collisions

No studies have been found that have used accident data to document the pedestrian safety implications of lane width. However, it may be reasoned that as crossing distances increase, pedestrian exposure time to motor vehicle traffic increases, increasing the potential of vehicle-pedestrian conflicts, and lane width can both directly and indirectly affect pedestrian crossing distance. As crossing distances increase, it may also be more difficult for pedestrians to judge the adequacy of gaps in traffic. It is also possible that as crossing distances increase, pedestrians compensate through greater attention to crossing safety. However, there is no research at present that explicitly addresses these issues.

Many countermeasures designed to improve pedestrian safety involve narrowing lanes to reduce vehicle speeds and enhance pedestrian mobility and safety. For example, curb extensions may be used at midblock locations to decrease pedestrian crossing distances and vehicle speeds. Chu and Baltes (*17*) studied midblock crossing difficulty as perceived by pedestrians and found that pedestrians perceive midblock street crossing to be more difficult with wider crossing distance. Other design features such as raised medians or refuge islands may also be used to reduce lane widths and/or crossing distances for pedestrians at both midblock and intersection locations. Curb extensions and other traffic calming measures may not be appropriate on major collector and arterial streets.

Zegeer et al. (*18*) performed a recent analysis of safety at marked and unmarked crosswalks. That study addressed the number of lanes that pedestrians needed to cross, but neither lane width nor crossing distance was considered explicitly.

Lane width may also affect a pedestrian's walking experience along a roadway. The width of travel lanes ultimately impacts the amount of space within the right-of-way for use by other modes (e.g., pedestrians and bicyclists). Space within the right-of-way may be designated for use by different modes of travel, and space may also be allocated as a buffer to separate modes or to provide a recovery for errant vehicles. Landis et al. (*19*) developed a pedestrian level of service model; this model does not deal with level of service in the same sense as that term is defined in the HCM (*1*) but, rather, is intended to quantify pedestrians' perception of safety and comfort in the roadside environment. The model quantifies how well roadways accommodate pedestrian travel along the roadway segment. Landis et al. found that a pedestrian's sense of safety and comfort is strongly influenced by the presence of a sidewalk and lateral separation relative to the motor vehicle traffic. In general, as the lateral separation increases, the pedestrian's sense of comfort or safety also increases. The elements in the final pedestrian level of service model related to lateral separation include:

- Width of outside lane
- Width of shoulder or bicycle lane
- Presence of on-street parking
- Buffer width (distance between edge of pavement and sidewalk)
- Presence and width of sidewalk

Thus, lane width (and shoulder width) is treated as explicit factors in the assessment of pedestrian safety and comfort. Earlier work by Dixon (*20*) to develop a pedestrian level of service methodology also incorporated lateral separation elements such as buffer space and pedestrian crossing widths.

Bicycle Collisions

No studies have been found that have used accident data to document bicycle safety implications of lane width. However, several studies indirectly addressed bicycle safety issues related to lane width by investigating the impacts of various types of bicycle facilities on surrogate measures of safety, such as motor vehicle speeds and lateral positioning. For the safety of bicyclists, the most advantageous facility is one that provides the greatest separation from motor vehicle traffic to allow for more reaction time to turning or entering vehicles into the traffic stream and to protect bicyclists from wind blasts and other effects from high volumes or high speeds of traffic (*21*).

McHenry and Wallace (*22*) studied the effects of different widths of wide curb lanes (WCLs) and the addition of bicycle lanes (BLs) in suburban settings. They determined that the optimal width for a WCL was 4.6 m (15 ft) and that BLs had advantages over WCLs such as less vehicle encroachment, lower vehicle displacement when passing a bicycle, and less variation in the lateral position of the vehicle and the bicycle.

In an evaluation of shared-use facilities, Harkey et al. (*21*) evaluated the impact of WCLs, BLs, and paved shoulders on motor vehicle and bicycle traffic. The principal findings included:

- On average, motorists positioned their vehicle approximately 2.0 m (6.4 ft) from a bicyclist in a WCL, 1.9 m (6.0 ft) from a bicyclist on a paved shoulder, and approximately 1.8 m (5.9 ft) from a bicyclist in a BL. Thus, the separation distance between bicyclists and motorists does not vary significantly by facility type.
- The distance between the bicyclist and the edge of roadway was considerably less along WCLs [0.43 m (1.4 ft)] compared to 0.73 m (2.4 ft) along facilities with paved shoulders or BLs.
- Motor vehicles moved to the left about 0.43 m (1.4 ft) further when passing a bicyclist on a WCL compared to paved shoulder and BL facilities.
- Encroachment into the adjacent lane to the left by motor vehicles when passing a bicycle was greater on WCLs (22.3 percent) than along BLs or paved shoulders (8.9 percent).

In similar research, Jilla (*23*) found that cars on narrow roads not only drifted toward the centerline when approaching bicyclists but also that motor vehicle speeds dropped. Bicyclists actually had a greater effect on the lateral position and speed of a vehicle than did other motor vehicles in the opposing lanes. Kroll and Ramey (*24*) found that the distance between passing motor vehicles and bicycles was more a function of a motorist's available travel space and not whether a BL was present. Based on their findings, Kroll and Ramey recommended that BLs be included when travel space is less than 4.6 m (15 ft). The overall conclusion of research conducted by Hunter et al. (*25*) was that both WCL and BL facilities can improve the safety and operations of bicycle traffic.

Several studies have been conducted, and different models developed, to measure the level of service that individual roads provide to bicyclists. As in the case of pedestrian level of service discussed above, the bicycle level of service concept departs from the traditional definition of level of service in the HCM (*1*) and is intended to characterize bicyclists' perceived level of safety and comfort, and not necessarily operational measures.

An early attempt to develop a bicycle level of service methodology was made in 1987 by Davis (*26*) at Auburn University. In this methodology each road segment and adjoining intersections were evaluated using a bicycle safety index rating (BSIR). The BSIR was found by calculating weighted averages of the following indexes:

- Roadway Segment Index (RSI)
  - ADT
  - Number of lanes
  - Speed limit

  - Width of outside traffic lane
  - Pavement factors
  - Location factors
- Intersection Evaluation Index (IEI)
  - Cross street volume
  - Traffic volume on route being indexed
  - Geometric factors
  - Signalization factor

Several counties in the state of Florida have used the Davis model, or variations of it, when rating the roads within their jurisdiction for bicycle LOS. It was this effort that led to Florida's roadway condition index (RCI).

Dixon (*20*) created a methodology to establish a point system by which Gainesville, Florida, could measure the LOS of the city's bicycle facilities. The factors incorporated into this approach included:

- Type of bicycle facility being provided
- Frequency of conflicts experienced by the bicyclist on the road
- Speed differential between bicyclist and motorist
- Motor vehicle LOS
- Maintenance problems of the road
- Availability of multimodal support for bicyclists

Landis et al. (*27*) focused upon creating a transferable model to determine bicycle level of service (BLOS) that could be applied in any metropolitan area. The BLOS is not a measure of capacity or level of operational service but is instead a measure of the comfort of the user within the roadway. The results are based exclusively upon human reactions to measurable traffic stimuli. In an effort to mathematically express the traffic conditions that affect bicyclist's perceptions, a model was created using the following variables:

- Per-lane traffic volume
- Traffic speed
- Traffic mix
- Cross-traffic generation
- Pavement surface condition
- Available roadway width for bicycling

Another measure of the suitability of roadways is the Bicycle Compatibility Index (BCI), developed for FHWA by Harkey et al. (*28*). The BCI is emerging as a useful measure for rating roadways with various types of bicycle facilities because of its broad applicability to a variety of locations and situations. It is intended to evaluate both urban and suburban roadways on their ability to accommodate motorists and bicyclists. The BCI model, presented in Table 1, incorporates a specific factor for curb lane width.

In summary, both of the two most widely used bicycle indices for urban and suburban arterials, the BLOS and the BCI, are directly sensitive to curb lane width. Neither the BCI nor the BLOS purports to represent a direct relationship to accident frequency.

TABLE 1. Bicycle Compatibility Index (BCI) model (*28*).

$$\mathbf{BCI} = 3.67 - 0.966\mathbf{BL} - 0.125\mathbf{BLW} - 0.152\mathbf{CLW} + 0.002\mathbf{CLV} + 0.0004\mathbf{OLV} + 0.035\,\mathbf{SPD} + 0.506\mathbf{PKG} - 0.264\mathbf{AREA} + \mathbf{AF}$$

where:

BL = presence of a bicycle lane or paved shoulder ≥ 3.0 ft
no = 0
yes = 1

BLW = bicycle lane for paved shoulder width
ft (to the nearest tenth)

CLW = curb lane width
ft (to the nearest tenth)

CLV = curb lane volume
Vph in one direction

OLV = other lane(s) volume – same direction
Vph

SPD = 85th percentile speed of traffic
mi/h

PKG = presence of a parking lane with more than 30 percent occupancy
no = 0
yes = 1

AREA = type of roadside development
residential = 1
other type = 0

AF = $f_t + f_p + f_{rt}$

where:

f_t = adjustment factor for truck volumes (see below)

f_p = adjustment factor for parking turnover (see below)

f_{rt} = adjustment factor for right-turn volumes (see below)

Adjustment factors

Hourly curb lane large truck volume[a]	f_t	Parking time limit (min)	f_p
≥ 120	0.5	≤ 15	0.6
60 – 119	0.4	16 – 30	0.5
30 – 59	0.3	31 – 60	0.4
20 – 29	0.2	61 – 120	0.3
10 – 19	0.1	121 – 240	0.2
< 10	0.0	241 – 480	0.1
		> 480	0.0

Hourly right turn volume[b]	f_{rt}
≥ 270	0.1
< 270	0.0

[a] Large trucks are defined as all vehicles with six or more tires.
[b] Includes total number of right turns into driveways or minor intersections along a roadway segment.

Shoulder Width and Curbs

Shoulders are desirable on all types of highways, including urban and suburban arterials. Shoulders serve multiple functions (*3,29*). For example, shoulders can:

- Enhance the controllability of vehicles that stray from the traveled way
- Accommodate stopped vehicles so that they do not encroach on the traveled way
- Make maintenance work easier (e.g., provide storage space for plowed snow)
- Facilitate access by emergency vehicles
- Provide space for slower vehicles to maneuver to allow faster vehicles to pass
- Serve as speed-change lanes for vehicles turning into driveways
- Provide space for pedestrians to walk where no sidewalks are present
- Provide space for bicyclists to ride where allowed by law
- Protect the structural integrity of the pavement

Roadways with shoulders provide open drainage arrangements where water can run off the traveled way onto the shoulder and then into a roadside ditch. Shoulders are typically provided on arterial roadways that enter developed areas from rural areas and on roadways with higher speeds. Despite the many advantages of shoulders, their use is generally limited on urban and suburban arterials because of restricted right-of-way and the necessity of using the available right-of-way for travel lanes. On many urban and suburban arterials, particularly where traffic speeds are low, a curb-and-gutter cross section that provides a closed drainage system is used. Curbs also provide a means for controlling access in heavily developed areas.

Even with these numerous advantages, the net safety benefit afforded by shoulders is difficult to ascertain because the net safety benefit of shoulders is due to the sum of several possibly opposite tendencies (*29*). For example, shoulders provide a fairly even and obstacle free surface where drivers of stray vehicles can regain control, recover from error, and resume normal travel. This should enhance safety. On the other hand, full shoulders also induce some amount of voluntary stopping (*30*), and vehicles stopped on shoulders pose a substantial risk of collisions with other vehicles. There is also the possibility that the presence of wider shoulders increases travel speeds, and even small increments of change in the mean speed may have noticeable effects on accident severity. In addition, the provision of wider shoulders may lead to steeper side slopes or back slopes, and the provision of wide shoulders may induce their occasional use as a travel lane. Thus, it is difficult to determine whether the combination of these conflicting tendencies benefits safety or is detrimental to it.

The relationship between shoulder width and safety has been studied extensively in the rural environment. An expert panel (*7*) recently reviewed the literature on safety for shoulder widths on rural two-lane highways for the IHSDM. The panel concluded that the most credible studies of shoulder width on rural two-lane highways were those by Zegeer et al. (*31*) for low-volume roads and another study by Zegeer et al. (*9*) for higher volume roads. Figure 3 presents the AMFs developed by the expert panel (*7*) based on these past studies. Another expert panel in a later research study (*10*) concluded that the AMFs for rural two-lane highways shown in Figure 3 are also the best available estimates for rural multilane highways.

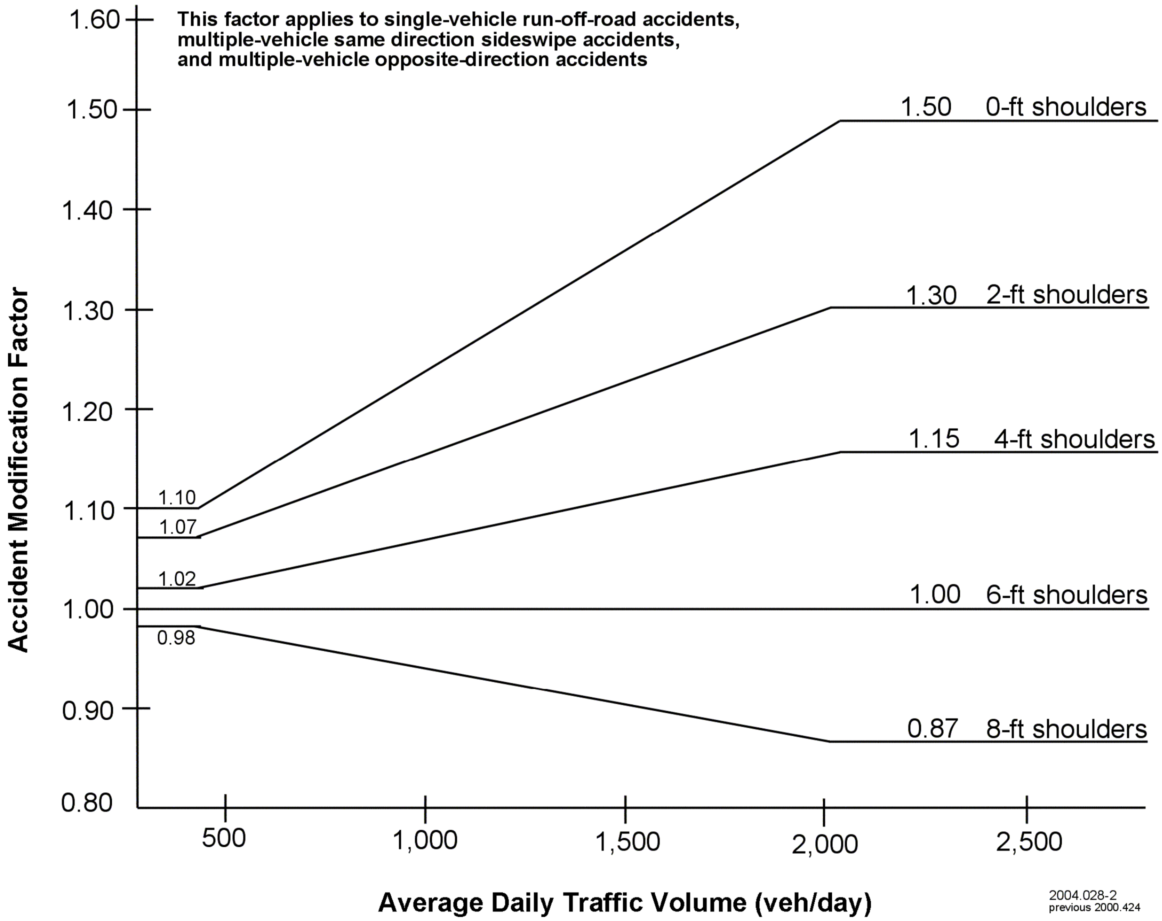

Figure 3. Shoulder width AMF for two-lane highways from IHSDM crash prediction module (7).

Hauer (*11*) developed several statistical models to estimate the frequency of nonintersection accidents on urban, four-lane, undivided roadways. Based upon models to predict "off-the-road" PDO, injury, and total accidents, Hauer concluded that the presence of a curb increases the likelihood of an accident compared to a flush shoulder. In addition, wider shoulders are expected to increase the number of "off-the-road" accidents. Based upon models to predict "on-the-road" accidents for the three severity types, Hauer concluded that wider shoulders are associated with more "on-the-road" injury accidents, while the relationship with PDO accidents is unclear, but Hauer notes that taking into consideration both shoulder type and width, the relationship of shoulders and safety is of marginal importance. Strathman et al. (*12*) also found the relationship between shoulder width and accident frequency to be insignificant.

Hadi et al. (*13*) developed negative binomial regression equations to estimate the safety effects of various cross-sectional elements for a number of different highway types. Hadi et al. found that the safety significance of outside total shoulder width, paved shoulder, and unpaved shoulder widths depends on the highway type. Figure 4 shows the degree to which Hadi et al. found increasing shoulders are expected to reduce the crash rates for the respective highway types.

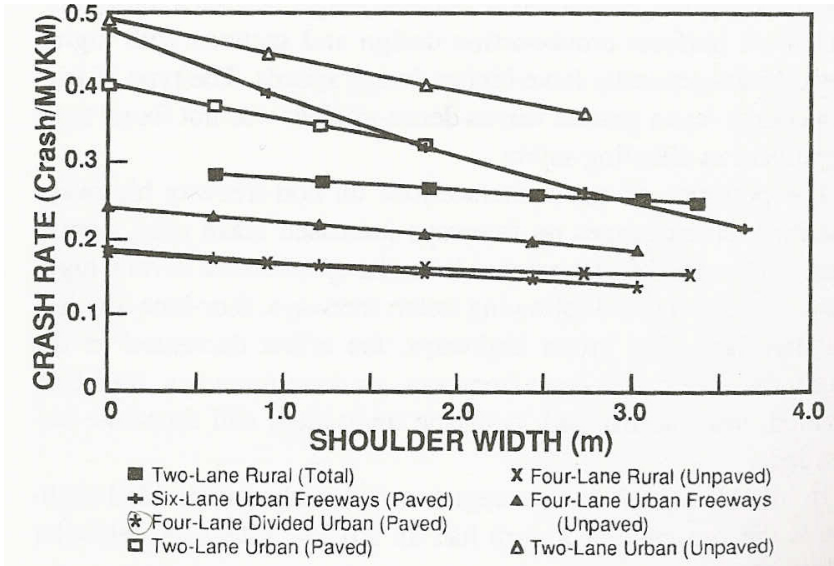

Figure 4. Effect of shoulder width on midblock crash rates (13).

Harwood (*32*) investigated and compared the safety, operational, and cost characteristics of selected multilane design alternatives for suburban highways. Harwood's finding in this study pertain to: roads with ADT greater than 7,000 veh/day, speed limit between 56 and 80 km/h (35 and 50 mph), and spacing between signalized intersections greater than 0.4 km (0.25 mi), driveway access from abutting properties, and no curb parking. Harwood concluded that the effect of providing full shoulders instead of a curb-and-gutter cross section decreases the accident rate by 10 percent. However, the safety effect of shoulders should depend on the roadside beyond the shoulder, and this factor was not taken into account within the analysis.

In their study of commercial bus accident characteristics and roadway treatments, Zegeer et al. (*15*) note that in suburban areas, some accidents occur when buses stop in the travel lane to pick up or drop off passengers, resulting in rear-end collisions. Zegeer et al. noted that such accidents could be reduced by providing paved shoulders of 2.4 to 3.6 m (8 to 12 ft) along such bus routes to allow buses to pull out of the through lane and onto the shoulder to pick up and unload passengers. Where continuous paved shoulders are not feasible, a paved pull-off lane at the bus stop should be considered. Such pull-off lanes are particularly important at locations where sight distance is severely limited.

There are no reliable studies in the literature that document the differences in safety between arterial roadways with shoulders and those with curb-and-gutter cross sections.

Horizontal Curves

Studies on rural highways have consistently shown that horizontal curves experience higher accident frequencies and greater accident severities than tangents (*33*). These higher accident frequencies and severities are not surprising given that curves require drivers not only to perceive a change in the roadway alignment but also to make necessary braking and steering adjustments. However, most of these studies address horizontal curves on rural roads.

Only one study has explicitly addressed the effects of horizontal curvature on urban arterials. Research by Hauer (*11*) concerning urban four-lane undivided arterials found that on-road accident frequencies on horizontal curves with radii greater than or equal to 103 m (337 ft) were lower than for comparable tangent sections of roadway. Figure 5 illustrates this relationship with the horizontal axis expressed as degree of curvature per 30 m (100 ft); the factor on the vertical axis represents the accident frequency relative to a tangent roadway. Hauer acknowledges that this relationship is counterintuitive and could represent a surrogate effect. While Hauer considers this relationship to be unresolved, he has encouraged consideration of the possibility—given the low proportion of single-vehicle accidents on arterials—that such an unexpected relationship could be correct.

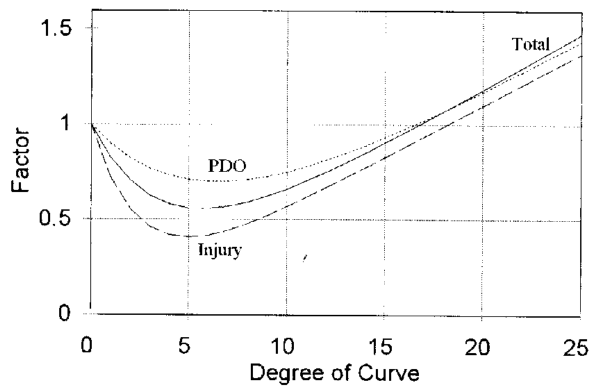

Figure 5. Relationship between on-road accidents and degree of curve for four-lane undivided urban arterials (11).

The remainder of the safety review of horizontal curves focuses on findings for curves on rural roadways, since there is no other research that directly concerns curves on urban roadways. Researchers have focused on both operational measures (i.e., design and operating speeds, speed change, lateral placement, and encroachments) and geometric measures (i.e., degree of curvature, length of curve, preceding tangent length, and roadway and shoulder widths) for horizontal curves. The applicability of these findings to urban arterials needs further assessment.

Operational Measures

Several studies have been conducted to evaluate the relationship between operating speed and accident experience at horizontal curves on two-lane rural highways. Two studies were conducted in the 1970s by Taylor et al. (*34*) and Stimpson et al. (*35*). Both evaluated the use of operational measures to compare the safety effectiveness of alternative delineation treatments. In the 1980s, FHWA sponsored studies by Datta et al. (*36*) and Terhune and Parker (*37*) to identify and evaluate various accident surrogates. Zegeer et al. (*38*) completed a study in 1990 which also considered the possible relationship between speed and accidents on horizontal curves. Finally, Anderson and Krammes (*39*) evaluated the use of speed reduction as a surrogate for accident experience at horizontal curves on rural two-lane highways. Each of these studies is discussed below in more detail.

Taylor et al. (*34*) evaluated the use of two operational measures, speed, and lateral placement, to compare the safety effectiveness of alternative delineation treatments. A sample of nine horizontal curves throughout Pennsylvania was selected. They measured speed and lateral placement at both the midpoint of the curve and the estimated point of curvature. A regression analysis indicated a "fairly strong correlation between accident rates and the variance of lateral placement." Although there appeared to be no correlation between accident rates and speed change from the beginning to the midpoint of the curve, their data did support the hypothesis that accident rates are correlated with deceleration rates on horizontal curves.

Stimpson et al. (*35*) performed correlation and stepwise multiple linear regression analyses of accident surrogates for tangent and winding alignments, and isolated horizontal curves. Unfortunately, the sample size (20 isolated horizontal curves and 78 accidents at those curves) was too small to produce conclusive results. For the tangent and winding alignments, however, they found lateral placement measures to be the best predictors of accident potential.

Datta et al. (*36*) conducted the first of two FHWA accident surrogate studies. They selected three highway situations to study: isolated curves on rural two-lane roads, signalized intersections on rural two-lane roads, and undivided two-lane tangent sections within urbanized areas. For the rural isolated horizontal curves, they studied both operational measures (encroachments and speed reduction) and nonoperational measures (degree of curvature; grade; shoulder width; distance since last traffic event requiring the driver to adjust speed or path; superelevation; roadside slope; and type, location and frequency of fixed objects). They used data from 25 curve sites in Michigan. They performed stepwise regression on all the sites and for subsets of sites with similar sight distance, grade, driveway density, and posted speed limits. They also evaluated accident rates for the inside lane, outside lane, and both lanes combined. The only variable that was statistically correlated with total accident rate was degree of curvature. However, using a subset of 15 curves with few driveways and a speed limit greater than or equal to 45 mph, they found a strong surrogate measure for outside lane accident rate. This model used the independent variables "distance to last traffic event on the outside lane" and "speed differential between the approach speed and curve midpoint speed for traffic in the outside lane" and resulted in a coefficient of determination (R^2) of 0.81.

Terhune and Parker (*37*) evaluated the surrogate measures identified by Datta et al. Their database consisted of 78 isolated horizontal curves in New York. They tested the Datta et al. equation for outside lane accident rates as a function of distance since last traffic event and speed differential and found an R^2 of only 0.01, compared with the R^2 of 0.81 in the study by Datta et al.

Zegeer et al. (*38*) conducted a study of the types of crashes that occur on rural two-lane curves. They studied police accident reports for 104 fatal and 104 nonfatal accidents on horizontal curves on two-lane rural highways in North Carolina. They found that 'the estimated speed prior to fatal crashes was much higher than to nonfatals." They concluded that "speed is a definite factor, perhaps in both the occurrence and also the severity of crashes on curves."

Using a speed-profile model, Anderson and Krammes (*39*) estimated the reduction in 85th percentile speeds from the approach tangent to the midpoint of 1,126 horizontal curve sites on rural two-lane highways in three states. The results suggested that estimated speed reduction is a useful measure that helps explain how accident experience at horizontal curves on rural two-lane highways varies with degree of curvature. Horizontal curves that require speed reductions have higher accident rates than curves that do not require speed reductions. Results of the analyses suggested that mean accident rate increases approximately linearly with the mean speed reduction.

As part of a large study of design consistency on two-lane roads for FHWA, Fitzpatrick et al. (*40*) developed a speed-prediction equation for horizontal alignments for use in assessing a design consistency on two-lane rural highways. Three years of accident data from over 5,000 horizontal curves were modeled as a function of log (AADT), log (curve length), and speed reduction. The following Poisson model suggested a strong relationship between speed reduction and accident frequency at horizontal curves:

$$Y = \exp(-0.8571)\ \mathrm{MVKT}\ \exp(0.0780\ \mathrm{SR}) \qquad \textbf{(1)}$$

where:

Y	=	number of accidents that occurred on the horizontal curve during a 3-year period
MVKT	=	exposure (million veh-km of travel for a 3-year period)
SR	=	speed reduction at horizontal curve from preceding tangent or curve (km/h)

The model indicates that the greater the speed reduction required by a horizontal curve, the greater its potential for accident experience. Fitzpatrick et al. concluded that speed reduction appears very promising as a design-consistency measure.

In response to a provision in the Surface Transportation Assistance Act of 1982, the National Research Council assembled a committee to conduct a study of the safety cost-effectiveness of geometric design standards for resurfacing, restoration, and rehabilitation (RRR) projects on existing highways (*41*). Regarding speed on curves, the committee recommended that highway agencies "evaluate the reconstruction of horizontal curves when the design speed of the existing curve is more than 24 km/h (15 mph) below the running speeds of approaching vehicles and the average daily traffic volume is greater than 750 vehicles per day."

Leisch and Leisch (*42*) presented a new concept in the definition and application of design speed, known as the "10-mph rule," which addressed the problem of the tendency of the driver to continually accelerate and decelerate due to inconsistencies in highway alignment. They state that "a reduction in design speed should be avoided if possible, but if it is required, it should be no more than 15 km/h (10 mph)" and that "within a given design speed, potential automobile speeds along the highway normally should vary no more than 15 km/h (10 mph)."

Choueiri and Lamm (*43*) recommended a method of evaluating designs based on ranges of change in 85th percentile speed. They recommend rating a design as follows:

Speed Range	Design Safety Level
$\Delta V_{85} \leq 6$ mph	good
6 mph $< \Delta V_{85} \leq 12$ mph	fair
$\Delta V_{85} > 12$ mph	poor

Geometric Measures

Glennon et al. (*44*) developed a discriminate analysis model to use in identifying horizontal curves with high accident potential. The model is based on geometric, traffic, and roadside conditions and was derived using data from 298 curve sections. They found that curves with higher degrees of curvature, greater curve length, more hazardous roadsides, lower skid resistance, and/or more narrow shoulders have a greater likelihood of being a high-accident location.

Zegeer et al. (*38*) evaluated the relationships between horizontal curve features and accident experience. They developed cross-sectional models that were then used to quantify the effects on accidents resulting from curve flattening, curve widening, adding a spiral, improving deficient superelevation, and clearing the roadside. Their database consisted of 10,900 curves in the state of Washington. Statistical models indicated that accident rates increased with sharper curves, narrower lane width, lack of spiral transitions, and inadequate superelevation. Based on predictive models they developed, Zegeer et al. estimated the effects of several curve improvements on accident experience as follows:

- *Curve flattening* reduces crash frequency by as much as 80 percent, depending on the central angle and amount of flattening.
- *Widening lanes* on horizontal curves is expected to reduce accidents by up to 21 percent for 1.2 m (4 ft) of lane widening.
- *Widening paved shoulders* can reduce accidents by as much as 33 percent for 3 m (10 ft) of widening.
- *Adding unpaved shoulders* is expected to reduce accidents by up to 29 percent for 3 m (10 ft) of widening.
- *Adding a spiral* to a new or existing curve will reduce total curve accidents by approximately 5 percent.
- *Improving superelevation* can significantly reduce curve accidents where there is a superelevation deficiency. An improvement of 0.02 would be expected to yield an accident reduction of 10 to 11 percent.

Most safety researchers have long supposed that sharp horizontal curves are likely to have higher accident rates when preceded by long tangent sections of roadway than when preceded by short tangents or another horizontal curve. However, there has been little hard evidence to support this idea. Potts et al. (*45*) used data from the FHWA Highway Safety Information System (HSIS) on the safety performance of horizontal curves and their preceding tangents on rural two-lane highways to investigate this issue. Specifically, the study examined whether the accident rate of a horizontal curve increases with preceding tangent length and how that effect varies with horizontal curve radius. The researchers found that the accident rate on a horizontal curve appears to be influenced by the length of the approach tangent only for two limited situations:

- horizontal curves with radii of 200 m (656 ft) or less and having approach tangents in the range from 300 to 500 m (984 to 1,640 ft)
- horizontal curves with radii of 200 to 340 m (656 to 1,115 ft) and having approach tangents in the range of 150 to 300 m (492 to 984 ft)

For horizontal curves with radii of 200 m (656 ft) or less, the accident rate for curves with approach tangents in the range of lengths from 300 to 500 m (984 to 1,640 ft) was found to be approximately 50 percent higher than for curves with other approach tangent lengths. For horizontal curves with radii of 200 m to 340 m (656 to 1,115 ft), the accident rate for curves with approach tangents in the range of lengths from 150 to 300 m (492 to 984 ft) was found to be approximately 10 percent higher than for curves with other approach tangent lengths.

Harwood et al. (*7*) developed 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 nominal or base condition for horizontal alignment is a tangent roadway section. An accident modification factor (AMF) was developed, based on research by Zegeer et al. (*38*) discussed above, to represent the manner in which accident experience of curved alignments differs from that of tangents. The accident modification factor is a function of the length of the curve, the radius of the curve, and the presence or absence of a spiral:

$$AMF = \frac{1.55 L_c + \frac{80.2}{R} - 0.012 S}{1.55 L_c} \qquad (2)$$

where:

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

An expert panel in later research (*10*) considered the applicability of Equation (2) to multilane highways and recommended that, for application in IHSDM, the AMF be increased by

20 percent for sharper horizontal curves [radius of 305 m (1,000 ft) or less] and that the AMF be decreased by 20 percent on flatter horizontal curves [radius of 488 m (1,600 ft) or more].

Access Features

The literature suggests a strong relationship between accident rates and various forms of access management. In fact, research shows that access managed roadways have between 50 and 65 percent fewer accidents than roadways with no access management (*46*). This section presents the results of research studies that have looked at the relationship of safety to access in terms of (a) general degree of access control, (b) access spacing, and (c) driveways located near intersections.

Degree of Access Control

Several studies have been conducted to evaluate the safety effectiveness of access management.

The Colorado Department of Transportation has demonstrated the safety benefits of access control on urban arterials (*47*). The results of this project showed that access controlled arterials have accident rates ranging from 27 to 69 percent of arterials without access control.

Flora and Keitt (*48*) examined the effect of general levels of access control on urban arterials. Table 2 presents urban arterial accident rates as a function of level of access control, as found by Flora and Keitt. Accident rates for each level of access control are presented in the table; each accident rate is also expressed as a percentage of the accident rate for arterials with no access control.

TABLE 2. Urban arterial accident rates as a function of level of access control (*48*)

Level of access control	Number of accidents per 100 million veh-km			
	Total		Fatal	
None	329	(100%)	2.50	(100%)
Partial	310	(94%)	2.80	(112%)
Full	116	(35%)	1.25	(50%)

Gattis and Hutchison (*49*) compared three urban arterial roadways in Springfield, Missouri, to determine the relationship between safety and access control. Each roadway had similar lengths, posted speed limits, volumes, and abutting land uses but different levels of access control. The roadway with the highest level of access management (nontraversable median and greater access spacing) was found to have a lower crash rate than the other two roadway sections with a center turn lane. A comparison of the two center turn lane roadways found that an increase in driveways spacing did not correspond to a lower crash rate.

Access Spacing (Driveway Density)

Over the past 50 years, many studies have documented how accidents increase with decreasing access spacing. In a recent paper on access spacing and safety, Levinson and Gluck (50) prepared a chronology of accident studies, conducted between 1952 and 1997, related to access spacing. Table 3 presents the chronology of studies.

In the study conducted by Lall et al. (*71*), an accident analysis was conducted on a 47-km (29-mi) segment of the Oregon Coast Highway (US 101) to evaluate the relationship between accident frequency and access density. Results of the research showed a direct relationship between access density and accident frequency and severity. Figure 6 illustrates the relationship between accident rate and access density that was found in the Oregon study, with a breakpoint at around 30 access points per km (50 access points per mi).

TABLE 3. Chronology of accident studies related to access spacing (*50*)

Study No.	Year	Description	Findings	Source
1	1952	McMonagle, Michigan	An increase from 0 to 4 or more roadside features per 1,000 ft increases accidents/million VMT from 3.37 to 13.48	(*51*)
2	1953	Staffield, Minnesota	Roadways with more than 20 access points per mile had more than double the rates of roads with less than 4 access points per mile	(*52*)
3	1957	Schoppert, Ohio	The number of access points along rural two-lane highways is a reasonably good predictor of potential accidents within an ADT group	(*53*)
4	1959	Head, Oregon	Accident rates increased as the number of commercial driveways and/or commercial units per mile increased	(*54*)
5	1967	Cribbins et al., North Carolina (92 roadway sections)	Accident and injury rates on multilane divided highways increased as the number of access points and their traffic volumes increased	(*55*)
6	1967	Mulinazzi and Michael, Indiana (100 roadway sections)	The number of medium and heavy volume commercial driveways per mi was significantly related to the accident rates for sections with less than 5,800 ADT	(*56*)
7	1970	Dart and Mann, Louisiana	Accident rates doubled as the traffic conflicts increased ten times	(*57*)
8	1970	Cirillo, Interstate System Accident Research	As intersections/per mi increased from 1 to 15, accident rates increased 4 to 5 times on urban highways and 2 to 3 times on rural highways. As business access points/per mi increased from 1 to 40, accident rates doubled	(*58*)
9	1973	McGuirk, Indiana (63 mi)	Accidents per mile may decrease when the number of commercial driveways, traffic volumes, or travel lanes is reduced	(*59*)
10	1974	Uckotter, Indiana (14 roadway sections)	Regression equations produced counter intuitive results	(*60*)

TABLE 3. Chronology of accident studies related to access spacing (*50*) (Continued)

Study No.	Year	Description	Findings	Source
11	1975	Glennon et al., Azzeh et al.	An increase from low to high driveway frequency doubles annual accident frequency. An increase from low to high volumes (over 15,000 ADT) triples annual accidents	(*61,62,63*)
12	1985	Arapahoe and Parker Roads, Denver (4.35 and 5.16 mi)	Two highly access managed roads had about 40% the accident rate of roads with frequent access	(*47*)
13	1986	Waushara County, Wisconsin	Annual accidents per mile for access spacing less than 300 ft was about 2 to 3 times greater than for longer spacing	(*64*)
14	1992-1993	Sokolow et al., Long et al., Florida	Accident rates doubled when driveways exceeded 20 per mile (Sokolow) Accident rates increased 70% as driveways per mile increased from less than 13 to more than 20. It was estimated that one driveway adds 0.02 crashes per year.	(*65,66*)
15	1993	British Columbia (176 roadways sections, 465 mi)	Accident rates increased as access density increased. Each business access impacted accident rates at about 50% of public roadway intersections	(*67*)
16	1993	Millard, Florida	Doubling access points from 20 to 40 per mile doubled the accident rate. Doubling signals from 2 to 4 per mile, more than doubled the accident rate	(*68*)
17	1994	Michigan	Midblock accident rates generally increased as the number of intersections per mile (including driveways) and the number of lanes increased	(*69*)
18	1995	Fitzpatrick and Balke, Texas	Total and midblock accidents generally increased as driveways became more numerous	(*70*)
19	1995	Lall et al., Oregon (US 101—29 mi)	Accidents per mile and driveways per mile followed similar patterns (except for roadways sections with a nontraversable median)	(*71*)
20	1996	Norwalk-Wilton, Connecticut (Route 7)	Accident rate per mile increased along roadway carrying 20,000 to 25,000 vehicles per day as access density increased	(*72*)
21	1996	Garber and White, Virginia (10 mi, 30 locations)	Multiple regression analysis assessed effects of ADT/lane, average speed, number of access points, left-turn lane availability, average access spacing and average difference in access spacing	(*73*)
22	1997	Australia	Each additional driveway per km increased accident rates about 1.5% for two-lane roadways and 2.5% for four-lane roadways	(*74*)

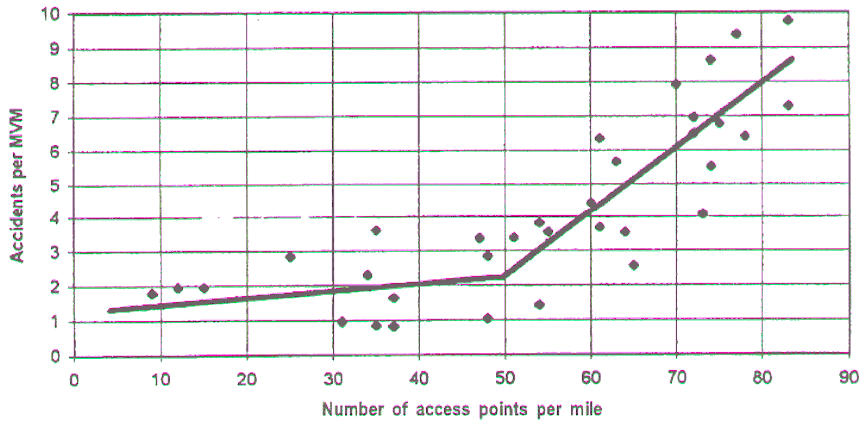

Figure 6. Density of accidents and access points on urban arterials (69).

Li (*67*) conducted a study of the relationship between safety and access density. Li found a clear relationship between increased number of access points (minor intersections or driveways) and accident rate. Figure 7 shows the relationship between accident rate and unsignalized intersection density on four-lane urban highways. Figure 8 illustrates the relationship between accident rate and business access density on four-lane urban highways.

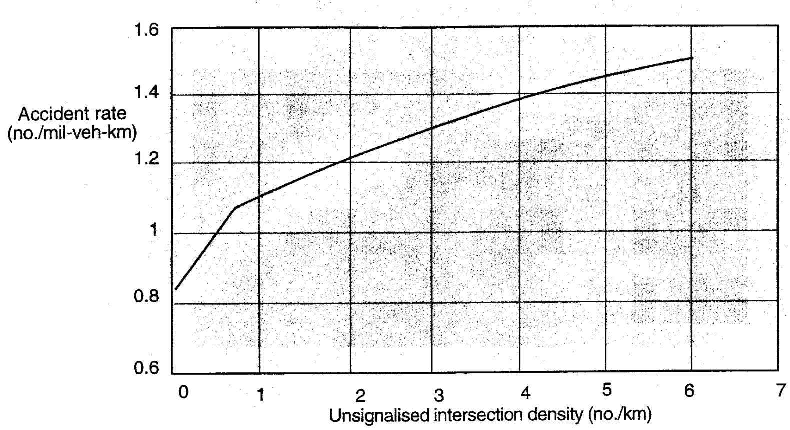

Figure 7. Estimated relationship between accident rate and unsignalized intersection density on four-lane urban highways (67).

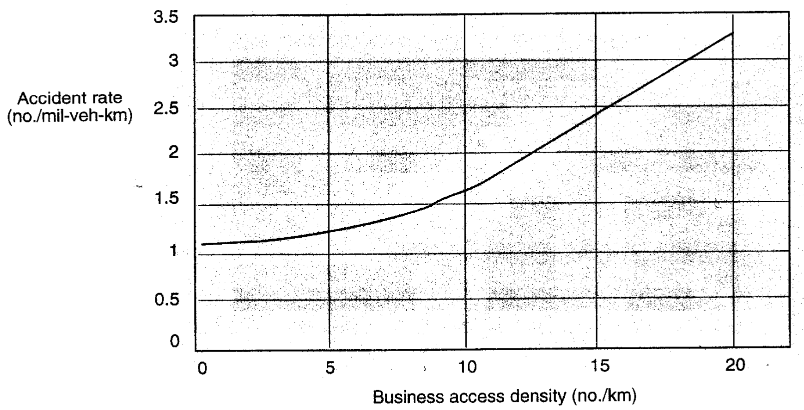

Figure 8. Estimated relationship between accident rate and business access density on four-lane urban highways (67).

A study was conducted for the Minnesota Department of Transportation to determine the relationship between access management and safety (*75*). Data were collected from a representative random sample of urban and rural roadway segments on Minnesota's state highway system. The data included 432 roadway segments, 1,231 km (765 mi) of roadway, over 9,000 access points and nearly 14,000 crashes (over the 3-year period 1994-1996). The roadway segments were divided into 11 roadway segment categories (5 rural and 6 urban) depending on number of lanes, presence of left-turn lanes, and whether the roadway segment was considered an expressway. A positive relationship between access density and crash rates was observed in 10 of the 11 highway categories. Analysis of the crash data in each of the roadway categories revealed that roadway segments with the highest crash rates have high levels of access density and segments with the lowest crash rates have low levels of access density.

NCHRP Report 420 (*76*) reports on a comprehensive safety analysis to investigate the relationship between access spacing and accidents. Accident data from 264 roadway segments, including 170 urban roadways, from five states were analyzed. Results of the analysis showed that each access point (or driveway) increases the annual accident rate by about 0.07 to 0.11 accidents per million veh-km (0.11 to 0.18 accidents per million veh-mi) of travel on undivided highways and by 0.06 to 0.08 accidents per million veh-km (0.09 to 0.13 accidents per million veh-mi) of travel on highways with two-way-left-turn lanes or nontraversable medians.

While nearly all studies have concluded that increased driveway density has a negative effect on safety, not all studies agree on the shape of this relationship. That is, some studies suggest that accident rate increases linearly with access point density, while other studies have found the increase to be more than linear. Some studies claim that accident rate increases with the square root of access point density. After reviewing a number of studies evaluating the relationship between driveway density and safety, Hauer (*77*) suggests that the linear assumption may be best supported.

No studies were found that address the effect of driveway traffic volumes on the frequency of driveway-related accidents.

Driveways Located Near Intersections

Another safety issue related to driveways is their location relative to intersections. It has been suggested that locating driveways away from intersections reduces the number of conflicts and provides more time and space for vehicles to turn or merge safely across lanes (*78*).

Box (*79*) analyzed over 15,000 accidents in two Illinois suburbs to determine the effect of intersections on driveway accidents. Driveway accidents related to intersections were found to represent 1.2 percent of total accidents in one suburb and 2.0 percent of total accidents in the other suburb. The research results support access management policies that restrict driveways from being located within some minimum distance of an intersection. Box recommended driveway spacing guidelines as presented in the ITE *Guidelines for Driveway Location and Design* (*80*).

Intersections

Intersections are one of the most complex areas or locations on the highway system. The complexity arises because motorists and other highway users, such as pedestrians and bicyclists, must share the right of way through intersections, and as a result, conflicts occur among users as they cross paths. In addition, various geometric design and traffic control features may be implemented at intersections, adding to the complexity. Statistics indicating the nature of the safety problem at intersections are presented followed by information on the safety effects of a number of intersection geometric design and traffic control features, including:

- Left-turn lanes
- Right-turn lanes
- Channelization
- Intersection skew angle
- Sight distance
- Approach width
- Number of approach lanes
- Median type and width
- Pedestrian facilities
- Bicycle facilities

Research results are reported for both urban and rural areas to provide a comparative perspective of intersection safety conditions.

Much of this section on intersections comes from a recent study conducted by Harwood et al. (*81*) which included a comprehensive literature review on the safety effects of a wide variety of geometric design, traffic, and control elements for at-grade intersections. This review focuses on the safety efforts of geometric design elements of at-grade intersections.

b

Left-Turn Lanes

Installation of left-turn lanes has been the focus of many research studies. Various safety-related impacts have been documented depending upon the type of intersection (signalized, unsignalized, four-leg, etc.) where the left-turn treatment was implemented, the location of the intersection (urban or rural), as well as the different types and/or severity of accidents. Results from studies conducted in urban areas are presented first, followed by research results from rural areas.

Harwood et al. (*81*) conducted the most recent and extensive investigation into the safety effectiveness of geometric design improvements at intersections, including the installation of left-turn lanes at urban intersections. Harwood et al. performed a before-after evaluation of the safety effects of providing left-turn lanes for at-grade intersections. Geometric design, traffic control, traffic volume, and accident data were collected at 100 urban intersections where at least one left-turn lane had been installed. Data were also collected at 100 intersections that exhibited similar characteristics to the improved sites but did not have left-turn lanes installed. Three contrasting approaches to before-after evaluations were used to evaluate the safety effectiveness of left-turn lane improvements: the yoked comparison or matched-pair approach, the comparison group approach, and the empirical Bayes (EB) approach.

Harwood et al. (*81*) concluded that left-turn lanes are effective in improving safety at signalized and unsignalized intersections in urban areas. At urban unsignalized intersections, installation of a left-turn lane on one approach would be expected to reduce total intersection accidents by 27 percent for four-leg intersections and by 33 percent for three-leg intersections. At four-leg urban signalized intersections, installation of a left-turn lane on one approach would be expected to reduce accidents by 10 percent. Installation of left-turn lanes on both major-road approaches to a four-leg intersection would be expected to increase, but not quite double, the resulting effectiveness measures for total intersection accidents. Table 4 presents the recommended accident modification factors (AMFs) for the installation of left-turn lanes on major-road approaches to urban intersections.

TABLE 4. AMFs for installation of left-turn lanes at urban intersections (*81*)

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]	0.67	–
	Traffic signal	0.93	–
Four-leg intersection	STOP sign[a]	0.73	0.53
	Traffic signal	0.90	0.81

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

McCoy and Malone (*82*) analyzed the accident experience at signalized and unsignalized intersections on urban, four-lane roadways in Nebraska to assess the safety effects of left-turn lanes. McCoy and Malone concluded that left-turn lanes at intersections on urban, four-lane roadways significantly reduce rear-end, sideswipe, and left-turn accidents; however, on the uncontrolled approaches of intersections on urban undivided roadways, left-turn lanes significantly increase right-angle accidents, as well as reduce rear-end, sideswipe, and left-turn accidents.

Based upon research by McFarland (*83*), Hauer (*84*) examined the ways in which the use of intersections by older persons, both as pedestrians and as drivers, could be made safer and easier. Hauer concluded that the provision of left-turn channelization at unsignalized intersections reduced accidents by 70 and 65 percent in urban and suburban areas, respectively, when combined with curb or raised bars. When channelization was painted at unsignalized intersections, accidents decreased by 15 and 30 percent, respectively, in urban and suburban areas. In general, Hauer concluded that adding left-turn lanes reduced accidents by varying amounts depending upon the type of intersection, whether it was signalized or unsignalized, and whether the intersection was urban, suburban, or rural.

Lacy (*85*) found that left-turn lanes, when coupled with several other safety improvements, reduced accident frequency by 35 percent and accident severity by 80 percent at urban intersections.

Not all studies, however, have shown that left-turn lanes are associated with reduced accidents. Bauer and Harwood (*86,87*) developed several statistical models to predict accidents for at-grade intersections. In the model development, left-turn channelization was found to significantly increase total accidents and fatal and injury accidents at urban, three-leg, stop-controlled intersections. However, left-turn channelization was found not to significantly affect total and fatal and injury accidents at urban, four-leg, stop-controlled intersections, nor at urban, four-leg, signalized intersections. Poch and Mannering (*88*) developed accident prediction models using negative binomial distributions based upon data from 63 urban intersections in Washington. The models indicated that approaches with a designated left-turn lane, a through lane, and a shared through-right-turn lane had more rear-end accidents than those with other conditions. Likewise, David and Norman (*89*) investigated the motor vehicle accident relationship with geometric and traffic features at intersections. They determined that for average daily traffic (ADT) volumes between 10,000 and 20,000 veh/day, four-leg intersections with opposing left-turn lanes had more accidents than those without. The concerns raised by these studies have largely been resolved by the Harwood et al. (*81*) study, which found that left-turn lanes have a definite positive effect on safety.

The Harwood et al. (*81*) study also evaluated the safety effects of providing left-turn lanes for rural at-grade intersections. This study concluded that the installation of a single left-turn lane on a major-road approach would be expected to reduce accidents at rural unsignalized intersections by 28 percent for four-leg intersections and by 44 percent for three-leg intersections. Table 5 presents the recommended AMFs for the installation of left-turn lanes on major-road approaches to rural intersections.

TABLE 5. AMFs for installation of left-turn lanes at rural intersections (*81*)

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]	0.56	–
	Traffic signal	0.85	–
Four-leg intersection	STOP sign[a]	0.72	0.52
	Traffic signal	0.82	0.67

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

Other studies document the safety effectiveness of left-turn lanes based upon data from both urban and rural intersections or, in some cases, do not distinguish whether the results pertained to urban or rural intersections. NCHRP Report 420 (*76*) reported accident rate reductions ranging from 18 to 77 percent due to the installation of left-turn lanes based upon a review of previous research by the New Jersey Department of Transportation (*90*), Griewe (*91*), Agent (*92*), Ben-Yakov and Craus (*93*), Craus and Mahalel (*94*), Tamburri and Hammer (*95*), and Wilson et al. (*96*). Maze et al. (*97*) developed a model that predicted a reduction in left-turn accident rate of 6 percent due to the installation of a left-turn lane with permitted signal phasing and a reduction of approximately 35 percent from the installation of a left-turn lane with protected/permitted signal phasing. Foody and Richardson (*98*) found that accident rates decreased by 38 percent with the addition of a left-turn lane at signalized intersections and by 76 percent at unsignalized intersections.

An issue in the design of left-turn channelization is the restriction in sight distance that opposing left-turn vehicles cause one another. A potentially effective countermeasure for safety problems where opposing left-turn lanes are present is to eliminate the sight restrictions by offsetting the left-turn lanes. Harwood et al. (*99*) reviewed the safety performance of a limited set of tapered and parallel offset left-turn lanes and found no safety problems. Both McCoy et al. (*100*) and Joshua and Saka (*101*) developed procedures to compute the amount of offset required to clear sight lines. However, no evaluations of the accident reduction effectiveness of offset left-turn lanes have been found.

Table 6 summarizes the results of those studies that provided quantitative estimates of the effectiveness of installing left-turn lanes at intersections.

Right-Turn Lanes

It is generally accepted that the installation of right-turn lanes improves safety for motor vehicles, but only a limited number of quantitative estimates are available to demonstrate this. The research findings that are available are summarized below. Research findings related to channelized right-turn lanes are included.

TABLE 6. Summary of research results concerning the safety effectiveness of installing left-turn lanes [adapted from Harwood et al. (*81*)]

Source	Percentage change in accident frequency for addition of a LTL: Total intersection accidents	Percentage change in accident frequency for addition of a LTL: Left-turn accidents	Conditions/comments	Area type: Urban/suburban	Area type: Rural	Area type: Combination or unknown
Harwood et al. (*81*)	–7 to –33	–	LTL on one major-road approach	X		
	–19 to –47	–	LTLs on two major-road approaches	X		
	–15 to –44	–	LTL on one major-road approach		X	
	–33 to –48	–	LTLs on two major-road approaches		X	
Harwood et al. (*7*)	–18 to –24	–	Two-lane highway; LTL on one major-road approach		X	
	–32 to –42	–	Two-lane highway; LTLs on two major-road approaches		X	
Vogt (*102*)	–38	–	LTL at four-leg rural intersection with four-lane major road and two-lane minor road		X	
Maze et al. (*97*)	–6	–	Signalized intersection; LTL with permitted phasing			X
	–35	–	Signalized intersection; LTL with protected/permitted phasing			X
New Jersey Department of Transportation (*90*)	–35 to –51	–	LTL installation along Route 130 in New Jersey			X
Griewe (*91*)	–58	–6	Eight LTLs added by restriping			X
Agent (*92*)	–77	–	Unsignalized intersection			X
	–54	–	Signalized intersection			X
Ben-Yakov and Craus (*93*)/Craus and Mahalel (*94*)	–38	–	LTL installation			X
McFarland (*83*)	–70	–	LTL with curbed median	X		
	–65	–	LTL with curbed median	X		
	–60	–	LTL with curbed median		X	
	–15	–	LTL with painted median	X		

TABLE 6. Summary of research results concerning the safety effectiveness of installing left-turn lanes [adapted from Harwood et al. (*81*)] (Continued)

Source	Percentage change in accident frequency for addition of a LTL		Conditions/comments	Area type		
	Total intersection accidents	Left-turn accidents		Urban/suburban	Rural	Combination or unknown
	–30	–	LTL with painted median	X		
	–50	–	LTL with painted median		X	
	–36	–	Signalized intersection with LTL and exclusive phase			X
	–15	–	Signalized intersection with LTL but no exclusive phase			X
Foody and Richardson (*98*)	–38	–	Signalized intersection			X
	–76	–	Unsignalized intersection			X
Dale (*103*)	–20	–	Two-lane highway intersection; installation of signal with LTL		X	
Lacy (*85*)	–35	–	Installation of LTL with other improvements	X		
Tamburri and Hammer (*95*)/Wilson et al. (*96*)	–18	–	Unsignalized intersection			X

In their investigation of geometric design improvements for at-grade intersections, Harwood et al. (*81*) investigated the safety effectiveness of right-turn lanes. Data were collected at 100 intersections where a right-turn lane had been installed and 100 intersections that exhibited similar characteristics to the improved sites but did not have right-turn lanes installed. Harwood et al. concluded that right-turn lanes are effective in improving safety at signalized and unsignalized intersections in both urban and rural areas. Installation of a single right-turn lane on a major-road approach would be expected to reduce total accidents at urban signalized intersections by 4 percent and total intersection accidents at rural unsignalized intersections by 14 percent. Right-turn lane installation reduced accidents on individual approaches to four-leg intersections by 18 percent at urban signalized intersections and by 27 percent at rural unsignalized intersections. Limited results were found for right-turn lane installation at three-leg intersections. Installation of right-turn lanes on both major-road approaches to four-leg intersections would be expected to increase, but not quite double, the resulting effectiveness measures for total intersection accidents. Table 7 presents the recommended AMFs for the installation of right-turn lanes on major-road approaches to both urban and rural intersections.

TABLE 7. AMFs for right-turn lanes at urban and rural intersections (*81*)

Intersection traffic control	Number of major-road approaches on which right-turn lanes are installed	
	One approach	Both approaches
STOP sign[a]	0.86	0.74
Traffic Signal	0.96	0.92

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

Bauer and Harwood (*86,87*) developed several statistical models to predict accidents for at-grade urban and rural intersections. The models for urban, four-leg and three-leg, stop-controlled intersections indicated that accidents increase when right-turn lanes are present. Similarly, the presence of right-turn channelization at urban, four-leg, signalized intersections is expected to increase accidents. On the other hand, the models for rural, four-leg, stop-controlled intersections indicate that accidents decrease when right-turn lanes are present. At rural, three-leg, stop-controlled intersections, total intersections accidents are expected to decrease with the presence of right-turn lines, while fatal and injury accidents are expected to increase with the presence of right-turn lanes.

Dixon et al. (*104*) analyzed the crash history at 17 signalized intersections with various right-turn treatments in Cobb County, Georgia, to identify the effects of those right-turn treatments on right-turn crashes. The intersections were located on both major and minor urban arterials. The analysis was based strictly upon crash frequencies over a two-year period and did not include exposure data related to traffic volumes. Dixon et al. noted the following general findings, indicating that they merit future research:

- The use of a traffic island appears to reduce the number of right-angle crashes
- The addition of an exclusive right-turn lane appears to correspond to elevated sideswipe crashes

- The addition of an exclusive lane on the cross street for right-turning vehicles (i.e., an acceleration lane) does not appear to reduce the number of rear-end crashes when no additional control is implemented

In another study focusing on the safety of rural two-lane highways, Vogt and Bared (*105*) modeled accidents for three-leg unsignalized intersections and found that the presence of a right-turn lane was associated with an increase in intersection-related accidents by 27 percent.

In developing guidelines for channelized right-turn lanes at unsignalized intersections on rural two-lane highways, McCoy et al. (*106*) evaluated the safety effects of channelized right-turn lanes. An analysis of the accident history at 89 rural intersections with and without channelized right-turn lanes over a five-year period found no effect of channelized right-turn lanes on the frequency, severity, or types of accidents that occur on approaches to unsignalized intersections. Thus, based upon the accident analysis, it was concluded that channelized right-turn lanes do not provide the road user with any safety benefits or disbenefits. There are no comparable findings available for channelized right turns on urban arterials.

Finally, Staplin et al. (*107*) conducted an accident analysis to examine the problems that older drivers have in intersection areas. Approximately 700 accident records were reviewed during the analysis. In general, the accident analysis did not find right-turn lanes to be a safety issue for older drivers.

Channelization

Four functional objectives form the basis for channelization design concepts:

- Limiting the points of conflict
- Limiting the complexity of the conflict area
- Limiting the conflict frequency
- Limiting the conflict severity

A variety of measures such as designation and arrangement of traffic lanes, traffic islands, median dividers, and various signs, signals, and markings may be used for channelization purposes.

In general, studies indicate that channelization improves safety in urban and suburban areas. David and Norman (*89*) found that raised pavement markings tended to decrease accidents, especially at three-leg intersections. Exnicios (*108*) determined that several safety measures, including channelization, resulted in a 31 percent reduction in total accidents (over two years), a 58 percent reduction in total accidents (over one year), and a 100 percent reduction in total accidents (over 26 months) at several suburban intersections located in or near several metropolitan areas. Rowan and Williams (*109*) found accident rates, personal injuries, and rear-end type accidents were reduced due to the introduction of channelization at intersections in northwest Houston.

In studies for which the area type could not be determined or where the area type was most likely rural, Washington et al. (*110*), Forrestel (*111*), and Templer (*112*) found that intersection approaches with raised medians had lower accident rates than intersection approaches without raised medians.

Intersection Skew Angle

The angle between the legs of an intersection, particularly whether the legs intersect at a right or an oblique angle, has long been considered to affect the safety performance of the intersection. Several studies have investigated the safety impact of angle of intersection, but all of the studies were conducted in rural areas. McCoy et al. (*113*) found that accidents at rural two-way stop-controlled intersections increase with increasing skew angle. Hanna et al. (*114*) found a difference in the safety performance between three-leg T intersections and three-leg Y intersections, which represents an effect of the angle of the intersection. Harwood et al. (*81*) developed AMFs for intersection skew angle as part of algorithms to predict the expected safety performance of rural two-lane highways. For a three-leg stop-controlled intersection, the AMF was calculated as:

$$\text{AMF} = \exp(0.0040\ SKEW) \tag{3}$$

For a four-leg stop-controlled intersection, the AMF was calculated as:

$$\text{AMF} = \exp(0.0054\ SKEW) \tag{4}$$

where:

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

Sight Distance

Sight distance is the distance ahead or along an intersecting roadway that a driver can see from any location on the roadway. Provision of adequate sight distance is fundamental to the design of roads and intersections for safe operation. Three types of sight distance are particularly critical to the safe operation of at-grade intersections: intersection sight distance, stopping sight distance, and sight distance to traffic control devices. Several studies have addressed the safety effects of sight distance at intersections, but only one study addressed sight distance in the urban environment.

David and Norman (*89*) studied the safety effects of intersection sight distance, based upon data from urban intersections where foliage and buildings obstructed the view of the intersections. In general, David and Norman found that within specific ADT levels, the reduction in accident experience from a sight distance improvement was highest for intersection approaches whose initial sight distance was lowest. In other studies of intersection sight distance in rural environments, Hanna et al. (*114*) found that intersections with poor sight distance had an observed accident rate of 1.33 accidents per million entering vehicles, where intersections as a whole had an accident rate of 1.13 accidents per million entering vehicles. Mitchell found that total intersection accidents were reduced by 67 percent when intersection sight distance obstructions were removed.

In a study of stopping sight distance, Fambro et al. (*115*) found that accident rates were high for intersections located on crest vertical curves with limited sight distance.

No studies were found on the safety effects of limited sight distance to traffic control devices.

The expert panel of safety researchers evaluated the effects of intersection sight distance on intersection-related accidents (*7*). The AMFs, established by the expert panel for intersection sight distance at intersections with stop control on the minor legs, are as follows:

- 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

Sight distance in a quadrant of an intersection is considered limited if the available sight distance is less that the sight distance specified by AASHTO policy for a design speed of 20 km/h (12 mph) less than the major-road design speed and the sight distance restrictions are due to roadway alignment and/or terrain.

Approach Width

The width of an intersection approach includes the combined widths of the approach lanes, and in some cases, the width of the shoulder. Bauer and Harwood (*86,87*) and Lacy (*85*) found that increasing the approach width to an intersection was associated with reduced the accident rate along the approach. Bauer and Harwood (*86,87*) found that increasing the lane width at both unsignalized and signalized intersections in urban areas tends to decrease accidents, while at rural unsignalized intersections increasing the shoulder width tends to decrease accidents. Lacy (*85*) found that widening the approaches, combined with other safety improvements, decreased accident frequency by 35 percent and accident severity by 80 percent. On the other hand, David and Norman (*89*) did not find any evidence that incremental changes in lane or shoulder width near intersections affects accident rates.

Number of Approach Lanes

The number of lanes on an intersection approach is determined primarily by traffic demand and the desired level of service. Intuitively, one might assume that the number of accidents is proportional to the number of lanes (i.e., as the number of lanes increases so would the total number of accidents, since the potential number of conflicts would appear to increase.). However, at unsignalized intersections in both urban and rural areas, Bauer and Harwood (*86,87*) found that accidents tended to be higher on facilities with one approach lane and accidents tended to be lower at intersections with two or more lanes. The opposite appears to be the case for urban, four-leg, signalized intersections. David and Norman (*89*) also indicated that accident frequencies can be reduced by adding through lanes at urban/suburban intersections with total entering volumes less than 10,000 veh/day. It should be noted that with a demand-related design parameter such as number of lanes, it is difficult to assess directly whether any observed safety effects are due to the number of lanes or to the traffic volume on the approach.

Median Type and Width

The width of a divided highway median influences the safety performance of intersections on that highway. At both unsignalized and signalized intersections in urban and suburban areas, Harwood et al. (*99*) found that accident frequencies tended to increase with increasing median width. In contrast, accident frequencies at rural four-leg signalized intersections decrease as median width increases. Similar results for rural divided highway intersections were found by Priest (*116*) in that intersection accident frequencies decrease as the median with increases, except at very low volumes. Van Maren (*117*) did not find any statistically significant relationship between median width and intersection accident rate.

Pedestrian Facilities

Along urban and suburban arterials, pedestrian safety at intersections is an important issue. The pedestrian safety performance of an intersection is most directly related to the geometric design of each location; with the required crossing distance being the most significant design feature. No studies have been found that have used crash data to document the pedestrian safety implications of crossing distance and lane width. However, it may be reasoned that as crossing distances increase, pedestrian exposure time to motor vehicle traffic increases, increasing the potential of vehicle-pedestrian conflicts, and lane width can both directly and indirectly affect pedestrian crossing distance.

Several studies have been conducted to determine at what width the intersection crossing distance becomes excessive for pedestrians. The *Florida Pedestrian Planning and Design Handbook* (*118*) recommends medians at intersections whenever the crossing distance exceeds 18 m (60 ft) to provide a refuge for slow or late crossing pedestrians. This implies the maximum desirable pedestrian crossing distance is 18 m (60 ft). Similarly, Pietrucha and Opiela (*119*) recommended that pedestrian refuge islands be provided when crossing distances exceed 23 m (75 ft). Many of the countermeasures recommended to improve pedestrian safety at intersections involve reducing crossing distances, which in some cases could involve narrowing lanes.

Crossing intersections is particularly difficult for pedestrians with vision impairments. These difficulties arise because of quieter cars, right turn on red (which masks the beginning of the through phase), continuous right-turn movements associated with channelized right turns, complex intersection geometry, wide streets, and complex signal operations.

Bicycle Facilities

There are several key factors that impact the safety of bicyclists at intersections: bicycle and motor vehicle weaving, lane widths, providing bicycle facilities through the intersection proper, and bicyclists making left turns. For intersections that include right-turn lanes or channelized right-turn lanes, there appears to be an inherent risk to bicyclists because motor vehicles entering the turn lane must weave across the path of bicycles traveling straight through the intersection, but no studies based upon crash history are available to support this presumption. This same type of conflict also occurs between through bicyclists and right-turn vehicles at conventional intersections.

One of the most difficult maneuvers facing bicyclists at intersections is making a left turn at four-leg intersections. Because bicycle users are often to the right of through traffic as they approach the intersection, it is difficult to weave across these through moving vehicles in order to execute their left turn. Research was conducted by Hunter (*120*) that investigated the use of "bike boxes" which are right-angle extensions to bike lanes at the head of the intersection that allow left-turning bicyclists to move to the head of the traffic queue on red traffic signals and then proceed in front of vehicle traffic when the traffic signal changes to green. Hunter noted that the rate of conflicts between bicycles and motor vehicles changed little with the installation of the bike boxes and that no conflicts took place while using the bike box as intended.

The amount of lane width or right-of-way space provided for bicyclists through intersections influences bicyclists' perceived level of safety. The overall conclusion of research conducted by Hunter et al. (*121*) was that both wide curb lane and bicycle lane facilities can improve the safety of bicycle traffic because of the space given for their exclusive use. As these facilities approach intersections there are no universal standards as to how they should be treated through the intersection proper; yet it can be reasoned that providing these continued facilities for bicyclists would enhance the intersections safety performance.

HSM USER SURVEY

A survey was conducted as part of the research to obtain assessments from potential HSM users concerning the following issues as related to the HSM chapter on urban and suburban arterials:

- usefulness of potential HSM applications
- current safety prediction methods
- priorities for candidate input variables to the HSM safety prediction methodology
- priorities for candidate output variables for the HSM safety prediction methodology (i.e., safety measures of effectiveness)

- data availability from specific highway agencies for the research

The survey questionnaire used to address these issues and the results obtained are presented in Appendix A. These survey results were used in planning the structure of the HSM methodology and the work plan for its development.

The mailing list for the survey included:

- 50 state highway agencies
- 100 local highway agencies
- 100 MPOs
- 28 TRB Task Force members

Thus, a total of 278 survey questionnaires were mailed.

Tables 8 and 9 summarize the number of survey responses received from various agencies. Of the 278 surveys that were mailed out, 109 responses were received, for an overall response rate of 39 percent. The highest response rate was received from state highway agencies (74 percent), followed by TRB task force members (68 percent), MPOs (33 percent), and local agencies (20 percent). Overall, the responses received include representatives from 41 state highway agencies, 23 local highway agencies, 33 MPOs, three other government agencies, five consulting firms, and four university/research agencies.

TABLE 8. Response rate for the HSM User Survey

Agency/organization type	Number of questionnaires mailed	Number of responses received	Response rate (%)
State highway agencies	50	37	74.0
Local highway agencies	100	20	20.0
MPOs	100	33	33.0
TRB Task Force	28	19	67.9
TOTAL	278	109	39.2

TABLE 9. Types of organizations responding for the HSM User Survey

Agency/organization type	Number (percentage) of responses
State highway agency	41 (37.6)
Local highway agency	23 (21.1)
MPO	33 (30.3)
Other Government Agency	3 (2.7)
Consulting firm	5 (4.6)
University/research agency	4 (3.7)
TOTAL	109

NOTE: Number of responses for some agency/organization types exceeds the value shown in Table 12 because some TRB Task Force members also represent that agency/organization type.

Through this report, we have referred to the survey as the HSM user survey. The state highway agencies, local highway agencies, and MPOs to whom the survey was sent all represent potential users of the HSM methodology. While the TRB Task Force members have specific responsibilities for guiding HSM development, they are also potential HSM users. All of the tables in Appendix A show the responses from each group of survey respondents separately, since each group has different responsibilities and perspectives.

Survey respondents were asked to rate the usefulness of six potential applications of a quantitative safety prediction procedure for urban and suburban arterials. Each group of respondents indicated that forecasting the safety effectiveness for a proposed improvement project on an existing arterial was the most important application. The other two applications of the HSM methodology that were most frequently cited by users were forecasting the future change in safety performance that may occur on an existing facility as traffic volumes grow and forecasting the effect on safety of new driveways or new development that may be proposed along an existing arterial (i.e., for development impact studies or driveway permit requests).

Survey respondents were also asked whether they were currently using, or developing, any methods to predict or estimate the safety performance of urban and suburban arterials. Only 4 percent of responding agencies indicated that they currently employ any sort of safety prediction methodology. However, the general lack of current safety prediction methodologies indicates a definite need for the HSM.

Detailed ratings and rankings of candidate input variables and suggestions for additional input variables are presented in Appendix A. The input variables included in the HSM methodology are identified in Chapter 7 of this report.

The output variables or safety measures of effectiveness to be estimated by the safety prediction methodologies in the first edition of the HSM, as identified by the TRB Task Force, are:

- annual accident or crash frequency for a roadway segment or intersection
- crash severity distribution (percentage of crashes by severity level)
- crash type distribution (percentage of crashes by collision type)

A number of additional output variables were suggested by survey respondents. All of the potential output variables are identified in Appendix A and specific output variables that were suggested with some frequency are discussed in Chapter 3 of this report.

Survey responses concerning data availability from highway agencies are presented in Appendix A. Follow-up contacts with the most promising agencies were made to select the highway agencies whose data were used to assemble the safety database used in the research.

CHAPTER 3

RECOMMENDED STRUCTURE FOR SAFETY PREDICTION METHODOLOGY

The key elements of the structure for the safety prediction methodology that will be developed for urban and suburban arterials include:

- potential applications or uses of the methodology
- input variables
- output variables (safety measures of effectiveness)
- arterial components to be modeled separately
- prediction methodology for each arterial component

Each of these elements is discussed below.

POTENTIAL APPLICATIONS OR USES OF THE METHODOLOGY

The specific applications that the HSM safety prediction methodology should be designed to address were assessed in the early stages of the research. These potential applications include:

- estimating the current safety performance for an existing arterial for which accident history data are not available or are not considered reliable
- estimating the safety performance for an existing arterial combining model predictions and actual, reliable accident history data
- forecasting the future change in safety performance that may occur on an existing facility as traffic volumes grow
- forecasting the safety effectiveness for a proposed improvement project on an existing arterial, including major reconstruction, access management, or signal enhancement projects
- forecasting the safety performance of a new arterial that has not yet been constructed
- forecasting the effect on safety of new driveways or new development that may be proposed along an existing arterial (e.g., for development impact studies or driveway permit requests)

Each of these applications will impose slightly different demands on the safety prediction methodology and require slightly different capabilities; some applications are more suited than others for planning, design, and operational analyses.

HSM users were asked in the survey conducted in Task 3 of the research to rank these applications in order of importance. The survey results are presented in Table A-3 in Appendix A of this report. The potential HSM applications, in descending order of importance, giving equal weight to the responses from each of the four categories of survey respondents, are as follows:

1. forecasting the safety effectiveness for a proposed improvement project on an existing arterial, including both major reconstruction and access management projects
2. forecasting the effect on safety of new driveways or new development that may be proposed along an existing arterial (e.g., for development impact studies or driveway permit requests)
3. forecasting the future change in safety performance that may occur on an existing facility as traffic volumes grow
4. estimating the safety performance for an existing arterial combining model predictions and actual, reliable accident history data
5. forecasting the safety performance of a new arterial that has not yet been constructed
6. estimating the current safety performance for an existing arterial for which accident history data are not available or are not considered reliable

The second and third ranked HSM applications were rated nearly equally, with Application 2 rated higher by state highway agencies, local highway agencies, and the TRB Task Force and Application 3 ranked higher by MPOs.

It will be desirable for the HSM methodology to serve as many of the applications listed above as possible but, clearly, the highest priority should be assigned to estimating the safety effects of proposed improvement projects (Application 1). It is likely that any methodology that serves this application well would also be capable of meeting the needs of Applications 3 through 6. It would also be desirable if the methodology developed were also capable of serving the second priority, forecasting the effect on safety of new driveways or new development that may be proposed along an existing arterial (Application 2).

CANDIDATE INPUT VARIABLES

The survey of potential HSM users conducted as part of the research asked respondents to rate the importance to their agency of various candidate input variables for the HSM safety prediction methodology for urban and suburban arterials. Tables 10 and 11 present lists of the candidate input variables included in the survey for roadway segments and intersections, respectively. The candidate input variables were rated by survey respondents on a scale from 1 to 5 for their potential inclusion in the safety prediction methodology, with 1 representing the lowest priority and 5 representing the highest priority. The complete results of the survey, including the ratings of candidate input variables, are presented in Appendix A of this report. The rankings of candidate input variables by the survey respondents were one factor in recommending variables for data collection for the project database in Chapter 4 of this report. The tables of candidate input variables for specific roadway segment and intersection types in Chapter 4 show the mean ratings and rankings for those variables from the HSM user survey.

TABLE 10. Candidate input variables for roadway segments included in HSM User Survey

Bicycle facilities
Bicycle volumes
Delineation
Design or posted speed
Grades
Horizontal curves
Lane widths
Lighting
Median type
Median width
Number and type of driveways
Number and type of median openings
Number of through lanes
Older drivers/driver population characteristics
One-way vs. two-way operation
Pavement friction
Pedestrian facilities
Pedestrian volumes
Presence of curb parking
Presence of frontage roads
Presence of reversible lanes
Roadside design/clear zones/roadside objects
Segment length
Shoulder width/curb type
Spacing between driveways
Spacing between signals
Speed variance
Traffic volume (AADT) (veh/day)
Traffic volume in peak period (veh/h)
Traffic volumes for individual driveways
Transit facilities
Vehicle mix (e.g., percent trucks)
Vehicle speed in off-peak periods
Vehicle speed in peak periods
Vertical curves

TABLE 11. Candidate input variables for intersections included in HSM User Survey

Approach speed in off-peak periods
Approach speed in peak periods
Bicycle facilities
Bicycle volumes
Curb parking on approaches
Grade of approaches
Horizontal alignment of approaches
Intersection skew angle
Lane widths on approaches
Level of Service (LOS)
Lighting
Median type/presence of median
Number and length of added through lanes
Number of intersection legs
Number of through lanes on approaches
Older drivers/driver population characteristics
Pedestrian facilities
Pedestrian volumes
Presence of median refuge area for pedestrians
Presence of right-turn lanes
Presence/number of left-turn lanes
Shoulder/curb type on approaches
Shoulder/curb width on approaches
Signal phasing (e.g., left-turn phasing)
Signal timing
Signal visibility
Spacing between intersections and nearby driveways
Traffic volumes (AADT) for major- and minor-road legs (veh/day)
Traffic volumes in peak period (veh/h)
Transit facilities
Type of left-turn channelization (painted vs. curb)
Type of traffic control
Vehicle mix (e.g., percent trucks)

It is not clear that all of the variables identified in Tables 10 and 11 can be included explicitly in the HSM methodology. Variables included in the methodology would need to have known relationships to safety or relationships that could be established as part of the research. Most of these variables were included in the data collection for the project database so that their appropriateness for inclusion in the methodology could be examined. Variables not included in the methodology for the first edition of the HSM can be considered further for later editions.

There are limited data available for some variables, such that their consideration would have required a data collection effort beyond the scope of the current study. The variables that it was decided could not be addressed in the methodology for urban and suburban arterials in the HSM first edition were:

- Adjacent land use—it was decided not to include an explicit variable for adjacent land use because land use is addressed through the driveway type variable introduced in Chapter 5.
- Bicycle facilities—the overall level of vehicle-bicycle collisions is addressed in the methodology and data on bicycle facilities have been collected, but it was decided that the resources of the project were not sufficient to model the safety effects of specific design features on bicycle facilities.
- Driveway volumes—the collection of traffic volume data in the field at individual driveways was considered, but was found to be beyond the resources of the project.
- Multiple turn lanes—multiple turn lane (i.e., double right-turn lanes and double or triple left-turn lanes) have been noted in the field where present. However, collection of sufficient data to evaluate their safety effects would be beyond the resources of this project.
- Presence of channelized right-turn lanes—the evaluation of the safety effectiveness of channelized right-turn lanes would have required data collection beyond the scope of the current project. Channelized right-turn lanes are being considered in ongoing research in NCHRP Project 3-72 and 3-78 and further research on this topic is under consideration. If the results of these research projects appear promising, they should be considered for inclusion in the HSM methodology.
- Roundabouts—a decision was reached not to collect data on roundabouts. The safety effectiveness of roundabouts is being investigated in NCHRP Project 3-65 and results from that project might be considered for incorporation in the HSM methodology when that project is complete.
- School zones—no safety effectiveness estimates for school zones are available; it is recommended that this issue be addressed in the second edition of the HSM or beyond.
- Sight distance—the collection of data on intersection sight distance, particularly for turning and crossing maneuvers at unsignalized intersections was considered, but was found to be beyond the resources of the project.

- Traffic volumes by time of day and turning movement counts—data on traffic volumes by time of day on roadway segments and at intersections and turning movement counts at intersections were not available for most of the study sites. The collection of these data in the field was considered, but was found to be beyond the resources of the project.

The best variable representing daily traffic volumes for use in the research would be the annual average daily traffic volume (AADT), which is typically derived from combining counts made at several times of the year or by adjusting a one-time count with seasonal factors from a continuous counting station. However, AADT data were not always available for the sites considered in this study and will not always be available to the users of the safety prediction methodology developed here. Therefore, in the methodology presented in this report, we have consistently referred to the average daily traffic volume (ADT), not the AADT. The ADT data used in this research are equivalent in quality to the data that are likely to be used in implementing the safety prediction methodology.

Modeling of the safety effects of pedestrian facilities was initially considered to be beyond the resources of this project. However, a decision was reached by NCHRP to expand the project scope to include the development of a pedestrian safety methodology. That work is currently underway and will be added to the methodology for the first edition HSM when complete.

OUTPUT VARIABLES

The output variables or measures of effectiveness for the safety prediction methodology will include the three safety measures of effectiveness that have been approved by the TRB Task Force. These are:

- expected total accident or crash frequency
- expected crash frequency distribution by crash severity level
- expected crash frequency distribution by crash type

The output variables or measures of effectiveness are dependent variables whose values would be estimated by the safety prediction methodology.

The survey asked potential HSM users whether there are other output variables or measures of effectiveness that should be considered. The full set of responses to this survey question is presented in Table A-11 in Appendix A of this report. That table shows that there were four additional output variables that were identified as desirable output variables or measures of effectiveness based on the survey responses. These are:

- expected accident/crash rate
- combination of expected accident/crash severity and type distribution
- expected accident/crash distribution by time of day

- expected accident/crash reduction factors based on improvement type (i.e., AMFs)

Each of these additional output variables is discussed briefly below.

Accident/Crash Rate as an Output Variable

Nearly 13 percent of respondents recommended that accident/crash rate be considered as an output variable. The traditional accident rate measures to be considered are:

- accidents per million veh-mi of travel for roadway segments
- accidents per million entering vehicles for intersections

In the past, accident rate measures like those shown above were used extensively in safety modeling. The use of accident rate measures has been largely discontinued for two reasons:

- accident rate measures cannot be reliably predicted because they incorporate the implicit assumption that accident frequency varies linearly with traffic volume when, in fact, accident frequency in most cases has a nonlinear relationship with traffic volume
- accident rate measures, even if derived correctly using a nonlinear relationship between accident frequency and traffic volume, appear to suggest to naïve users that the change in accident frequency resulting from a given change in traffic volume will be linear

Based on the first bullet item presented above, it is recommended that the dependent variable for safety prediction models should be accident frequency, not accident rate, and that traffic volume be considered as an independent variable in the prediction model rather than as part of the dependent variable. With this approach, the effect of traffic volume on accident frequency can take any functional form that best suits the data; in most cases, this functional form will be nonlinear.

Given that the safety prediction models will estimate accident frequency, as recommended by the TRB Task Force, it is still possible to express the predicted accident frequency as an accident rate by dividing the accident frequency by an exposure measure such as million veh-mi of travel for segments or million entering vehicles for intersections. There is both an advantage and a disadvantage to doing this. The advantage is that the accident rate, expressed in this manner, represents something useful and important—the risk to an individual motorist of traveling through the segment or intersection in its current condition and traffic volume level. Accident rates can, thus, be used to compare the risks of travel between sites to determine the sites at which travel is most safe and least safe. The disadvantage is that, as noted above, accident rate measures may suggest to naïve users that a change in traffic volume (e.g., traffic volume growth over time) will have a linear or proportional effect on accident frequency; there is clear evidence that such effects are usually nonlinear.

On balance, it is recommended that the HSM provide procedures to express the safety prediction results as accident rates, as well as accident frequencies, so that the relative risks of travel through various sites can be explicitly compared, but that a caveat against assuming that accident frequency varies linearly with traffic volume be stated clearly at appropriate locations in the HSM.

To summarize, we recommend the following policy for consideration by the NCHRP project panel and the TRB Task Force:

- all HSM safety prediction models should predict accident frequencies, not accident rates
- where appropriate, the HSM should provide procedures to enable users to compute accident rates per million veh-mi for segments and per million entering vehicles for intersections
- wherever procedures to compute accident rates are provided, the HSM should state that accident rates should be used to compare the relative risks of travel through specific segments and intersections, but should not be used to predict the effect of changes in traffic volume on accident frequency. In other words, it should be clearly stated that it should not be presumed that the accident rate will remain unchanged if the traffic volume changes.

Accident/Crash Distribution by Time of Day

Consideration was given to whether safety predictions should be made for a full 24-hr day or for separate periods within the day. Two distinct sets of contrasting periods with a 24-hr day are of potential interest:

- Peak traffic periods vs. off-peak periods
- Daytime vs. nighttime periods

Separate modeling of peak and off-peak periods requires accident and traffic data to be available separately for these periods. Accident records normally indicate the time of day at which the accident occurred, so separating accidents by time of day is not typically an issue, although the specific times considered to be the peak period may vary between locations and between metropolitan areas. However, most sites with available ADT estimates do not have peak period traffic volumes available. Furthermore, even where traffic volume data for the peak period are available, modeling to predict peak-period accident frequency is not always successful because of the smaller sample size of accidents. Initial efforts in this project did not indicate that satisfactory models of peak vs. off-peak traffic periods could be developed without extensive additional data collection. Therefore, consideration of this issue has been deferred to a later edition of the HSM.

Separate accident predictions for daytime and nighttime accidents are useful particularly in estimating the potential effectiveness of lighting as a countermeasure. The safety effectiveness of lighting would typically be expressed as a combination of the expected annual nighttime accident frequency for a location and the AMF for lighting expressed as a proportional change in nighttime accident frequency. Formal models for predicting nighttime accident frequencies would require a substantial number of locations for which hourly traffic volume counts are available. Hourly counts are needed because the hours during which daytime and nighttime conditions occur vary by season of the year (and by latitude). It does not appear feasible to conduct an extensive program of hourly traffic counting in Project 17-26, so the research team recommends a simpler approach to estimation of nighttime accident frequencies based on proportions of daytime and nighttime accidents from a substantial number of similar sites. The distribution of daytime vs. nighttime accidents is used in conjunction with the AMF for lighting in the HSM methodology.

Accident/Crash Reduction Factors Based on Improvement Type

Accident/crash reduction factors (i.e., AMFs) for specific improvement types will have a key role in the safety prediction methodology, but they are not output variables. However, if the safety prediction methodology is applied to determine the accident frequency for two alternatives, one with and one without a specific improvement, then the difference in accident frequency, representing the expected safety benefit of the improvement, can be considered as an output of the methodology.

ARTERIAL COMPONENTS TO BE MODELED SEPARATELY

It is likely that the most effective approach to development of a safety prediction methodology will be to model the safety performance of different arterial components separately. The safety prediction algorithm for rural two-lane highways, used in the HSM prototype chapter, used separate models for roadway segments and at-grade intersections (*2*). The overall safety prediction for a highway segment was obtained by making separate safety predictions for each individual roadway segment and each at-grade intersection with the analysis boundaries or project limits of interest. The safety prediction modeling for at-grade intersections included accidents that occurred at the intersection (within the curb line limits) and intersection-related accidents that occurred within 76 m (250 ft) of the intersection (which included nearly all intersection-related accidents on rural two-lane highways). Roadway segments were defined as continuous segments that were homogeneous with respect to the model input variables. The safety prediction modeling for roadway segments included all accidents not attributable to a specific intersection.

The concept of modeling and predicting accidents separately for roadway segments and intersections that was used for rural two-lane highways has also been applied to urban and suburban arterials. The input variables that are potentially applicable to roadway segments and intersections are quite distinct. Thus, it seems appropriate to use the same component-based or building-block approach for urban and suburban arterials as was used for rural two-lane highways. The distance of 76 m (250 ft) used to select intersection-related accidents appears to be appropriate for unsignalized intersections and most signalized intersections on urban and suburban arterials. In jurisdictions that routinely experience vehicle queues longer than 76 m (250 ft) at signalized intersections, an adjustment to the predicted accident frequencies can be made by way of the calibration process.

A key difference between the rural two-lane highway methodology and the urban and suburban arterial methodology is the treatment of driveway accidents. In the rural two-lane highway methodology, the influence of driveways was handled through an AMF representing the safety effect of driveway density (driveways per mile). No distinction between types of driveways was made in the rural two-lane highway methodology. Driveway-related collisions are a more substantial percentage of the accident frequency of urban and suburban arterials than for rural two-lane highways. For this reason, the effects of individual driveways are considered separately and the overall driveway-related accident frequency is the sum of the collision frequencies for the individual driveways. The collision frequencies for individual driveways are a function of the type of land use served by the driveway and the roadway type on which the driveway is located.

For driveways on divided arterials, we hope to include a factor that distinguishes between driveways located at a median opening and driveways not located at a median opening (i.e., right-in, right-out operation). This factor is still under development.

SAFETY PREDICTION METHODOLOGY FOR EACH ARTERIAL COMPONENT

Two alternative modeling approaches were initially considered for development of the HSM methodology:

- safety prediction directly with a regression model
- safety prediction with a combination of a base regression model and AMFs, as was done for the IHSDM crash prediction model for two-lane highways and the HSM prototype chapter

Each approach and its strengths and weaknesses is described below.

The latter approach was selected as the preferred approach for the HSM methodology for urban and suburban arterials.

Safety Prediction Directly With a Regression Model

Safety prediction directly with a regression model is the traditional approach to safety modeling. Regression models for safety prediction typically have the following forms for roadway segments and intersections, respectively:

Roadway Segments

$$N_{br} = EXPO\ \exp(b_0 + b_1 X_1 + b_2 X_2 + \ldots + b_n X_n) \tag{5}$$

or

$$N_{br} = \exp(b_0 + b_1 \ln ADT + b_2 \ln L + b_3 X_3 + \ldots + b_n X_n) \tag{6}$$

where:

N_{br} = predicted accident frequency per year for a given roadway segment
EXPO = exposure in million veh-mi of travel per year = (ADT)(365)(L)(10^{-6})
ADT = average daily traffic volume (veh/day) for roadway segment
L = length of roadway segment (mi)
$b_0,\ldots,b_n$ = regression coefficients determined by model fitting
$X_1,\ldots,X_n$ = roadway segment characteristics

Intersections

$$N_{bi} = \exp(b_0 + b_1 \ln ADT_{maj} + b_2 \ln ADT_{min} + b_3 X_3 + \ldots + b_n X_n) \tag{7}$$

where:

N_{bi} = predicted accident frequency per year for a given intersection
ADT_{maj} = average daily traffic volume (veh/day) on the major road
ADT_{min} = average daily traffic volume (veh/day) on the minor road
$X_3,\ldots,X_n$ = intersection characteristics

The major advantage of regression models is that they are relatively easy to develop and apply. The major disadvantage of regression models is that their coefficients do not always represent the independent effects of the variables with which they are associated. Consider, for example, a term, b_2X_2, from Equation (5) above. If X_2 represents the lane width of a roadway section, then coefficient b_2 purports to represent the sensitivity of safety to changes in lane width. In some regression models, b_2 may, in fact, represent the sensitivity of safety to lane width well, but in other cases the value of b_2 may be influenced not only by lane width, but also by other variables, not included in the model, that are statistically correlated with lane width. Thus, if the presence of narrow lanes is correlated with the presence of narrow shoulders and poor roadsides, the value of the coefficient, b_2, may well overstate the actual effect of lane width on safety. Another disadvantage of regression model approaches is that the values of regression coefficients may vary substantially between jurisdictions.

Regression models often are good tools for predicting total accident frequency, although this may arise from multicollinearity between included and omitted variables. The weakness described above only applies to the use of individual regression coefficients to represent the independent effects of the specific variables with which they are associated, not the ability of the overall regression model to provide a good estimate of accident frequency for a particular facility. This weakness of regression models does not mean that they cannot be used, but only that they must be used cautiously. If a regression coefficient is used to represent the effect of, say, lane width, it is important that the effect implied by that coefficient be verified as reasonably in accordance with current knowledge about the safety effect of lane width.

Safety Prediction With a Combination of a Base Regression Model and AMFs

In the rural two-lane highway crash prediction algorithm developed for IHSDM and also used in the HSM prototype chapter, a new approach was developed to overcome the weaknesses of regression models discussed above. This approach combined a base regression model with a series of AMFs representing the effects of individual geometric design or traffic control features. This approach is represented by the following equation:

$$N = N_{base}\,(AMF_1\,AMF_2 \ldots AMF_n) \quad \textbf{(8)}$$

where:

N	=	predicted accident frequency per year for a given roadway segment or intersection
N_{base}	=	accident frequency per year predicted by base model for actual ADT under nominal or base conditions
$AMF_1,\ldots,AMF_n$	=	accident modification factors to convert accident frequencies from nominal or base conditions for specific site characteristics to actual conditions

In this approach, a regression model like Equations (5) through (7) is used as a base model. The model is used to predict the safety performance of a roadway or intersection with the same ADT as the site of interest, but with nominal or base values of other variables. For example, the lane width variable in the base model might be set equal to 3.6 m (12 ft), the shoulder width variable equal to 1.8 m (6 ft), and the driveway density variable set equal to zero (no driveways). This provides a reliable overall safety prediction for a nominal or "ideal" facility. Then, AMFs for geometric design and traffic control variables of interest are developed to represent the best-known effects of these variables. For example, the AMF for lane width would not be based on the regression coefficient for lane width in the base model, but rather would represent the known effect of lane widths from the best available studies. For example, the AMF for a 3-m (10-ft) lane width would be determined as the ratio between the accident frequency for 3-m (10-ft) lanes and the accident frequency for the nominal lane width of 3.6 m (12 ft), based on the results of the best available research. This ensures that the sensitivity of safety to lane width in the prediction methodology is based on the current state of knowledge about lane width and not on a regression coefficient that may not match the known sensitivity of safety to lane width.

Depending on the research on which they are based, AMFs may reflect an effect on total crash frequency or on a particular crash severity level or crash type. For example, Figure 3, presented in Chapter 2 of this report, illustrates the AMFs for shoulder width in the IHSDM crash prediction methodology. This AMF applies to single-vehicle run-off-road, multiple-vehicle same direction sideswipe, and multiple-vehicle opposite direction accidents (collectively known as related accidents), and not to total accident frequency. However, it can be converted to an AMF for total crash frequency if the proportion of these related accident types in total accidents is known. An AMF value greater than 1.0 indicates that a particular shoulder width is associated with an accident frequency higher than the nominal shoulder width of 1.8 m (6 ft). For example, the AMF of 1.30 for 0.6-m (2-ft) shoulders means that 0.6-m (2-ft) shoulders would be expected to experience 30 percent more accidents than 1.8-m (6-ft) shoulders. An AMF value less than 1.0 indicates that a particular shoulder width is associated with an accident frequency lower than the nominal shoulder width of 1.8 m (6 ft). For example, the AMF of 0.87 for 2.4-m (8-ft) shoulders means that 2.4-m (8-ft) shoulders would be expected to experience 13 percent fewer accidents than 1.8-m (6-ft) shoulders. These AMF apply to specific types of related accidents that are defined in Figures 1 and 3. It should also be noted that the AMFs shown for lane and shoulder widths in Figures 1 and 3 are applicable to rural two-lane highways and are not being proposed for application to urban and suburban arterials.

Ideally, AMFs should be based on the results of well-designed before-after studies. However, since only a limited number of well-designed before-after studies have been conducted, many AMFs are necessarily based on regression coefficients.

STATISTICAL MODELING APPROACH

Regression models were developed for use as base models for the HSM methodology. This section describes the statistical modeling approach that was used. The regression models developed with this approach are presented in Chapter 5 of this report.

Roadway Segment and Intersection Models

Roadway segment and intersection models for collision types other than driveway-related collisions were developed initially in the forms shown in Equations (6) and (7). Functional relationships between expected number of roadway or intersection accidents and explanatory variables, including ADT and roadway segment or intersection characteristics, were developed using a stepwise multiple regression approach, assuming a negative binomial (NB) error distribution of accident frequencies. Models were developed individually within each state and for combined states, separately for each roadway type, intersection type, and collision type. A backwards stepwise regression procedure was used to select those independent variables that contribute significantly to the model under consideration, starting with all variables, and excluding, one at a time, the variables for which the significance level was above 0.20. Other criteria used in the final variable selection included whether the statistical algorithm converged; whether the coefficient of a particular continuous variable was in the anticipated direction; and whether the effect of a categorical variable with ordinal levels was monotonic over the levels of that variable.

The concern about the direction of the effect for continuous or ordinal categorical variables is a key consideration in developing useful models for engineering applications. Regression analysis provides no assurance that the values of the fitted regression coefficients represent cause-and-effect relationships of specific independent variables to safety. Thus, each regression result was reviewed to assess whether the results obtained were consistent with existing knowledge about the effect of that variable. Anomalous relationships, such as coefficients with an opposite sign to that expected, can result when variables in the model correlate with variables not included in the model. Where such anomalous relationships were found, the variable in question was excluded from further modeling for the roadway or intersection type in question. This approach provided assurance that the resulting models were not only statistically significant, but were also meaningful in engineering terms.

In all models developed for nondriveway collisions on roadway segments, the coefficient for segment length [b_2 in Equation (6)] was assumed to be equal to 1.0. Previous modeling efforts for total accident frequencies on arterials have reported coefficients for segment length other than 1.0. Since there is no inherent reason that safety should not be proportional to length, such results are presumed to result from close spacing between intersections, which is correlated with short segment lengths, and from the presence of multiple driveways. In the present study, with all intersection- and driveway-related collisions removed from roadway segment modeling, there is no theoretical reason that segment length should not have a linear relationship to safety. Therefore, the coefficient for segment length was forced to be 1.0 in all cases.

To develop regression models for the data from individual states, the GENMOD procedure of SAS (*122*) was used to estimate model coefficients, over dispersion, the standard error of each estimate, and their significance level (p-value). This procedure fits a generalized linear model to the data by maximum likelihood estimation of the regression and over dispersion parameters; an NB error structure of accident counts is assumed. For each final model selected, a likelihood ratio test statistic, R^2_{LR}, was calculated (*123*). This statistic is a function of the log-likelihood of the full model (including the selected parameters) and that of the intercept-only model and sample size. As such, it is a measure, on a scale of 0 to 1, of how much better the full model is than the intercept-only model. Thus one looks for values above zero; naturally, higher values of R^2_{LR} indicate a greater improvement, although this statistic is constrained mathematically such that it cannot reach a value of 1. Thus, the definition of R^2_{LR} differs from the conventional definition of R^2. In ordinary least squares regression, R^2 represents the proportion of the variation in the dependent variable that is explained by the regression model. In the present study, R^2_{LR} represents the improvement of the model in comparison to an intercept-only model. Thus, even very low values of R^2_{LR} indicate that the model including one or more independent variables fits the data better than an intercept-only model. Thus, there is a good case for using NB models with low values of R^2_{LR} if the model includes effects of independent variables that make sense in engineering terms.

In developing regression models from the combined data from more than one state, a slightly different modeling approach was used. In this case, it was assumed that the two states provide a random sample of data from the pool of all states. Thus, state was included in the NB regression as a random factor rather than a fixed factor like other variables. This approach allows one to estimate the effect of each fixed effect while controlling for the random state effect. All other variable and model selection criteria were kept identical to those used in the individual state modeling approach.

To develop models from combined state data, the GLIMMIX procedure of SAS (*124*) was used to estimate model coefficients, over dispersion, the standard error of each estimate, and their significance level (p-value), assuming an NB accident distribution and a random error structure within and between states. All regression and over dispersion parameters were estimated by maximum likelihood method. Again, for each final model selected, a likelihood ratio test statistic, R^2_{LR}, was calculated. When estimating generalized linear mixed models, this statistic is a measure, on a scale of 0 to 1, of how much better the full model, including the random state effect, is than the intercept-only model. In some cases, the log-likelihoods of the full and intercept models were mathematically too close to compute an R^2_{LR} value during the iterative model process. In those cases and when the regression coefficients were sensible, then the full model was selected over the intercept-only model and an R^2_{LR} value could simply not be reported.

The HSM methodology was developed in the form illustrated in Equation (8). In this approach, all independent variables other than traffic volume were represented by AMFs whose development is discussed in Chapter 6 of this report. Therefore, to obtain a model appropriate to estimate N_{base}, each regression model was reduced to an ADT-only model by substituting nominal or base values of the other independent variables.

Driveway Models

Functional relationships between expected number of driveway-related collisions and explanatory variables, including ADT and roadway segment characteristics, were developed using a sequence of models as shown in Equations (9) through (13).

$$N_{brdway} = a\left(\frac{ADT}{10{,}000}\right)^b \tag{9}$$

$$N_{brdway} = a\left(\frac{ADT}{10{,}000}\right)^b L^c \tag{10}$$

$$N_{brdway} = a\left(\frac{ADT}{10{,}000}\right)^b n^c \tag{11}$$

$$N_{brdway} = a\left(\frac{ADT}{10{,}000}\right)^b \sum_{i=1}^{7} c_i n_i \tag{12}$$

$$N_{brdway} = a\left(\frac{ADT}{10{,}000}\right)^b e^{\left(\frac{ADT}{1000}\right)} \sum_{i=1}^{7} c_i n_i \tag{13}$$

where:

N_{brdwy}	=	predicted frequency per year of driveway-related collisions for a given roadway segment
n	=	total number of driveways on roadway segment
n_i	=	total number of driveways of type i on roadway segment
a, b, c, c_i, d	=	regression coefficients

The assumption was made that the accident counts in each of the five years come from a multinomial distribution with an NB error structure. Regression parameters and the over dispersion parameter were estimated by maximum likelihood. Thus, in all initial models, a separate intercept was estimated for each single year; these estimates were then simply averaged for the final models shown in Table 43 in Chapter 5.

The models represented by Equations (9) through (13) and an intercept-only model were applied to the yearly data, separately for each roadway type and collision type. A number of options were investigated and at each step, the log-likelihoods of two competing models were compared and the best model selected, assuming that the final model is meaningful in engineering terms. The various approaches include the following:

- In all regression models, driveway-related collisions are estimated relative to minor residential driveway-related collisions to express the main modeling idea that under identical conditions, a driveway of type i will have c_i times the number of collisions on a minor residential driveway (base case). Thus the coefficient of minor residential driveways is always set to 1.
- All models were restricted to having driveway coefficients, c_i, equal to 1 or higher. In those cases were the accident data were too sparse across the various cells defined by the model parameters (e.g., model did not converge), those coefficients were constrained to 1 and the remaining coefficients were estimated by maximum likelihood.
- The potential for a straight proportional effect of some parameters (e.g., ADT) on the number of collision was investigated; this was achieved by setting that parameter to 1. If setting a particular coefficient to 1 provided a better fitting model than estimating the coefficient, than the coefficient was set to 1.
- The additional effect of other parameters such as speed limit category, on-street parking, lane with (categorical), and shoulder width (categorical) was investigated. In most cases, no systematic pattern of difference between levels of a parameter were found; these parameters were therefore not included in any of the final models.

The final model form selected was of the type shown in Equation 12 since it provided some consistency across all roadway and collision types. Coefficients of that model could be set to 1, obtained through maximum likelihood estimation, or simply constrained to 1 due to sparse data.

CALIBRATION FACTORS FOR APPLICATION OF THE PREDICTION METHODOLOGY BY SPECIFIC HIGHWAY AGENCIES

The prediction methodology is likely to include regression models for use to directly predict safety performance of specific arterial components or as base models in a base-model-plus-AMF methodology. It is our intention that each regression model should be developed with the best available data set suited to that particular model. It might be desirable if each highway agency that uses the safety prediction methodology were to refit the models with their own data but, in most cases, this is unlikely to occur. It is unlikely that most agencies that want to use the model will have available the resources, expertise, and data to develop such models themselves. Therefore, most users of the safety prediction methodology can expect to be using regression models developed with data from other jurisdictions.

A calibration factor can be included in models like Equations (5) through (7) to allow a highway agency to compensate for the use of data from other jurisdictions in the development of the model. Such calibration factors are incorporated in the IHSDM crash prediction methodology for rural two-lane highways. Equation (8) can be recast as follows to incorporate a calibration factor:

$$N = N_{base}\, C\,(AMF_1\, AMF_2 \ldots AMF_n) \qquad \textbf{(14)}$$

where:

C = calibration factor to convert predicted accident frequency to local conditions

A calibration procedure is presented by Harwood et al. (*7*). A similar calibration approach used for rural two-lane highways is potentially applicable to urban and suburban arterials, as well. The calibration procedure should involve:

1. Selecting a set of urban and suburban arterials with specified characteristics in the jurisdiction of interest.
2. Applying the safety prediction model to those arterials to estimate the predicted accident frequency for a particular time period.
3. Obtaining actual accident history data for the selected sites for the same time period.
4. Determining the calibration factor as the ratio of the observed accident frequency divided by the predicted accident frequency.

The calibration procedure can be developed with different levels of detail depending upon which characteristics the user chooses to consider in Step 1. This allows the calibration procedure to be adapted to highway agencies with varying accident records systems. Consideration is currently being given in NCHRP Project 17-36 to developing a common calibration procedure that can be applied to all of the HSM prediction methodologies.

PREDICTIONS FOR ACCIDENT SEVERITY AND ACCIDENT TYPE

In the HSM methodology for urban and suburban arterials, base models are provided for total accidents, as well as components of total accidents by accident severity level. The models for individual accident severity levels address fatal-and-injury accidents and property-damage-only accidents. Since the models for total accidents, fatal-and-injury accidents, and property-damage-only accidents are developed separately, there is no assurance that the predicted accident frequencies for the two severity-level components will add up to the predicted total accident frequency. Therefore, the predicted value of total accident frequency is treated as the primary predicted value and the relative predicted values for fatal-and-injury and property-damage-only accidents are used to proportion the total accident frequency prediction into severity level components.

For roadway segments, accident frequency estimates are made for five accident types:

- multiple-vehicle nondriveway collisions
- single-vehicle accidents
- driveway-related collisions
- vehicle-pedestrian collisions
- vehicle-bicycle collisions

The same accident types are used for intersections, except that driveway-related collisions are not applicable for intersections and are omitted. Multiple-vehicle nondriveway collisions and driveway-related collisions are predicted by base regression models in most cases; in a few cases where single-vehicle accidents are sparse, base models could not be developed, and single-vehicle accidents are predicted by factors that represent the average proportion of single-vehicle accidents relative to multiple-vehicle nondriveway collisions. Vehicle-pedestrian and vehicle-bicycle collisions are, in all cases, predicted by factors that represent their average frequency in proportion to other accident types.

Tables are provided as part of the methodology to break down the multiple-vehicle nondriveway collisions by manner of collision and the single-vehicle accidents by object struck. These tables represent average proportions for each roadway type.

CONSIDERING OBSERVED ACCIDENT HISTORY DATA IN THE SAFETY PREDICTION METHODOLOGY

The safety prediction methodology should be capable of estimating the expected accident frequency for an urban or suburban arterial in the absence of any actual observed accident history data for that specific arterial roadway. This makes it possible to apply the safety prediction methodology to planned roadways that have not yet been constructed and to sites for which reliable observed accident history data are not available. However, when observed accident history data are available for the site in question, their use, together with the model predictions, results in a more reliable estimate of expected accident frequency than could be obtained with either the observed accident history data or the model predictions alone. The accepted statistical approach to combining observed accident history data and model predictions of accident frequencies is known as the Empirical Bayes (EB) procedure. If observed accident history data were used by themselves, they would be potentially biased due to regression to the mean; the EB approach compensates for regression to the mean and allows the two available pieces of information—the observed accident frequency and the predicted accident frequency—to be combined as a weighted average.

The EB approach has been developed to its current state by Hauer, as presented in his book, *Observational Before-After Studies in Road Safety* (*125*); the application of the EB approach to HSM safety prediction models is illustrated in the IHSDM crash prediction methodology for rural two-lane highways developed by Harwood et al. (*7*) and in the HSM prototype chapter. This same procedure, with minor changes, can be applied in the safety prediction methodology for urban and suburban arterials to ensure that full advantage is taken of the valuable information represented by the observed accident frequency for the site being analyzed.

Specifically, the expected accident frequency for any site can be determined by combining the predicted and observed accident frequencies in the following manner (*7*):

$$E_p = (w)\,N + (1\text{-}w)\,O \qquad \textbf{(15)}$$

where:

E_p = expected accident frequency based on a weighted average of N and O
w = a weight factor defined below in Equation (16)
N = number of accidents predicted by a safety prediction model such as Equation (9) during a specified time period
O = number of accidents observed during the same time period as N

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

$$w = \frac{1}{1 + k\,(N)} \qquad \textbf{(16)}$$

where:

k = over dispersion parameter of the accident prediction model

Because the HSM methodology presented in Chapter 7 of this report uses separate base models for multiple-vehicle nondriveway, single-vehicle, and driveway-related collisions, two additional issues must be considered. Furthermore, observed accident data are often available only for a combined set of roadway segments and intersections to which several different base models may apply. First, the over dispersion parameter, k, in the denominator of Equation (16) is not uniquely defined, because two or more base models with different over dispersion parameters must be considered. Second, it cannot be assumed, as is normally done, that the expected numbers of accidents for different collision types and different site types are statistically correlated with one another. Rather, an estimate of expected accident frequency should be computed based on the assumption that the various roadway segments and intersections 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:

$$N_{tot} = \sum_{j=1}^{5} N_{rmj} + \sum_{j=1}^{5} N_{rsj} + \sum_{j=1}^{5} N_{rdj} + \sum_{j=1}^{4} N_{imj} + \sum_{j=1}^{4} N_{isj} \tag{17}$$

$$O_{tot} = \sum_{j=1}^{5} O_{rmj} + \sum_{j=1}^{5} O_{rsj} + \sum_{j=1}^{5} N_{rdj} + \sum_{j=1}^{4} O_{imj} + \sum_{j=1}^{4} O_{isj} \tag{18}$$

$$N_{w0} = \sum_{j=1}^{5} k_{rmj} N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj} N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj} N_{rdj}^2 + \sum_{j=1}^{4} k_{imj} N_{imj}^2 + \sum_{j=1}^{4} k_{isj} N_{isj}^2 \tag{19}$$

$$N_{w1} = \sum_{j=1}^{5} \sqrt{k_{rmj} N_{rmj}} + \sum_{j=1}^{5} \sqrt{k_{rsj} N_{isj}} + \sum_{j=1}^{5} \sqrt{k_{rdj} N_{rdj}} + \sum_{j=1}^{4} \sqrt{k_{imj} N_{imj}} + \sum_{j=1}^{4} \sqrt{k_{isj} N_{isj}} \tag{20}$$

$$w_0 = \frac{1}{1 + \frac{N_{w0}}{N_{tot}}} \tag{21}$$

$$E_0 = w_0 N_{tot} + (1 - w_0) O_{tot} \tag{22}$$

$$w_1 = \frac{1}{1 + \frac{N_{w1}}{N_{tot}}} \tag{23}$$

$$E_1 = w_1 N_{tot} + (1 - w_1) O_{tot} \tag{24}$$

$$E_{comb} = \frac{E_0 + E_1}{2} \tag{25}$$

where:

N_{rmj} = Predicted number of multiple-vehicle nondriveway collisions for roadway segments of type j, j=1,..,5

N_{rsj} = Predicted number of single-vehicle collisions for roadway segments of type j
N_{rdj} = Predicted number of driveway-related collisions for roadway segments of type j
N_{imj} = Predicted number of multiple-vehicle collisions for intersections of type j, j=1,..,4
N_{isj} = Predicted number of single-vehicle collisions for intersections of type j
O_{rmj} = Observed number of multiple-vehicle nondriveway collisions for roadway segments of type j
O_{rsj} = Observed number of single-vehicle collisions for roadway segments of type j
O_{rdj} = Observed number of driveway-related collisions for roadway segments of type j
O_{imj} = Observed number of multiple-vehicle collisions for intersections of type j
O_{isj} = Observed number of single-vehicle collisions for intersections of type j
k_{rmj} = Overdispersion parameter for multiple-vehicle nondriveway collisions for roadway segments of type j
k_{rsj} = Overdispersion parameter for single-vehicle collisions for roadway segments of type j
k_{rdj} = Overdispersion parameter for driveway-related collisions for roadway segments of type j
k_{imj} = Overdispersion parameter for multiple-vehicle collisions for intersections of type j
k_{isj} = Overdispersion parameter for single-vehicle collisions for intersections of type j
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_{comb} = expected accident frequency of combined sites including two or more roadway segments or intersections

For roadway segments, Hauer (*126*) has recommended that the over dispersion parameter, k, be estimated so that it applies to a unit length of roadway segment; thus, the over dispersion parameter, k_i, for a roadway segment, i, of length L_i would be $k_i=k/L_i$. Miaou and Lord (*127*) suggest that k itself be modeled as a function of traffic volume and other variables. However, the research team used the more conventional approach in developing NB models using a single common over dispersion parameter for all sites, thus making the assumption that the over dispersion parameter does not vary from site to site within a given roadway type, regardless of their length. This approach, expressed mathematically as $k_i = k$ for all L_i, is at one end of the bracketing assumptions about the over dispersion parameter, the other end being

Hauer's assumption that $k_i = k/L_i$. Hauer (*126*) indicates that the correct value may, in fact, be somewhere in between these two assumptions.

An alternative procedure for combining predicted and observed accident frequencies is the full Bayes procedure. The full Bayes procedure has the advantage of using a distribution of prior probabilities rather than a point estimate as used in the EB approach. However, the full Bayes procedure is computationally more difficult to apply than the EB procedure and would be difficult for the HSM to use in incorporating actual observed accident history data into the analysis of a specific site since the observed accident history data for a site are available only to the user of the HSM methodology, not to the researchers who develop the methodology. Because of its computational simplicity and wider application, the EB approach is recommended for use in the HSM methodology.

CHAPTER 4.

DEVELOPMENT OF PROJECT DATABASE

This section of the report describes the project database that was analyzed in the development of the HSM methodology for urban and suburban arterials.

PARTICIPATING HIGHWAY AGENCIES

The project database was developed for arterial roadways and intersections under the jurisdiction of eight highway agencies. Data for roadway segments were obtained for arterial roadways under the jurisdiction of:

- Minnesota Department of Transportation
- Hennepin County, Minnesota
- Ramsey County, Minnesota
- Michigan Department of Transportation
- Oakland County, Michigan
- Washington State Department of Transportation

Data for intersections were obtained for roadway segments under the jurisdiction of:

- Minnesota Department of Transportation
- City of Charlotte, North Carolina
- Florida Department of Transportation

The data from Minnesota, Michigan, and North Carolina were used in developing the HSM methodology. The data from Washington and Florida were used in validating the methodology.

SITE SELECTION FOR ROADWAY SEGMENTS

Based on project priorities and a preliminary review of the available sites, a decision was reached to focus the analysis on five types of roadway segments:

- two-lane undivided arterials (2U)
- three-lane arterials including a center TWLTL (3T)
- four-lane undivided arterials (4U)
- four-lane divided arterials (4D)
- five-lane arterials including a center TWLTL (5T)

In Minnesota, urban and suburban arterial roadway segments of these types were selected on state routes under the jurisdiction of the Minnesota Department of Transportation and on county roads in Hennepin and Ramsey Counties. Most of the state highway routes and all of the county roads that were studied were located in the Twin Cities metropolitan area. A few urban and suburban state routes outside the Twin Cities metropolitan area were included where necessary to obtain a sufficient number of sites.

Study sites in Minnesota were identified through review of roadway inventory files obtained from the FHWA Highway Safety Information System (HSIS), from videolog review, from field visits to sites, and from suggestions from the participating highway agencies. All candidate sites were reviewed on videolog or visited in the field to verify their suitability.

Study sites in Oakland County, Michigan, were chosen primarily on county roads and city streets, although a few roadway segments on state highways in Oakland County were also selected. Initial site selections were made from an inventory of arterial roadway segments maintained by the Traffic Improvement Association of Oakland County. All candidate sites were then visited in the field to verify their suitability.

The project database used for roadway segment model development was divided into individual blocks where each block began and ended at public road intersections along the arterials being studied. The database included 4,255 blocks; 2,436 in Minnesota and 1,819 in Michigan. The individual blocks ranged in length from 0.06 to 2.28 km (0.04 to 1.42 mi). The total length of all blocks considered in the study was 890.2 km (553.3 mi), 489.0 km (303.9 mi) in Minnesota, and 473.7 km (294.4 mi) in Michigan. The average block length was 0.19 km (0.12 mi) in Minnesota and 0.23 km (0.14 mi) in Michigan. Table 12 summarizes the number of blocks for each roadway type included in the project database. Table 13 presents a comparable summary of the total length of roadway segment used for model development.

TABLE 12. Summary of number of study sites for individual blocks on arterial roadway segments

Roadway type	Number of study sites		
	Minnesota	Michigan	Total
2U	577	590	1,167
3T	380	100	480
4U	741	440	1,181
4D	540	140	680
5T	198	549	747
Total	2,436	1,819	4,255

TABLE 13. Summary of total length of study sites for arterial roadway segments

Roadway type	Length of study sites (mi)		
	Minnesota	Michigan	Total
2U	77.6	88.1	165.7
3T	45.4	14.3	59.7
4U	77.9	37.6	115.5
4D	80.5	29.6	110.1
5T	22.5	79.8	102.3
Total	303.9	249.4	553.3

SITE SELECTION FOR INTERSECTIONS

The intersection analysis focused on four types of at-grade intersections:

- three-leg intersections with STOP control on the minor-road approach (3ST)
- three-leg signalized intersections (3SG)
- four-leg intersections with STOP control on the minor-road approaches (4ST)
- four-leg signalized intersections (4SG)

In Minnesota, intersections on urban and suburban arterials were selected on state routes under the jurisdiction of the Minnesota Department of Transportation. All of the intersections studied were located in the Twin Cities metropolitan area. Study intersections in Minnesota were identified through review of HSIS roadway inventory files, from videolog review, or from field visits to sites. All candidate sites were reviewed on videolog or in the field to verify their suitability.

Study intersections in Charlotte, North Carolina, were selected on recommendations by the City traffic engineering staff and from field review. The City also provided aerial photographs of the candidate sites. The suitability of each intersection for inclusion in the study was verified from the review of the aerial photograph and/or from a field visit.

The project database included 363 intersections, 182 in Minnesota, and 181 in North Carolina. The number of intersections of each type is summarized in Table 14. The table includes only intersections for which accident, major-road traffic volume, minor-road traffic volume, and site geometric data were available. Candidate sites for which minor-road traffic volume or geometric design data were not available have been excluded from the table.

TABLE 14. Summary of number of study intersections

Intersection type	Number of study intersections		
	Minnesota	North Carolina	Total
3ST	36	47	83
3SG	34	42	96
4ST	48	48	76
4SG	64	44	108
Total	182	181	363

DATA COLLECTION

Several types of data were collected for each study site including site characteristics and accident data. The collection of each of these types of data for both roadway segments and intersections is addressed below.

Site Characteristics Data for Roadway Segments

Site characteristics data obtained for roadway segments included geometric design, traffic control, and traffic volume data. Sources of site characteristics data for roadway segments included existing roadway inventory files, videologs, aerial photographs, and field visits. Data from roadway inventory files, videologs, and aerial photographs were used where available and were verified in the field where possible.

Table 15 lists all of the site characteristics data elements included in the project database for roadway segments and identifies the source of each data element. As shown in the table, most of the data elements were obtained from field or videolog review. For roadway segments, most of the sites in Minnesota and all of the sites in Oakland County, Michigan, were reviewed in the field. Videolog review was used for selected sites in Minnesota.

The data elements obtained for the study sites are intentionally broader than needed for the initial project analyses. For example, the additional data gathered on pedestrian facilities should also be useful for the planned pedestrian safety modeling activities to be conducted as part of this same project.

Driveway locations, driveway types, median opening locations, median opening types, horizontal curve locations, horizontal curve radii, and horizontal curve locations were determined from review of aerial photographs.

Driveway types were classified into seven categories:

- Major commercial driveways
- Minor commercial driveways
- Major industrial/institutional driveways
- Major residential driveways
- Minor residential driveways
- Other driveways

Major driveways are those that serve sites with 50 or more parking spaces. Minor driveways are those that serve sites with less than 50 parking spaces. Commercial driveways provide access to establishments that serve retail customers. Residential driveways serve single- and multiple-family dwellings. Industrial/institutional driveways serve factories, warehouses, schools, hospitals, churches, public facilities, and other places of employment.

TABLE 15. Site characteristics data obtained for roadway segments on urban and suburban arterials

Data element	Description of data element	Primary source
Beginning landmark	Name of intersecting street or other landmark at beginning of block	Highway agency data or field review
Beginning milepost	Milepost or log mileage at beginning of block to tie field locations to accident data	Highway agency data
Bicycle route (marked)	Presence of bicycle route marked by signs	Field or videolog review
Bicycle facilities	Presence of bicycle facilities including sidepath, marked bicycle lane, or wide curb lane	Field or videolog review
Driveway locations	Location of driveway (side of road and distance from beginning of block)	Aerial photograph and field review
Driveway types	Each driveway was classified into one of seven categories (see accompanying text)	Aerial photograph and field review
Ending landmark	Name of intersecting street or other landmark at end of block	Highway agency data or field review
Ending milepost	Milepost or log mileage at end of block to tie field locations to accident data	Highway agency data
Grade	Roadway grade within block (level, moderate, or steep)	Field or videolog review
Horizontal curve length	Length of horizontal curve (mi) computed from beginning and ending locations	Computed
Horizontal curve location	Distance of beginning and end of horizontal curve from beginning of block (mi)	Aerial photographs
Horizontal curve radius	Radius of horizontal curve (ft) measured on aerial photograph	Aerial photographs
Intersections	A basic data set was collected for the intersections at each end of each block including number of legs, side of road (for three-leg intersections), traffic control device, and type of pedestrian facilities (if any). This data set is less extensive than the data collected for the full intersection study sites.	Field or videolog review
Lane width	Width of through lanes (ft) not including gutters. Measured at first block in a series of consecutive blocks and points of change. Lane width is averaged over multiple lanes where present.	Field measurement

TABLE 15. Site characteristics data obtained for roadway segments on urban and suburban arterials (Continued)

Data element	Description of data element	Primary source
Length of site	Length of site (mi) from beginning landmark to end landmark. Measured from center of intersection where intersections are site boundaries.	Highway agency data
Lighting	Presence of street lighting (none, intersection only, or continuous lighting along street) and presence of other ambient lighting	Field review
Median opening location	Distance of median opening from beginning of block (mi). Applicable to divided streets only.	Aerial photograph or field review
Median opening type	Type of median opening (conventional/directional)	Aerial photograph or field review
Median type	Presence and type of median (none, raised, depressed, flush)	Field or videolog review
On-street parking	Presence and type of on-street parking (none, parallel parking, angle parking)	Field or videolog review
Pedestrian crosswalk (midblock)	Location and type of midblock crosswalk (if any)	Field or videolog review
Roadside hazard rating	Rating on 1 to 7 scale (see accompanying text)	Field or videolog review
Roadway type	Number of through lanes and divided undivided (2U, 3T, 4U, 4D, or 5T as defined earlier in this chapter.	Field or videolog review
Route number or street name	Route number or street name for arterial used to tie field locations to accident data	Highway agency data
Shoulder type	Type of shoulder (paved, gravel, turf, composite) and presence/absence of curb. Determined separately for each side of the road.	Field or videolog review
Shoulder width	Width of shoulder (ft). Determined separately for each side of the road. Entered as zero for curbed sections.	Field measurement
Sidewalks	Presence/absence of sidewalk. Determined separately for each site of the road.	Field or videolog review
Speed limit	Posted speed limit (mph) or speed limit applicable under state law	Field or videolog review
Traffic volume	ADTs for each year of study period. Interpolated between count years when not available for every year.	Highway agency data

Roadside hazards are rated on a 1 to 7 scale developed by Zegeer et al (*9*) with 1 representing the best-designed roadside and 7 representing a poorly-designed roadside.

Site Characteristics Data for Intersections

Site characteristics data obtained for intersections included geometric design, traffic control, and traffic volume data. Sources of site characteristics data for intersections included existing roadway inventory files, aerial photographs, and field visits. All of the primary study intersections were visited in the field, including the intersections in both Minnesota and North Carolina.

Table 16 lists all of the site characteristics data elements included in the project database for intersections and identifies the source of each data element. The table is subdivided into sections for data elements applicable to:

- the intersection as a whole
- individual intersection approaches
- pedestrian facilities on individual intersection approaches
- bicycle facilities on individual intersection approaches

As shown in the table, most of the data elements were obtained from field review. Where highway agency data were used, they were confirmed during the field review.

The data elements obtained for the study intersections are intentionally broader in scope than needed for the initial project analyses. For example, the data should also be useful for the planned pedestrian safety modeling activities to be conducted as part of this same project.

ACCIDENT DATA

Accident data were obtained from the participating highway agencies for the study sites under their jurisdiction. The accident study period for each site type and jurisdiction were as follows:

- Minnesota roadway segments, 1998-2002
- Michigan roadway segments, 1999-2003
- Minnesota intersections, 1998-2002
- North Carolina intersections, 1997-2003

TABLE 16. Site characteristics data obtained for intersections on urban and suburban arterials

Data element	Description of data element	Primary source
Data for Intersection as a Whole		
Angle of intersection	Angle between intersecting streets (90° for right-angle intersections; less than 90° for skewed intersections)	Aerial photographs or field review
Character of development	Type of surrounding land use (central business district, outlying commercial, industrial mixed, residential, undeveloped)	Field review
Intersection type	Number of intersection legs and type of traffic control (3ST, 3SG, 4ST, and 4SG as defined earlier in this chapter)	Field review
Level of pedestrian activity	Level of pedestrian activity at intersection based on observed number of pedestrians and adjacent development (low, moderate, high)	Field review
Lighting	Presence of street lighting (none, intersection only, lighting along street) and presence of other ambient lighting	Field review
Milepost or node number	Milepost or node number to tie field locations to accident data	Highway agency data
Number of intersection legs	Number of legs at the intersection (3 or 4)	Field review
Route number or street name	Route number or street name used with milepost to tie field locations to accident data	Highway agency data
Traffic volume for major road	Two-way traffic volume (ADT) on the major road at the intersection. If the traffic volumes for the two major-road legs differ, the larger volume is used.	Highway agency data
Traffic volume for minor road	Two-way traffic volume (ADT) on the minor road at the intersection. If there are two minor road legs and the traffic volumes on those legs differ, the larger volume is used.	Highway agency data
Data for Individual Intersection Approaches		
Advance warning signs	Presence of advance warning signs such as crossroad, STOP ahead, or signal ahead	Field review
Approach grade	Grade of approach (level, moderate upgrade, steep upgrade, moderate downgrade, steep downgrade	Field review
Crest/sag vertical curve	Presence of crest or sag vertical curve on approach within 76 m (250 ft) of the intersection	Field review

TABLE 16. Site characteristics data obtained for intersections on urban and suburban arterials (Continued)

Data element	Description of data element	Primary source
Divided/undivided	Presence/absence of median intersection approach	Field review
Horizontal alignment	Presence of horizontal curve on intersection approach within 76 m (250 ft) of intersection (tangent, gentle curve, moderate curve, sharp curve)	Field review
Lane width-right lane	Lane width of rightmost through traffic lane not including gutters	Field measurement
Lane width-left lane	Lane width of leftmost through traffic lane for approaches with two or more through lanes	Field measurement
Lane width-middle lane	Lane width of middle through traffic lane for approaches with three through lanes	Field measurement
Lane width-left-turn lane	Lane width for left-turn lane (ft)	Field measurement
Lane width-right-turn lane	Lane width for right-turn lane (ft)	Field measurement
Left-turn prohibition	Left turns prohibited by sign (no sites with left-turn prohibitions were included)	Field review
Left-turn phasing	Presence of exclusive left-turn signal phasing (none, protected, protected/permitted). Applicable only to signalized intersections.	Field review
Median width	Width of median (ft) on intersection approach. Recorded as zero for undivided roadways.	Field measurement
Median type	Presence and type of median on intersection approach (none, raised, depressed, flush)	Field review
Number of through lanes	Number of through traffic lanes on intersection approach	Field review
Number of left-turn lanes	Number of left-turn lanes on intersection approach	Field review
Number of right-turn lanes	Number of right-turn lanes on intersection approach	Field review
Number of driveways	Number of driveways on both sides of the road within 76 m (250 ft) of the intersection	Field review
Type of driveways	Predominant driveway type on the intersection approach (see accompanying text)	Field review
One-way/two-way operation	Type of traffic operation on intersection leg containing the approach (one-way vs. two-way). No sites with one-way approaches were included.	Field review
On-street parking	Presence of on-street parking on the approach within 76 m (250 ft) of the intersection	Field review
Posted speed limit	Posted speed limit (mph) or speed limit applicable under state law on the intersection approach	Field review

TABLE 16. Site characteristics data obtained for intersections on urban and suburban arterials (Continued)

Data element	Description of data element	Primary source
Route number or street name	Route number or street name of the intersection approach	Field review
Right turn on red	Presence of right turn on red from the intersection approach	Field review
Shoulder type	Type of shoulder on the intersection approach (paved, gravel, turf, or composite) and presence/absence of curb. Determined for right side of road only.	Field review
Shoulder width	Width of shoulder (ft) on right side of intersection approach. Entered as zero for curbed sections.	Field review.
Signal visibility from approach	Indicator of whether signal is visible to approaching traffic	Field review
Traffic control	Type of traffic control on the intersection approach	Field review
Transit stop locations	Locations of transit stops on the intersection approach or on the departing roadway beyond the intersection (none, rear side, far side, both)	Field review
Type of intersection leg	Classification of intersection leg as major road or minor road. At STOP-controlled intersections, the uncontrolled approaches are considered to be the major road. At signalized intersections, the higher volume road is considered to be the major road.	Field review and highway agency
Type of left-turn treatment	Configuration of left-turn lane on the intersection approach (none, painted, curbed, offset)	Field review
Type of right-turn treatment	Configuration of right-turn lane on the intersection approach (none, right-turn lane only, channelizing island only, right-turn lane, and channelizing island	Field review
Data for Pedestrian Facilities on Individual Intersection Approaches		
Pedestrian signals	Presence of pedestrian signals for crossing the intersection approach. Applicable to signalized intersections only.	Field review
Special pedestrian signal features	Presence of special pedestrian signal features including pushbutton actuation and countdown signals	Field review
Marked crosswalk	Presence of marked crosswalk for crossing the intersection approach	Field review

TABLE 16. Site characteristics data obtained for intersections on urban and suburban arterials (Continued)

Data element	Description of data element	Primary source
Pedestrian refuge area	Presence of pedestrian refuge area in the median for pedestrians crossing the leg containing the intersection approach	Field review
Pedestrian crossing distance	Length of pedestrian path (ft) for crossing the leg containing the intersection approach. Measured as two separate distances if a refuge area is present.	Field review
Pedestrian crossing prohibition	Presence of signs or barriers to prevent legal crossing of the intersection approach by pedestrians	Field review
Sidewalks	Presence of sidewalks on the leg containing the intersection approach. Determined separately for the left and right sides of the road.	Field review
Sidewalk curb ramps	Presence and type of sidewalk curb ramps for pedestrians crossing the leg containing the intersection approach (none, perpendicular, parallel, diagonal). Determined separately for the corners on the left and right side of the road.	Field review
Data for Bicycle Facilities on Individual Intersection Approaches		
Marked bicycle lane	Presence of marked bicycle lane on intersection approach	Field review
Wide curb lane	Presence of wide curb lane on intersection approach defined as a right lane over 16 ft in length	Field review
Parallel bike path	Presence of a side path for bicycles along the intersection approach	Field review

The study period for each agency was five years, except that for North Carolina intersections the study period was seven years. The entire computerized record for each crash during the study period for each agency was obtained. No attempt was made to impose *a priori* restrictions on which accident characteristics might be of interest.

Each accident in the project database was assigned to a particular roadway segment or intersection. The criteria used to make this assignment, which are the same as those used in the FHWA IHSDM (*7*), are illustrated in Figure 9. These criteria are as follows:

- Accidents classified by the investigating officer or accident coder as "at intersection" (i.e., within the curbline limits of the intersection) were assigned to the intersection.

- Accidents on an intersection leg within 76 m (250 ft) of the intersection were assigned to the intersection if they were classified by the investigating office or accident coder as "intersection related"; otherwise, they were assigned to the roadway segment that contains the intersection leg. For example, driveway-related collisions would not be considered as "intersection related."
- Accidents more than 76 m (250 ft) from the intersection were assigned to the roadway segment on which they occurred.

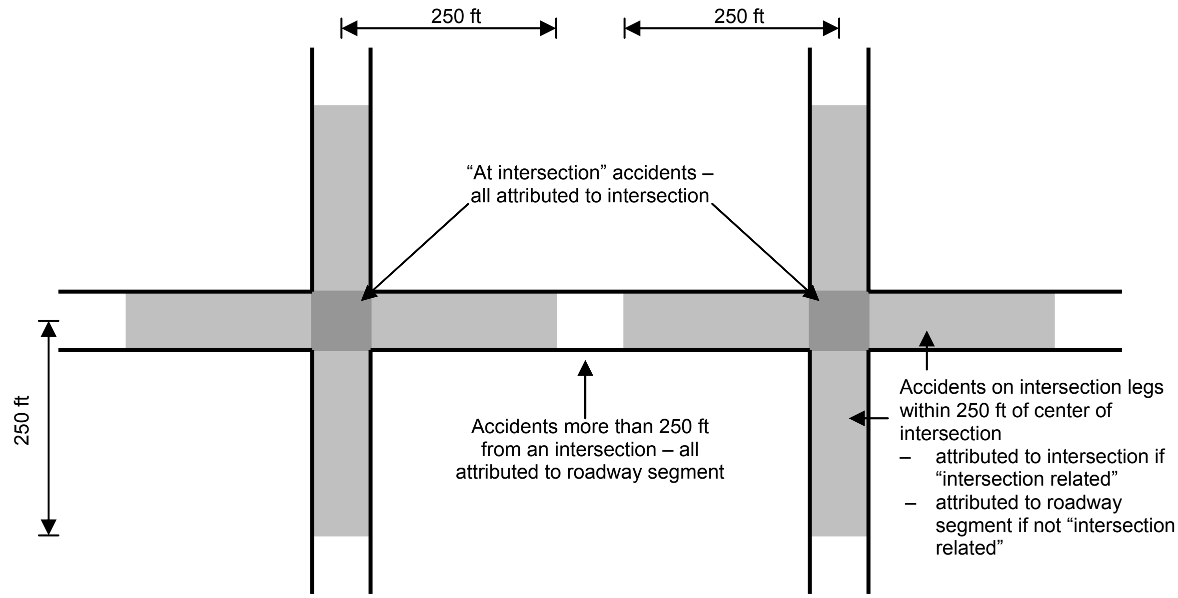

Figure 9. Assignment of accidents to roadway segments and intersections

The objective of these criteria was to attribute to the roadway segment all accidents that would have occurred had the intersection not been present; the accidents attributed to the intersection are those that occurred only because the intersection was present.

All "at-intersection" accidents were attributed to the intersection. A case could be made that single-vehicle accidents within the curbline limits of an intersection (i.e., running off the road and hitting a utility pole) might better be attributed to the adjacent roadway segment because they might have occurred even if the intersection were not present. However, a case can also be made that single-vehicle accidents at an intersection may involve a single-vehicle avoiding a multiple-vehicle collision and, thus, may be related to the intersection. Vogt and Bared (*105*) considered this issue for IHSDM and found that attributing all "at-intersection" accidents to the intersection worked best.

Accidents within 76 m (250 ft) of the intersection may be attributed to either the intersection or the roadway segment that contains the intersection leg based on whether the accident is classified as "intersection related." The available databases include only public-road accidents; therefore, accidents on private property within 76 m (250 ft) of an intersection would not be attributed to either the intersection or a roadway segment. Where two intersections are less than 152 m (500 ft) apart, intersection-related accidents were generally attributed to the closest intersection. However, if all accident-involved vehicles were traveling away from one intersection and toward another closely-spaced intersection, the accident was attributed to the intersection the vehicles were traveling toward.

These criteria were applied as rigorously as possible given the nature of each agency's database. The only exception to these criteria was that, for intersection accidents in Charlotte, North Carolina, there were no explicit data on the distance from the site of each accident to the intersection, so some "intersection-related" accidents beyond 76 m (250 ft) from the intersection may have been included.

Driveway-related collisions on roadway segments were also modeled separately from other multiple-vehicle collisions. Driveway-related collisions are those identified by the investigating officer or accident coder as related to the operation of driveway and typically involve one or more vehicles entering or leaving a driveway.

The accident data set for the project included 43,798 accidents for all study jurisdictions combined, 32,277 for roadway segments, and 11,521 for intersections. Table 17 presents the distribution of roadway segment accidents by roadway type and jurisdiction. Table 18 presents comparable data for intersections.

TABLE 17. Distribution of roadway segment accidents by roadway type and jurisdiction

Roadway type	Number of accidents in five-year study period		
	Minnesota[a]	Michigan[b]	Combined
2U	1,539	4,069	5,608
3T	1,184	940	2,124
4U	2,955	2,795	5,750
4D	3,154	1,531	4,685
5T	974	13,136	14,110
Total	9,806	22,471	32,277

[a] 1998-2002.
[b] 1999-2003.

TABLE 18. Distribution of intersection accidents by intersection type and jurisdiction

Intersection type	Number of accidents in five-year study period		
	Minnesota[a]	North Carolina[b]	Combined
3ST	161	896	1,057
3SG	602	2,404	3,006
4ST	382	1,038	1,420
4SG	1,516	4,522	6,038
Total	2,661	8,860	11,521

[a] 1998-2002.
[b] 1997-2003.

Table 19 summarizes the number of roadway segment sites (blocks) and their total length, average traffic volume, total exposure (million veh-mi of travel), total number of accidents, and average accident rate.

Table 20 presents summary accident distribution data for roadway segments by roadway type and jurisdiction. In the table, accidents are broken down into five collision types which were addressed separately in project analyses: multiple-vehicle nondriveway collisions; single-vehicle collisions; driveway-related collisions; vehicle-pedestrian collisions; and vehicle-bicycle collisions. Tables 21 and 22 present the distribution of the multiple-vehicle nondriveway collisions and single-vehicle collisions by collision type.

Tables 23 through 26 present comparable data to Tables 19 through 22 for intersection accidents. In Table 23 for intersections, the exposure measure is million entering vehicles. In Table 24 for intersections, the category of driveway-related collisions is not relevant and is, therefore, omitted.

TABLE 19. Summary of roadway segment accident and exposure data

Roadway type	Number of blocks	Total length (mi)	Average ADT (veh/day)	Total exposure (10^6 veh-mi)[a]	Total number of accidents[a]	Average accident rate (per 10^6 veh-mi)
MINNESOTA						
2U	577	77.57	9,376	1,327.2	7,539	1.16
3T	380	45.40	10,806	895.3	1,184	1.32
4U	742	78.00	13,534	1,926.5	2,955	1.53
4D	540	80.47	22,260	3,268.9	3,154	0.96
5T	205	23.57	15,013	645.8	974	1.51
MICHIGAN						
2U	590	88.06	13,246	2,128.9	4,069	1.91
3T	100	14.27	14,846	386.6	940	2.43
4U	440	37.60	21,259	1,458.8	2,795	1.92
4D	140	29.62	17,784	961.4	1,531	1.59
5T	549	79.82	29,703	4,326.8	13,136	3.04
COMBINED						
2U	1,167	165.63	11,434	3,456.1	5,608	1.62
3T	480	59.67	11,772	1,281.9	2,124	1.66
4U	1,182	115.60	16,046	3,385.3	5,750	1.70
4D	680	110.09	21,056	4,230.3	4,685	1.11
5T	754	103.39	26,354	4,972.6	14,110	2.84

[a] In five-year period.

TABLE 20. Roadway segment accident distribution by roadway type and jurisdiction

Roadway type	Multiple-vehicle nondriveway collisions		Single-vehicle collisions		Driveway-related collisions		Vehicle-pedestrian collisions		Vehicle-bicycle collisions		Total	
	Number (percentage) of accidents in five years											
MINNESOTA												
2U	727	(47.2)	514	(33.4)	256	(16.6)	27	(1.8)	15	(1.0)	1,539	(100.0)
3T	700	(59.1)	176	(14.9)	272	(23.0)	20	(1.7)	16	(1.4)	1,184	(100.0)
4U	1,748	(59.2)	488	(16.5)	589	(19.9)	75	(2.5)	55	(1.9)	2,955	(100.0)
4D	2,443	(77.5)	452	(14.3)	187	(5.9)	38	(1.2)	34	(1.1)	3,151	(100.0)
5T	511	(52.5)	102	(10.5)	329	(33.8)	20	(2.1)	12	(1.2)	974	(100.0)
MICHIGAN												
2U	2,310	(56.8)	1,031	(25.3)	693	(17.0)	14	(0.3)	21	(0.5)	4,069	(100.0)
3T	636	(67.7)	73	(7.8)	228	(24.3)	1	(0.1)	2	(0.2)	940	(100.0)
4U	1,754	(62.8)	225	(8.1)	767	(27.4)	22	(0.8)	27	(1.0)	2,765	(100.0)
4D	1,066	(69.6)	197	(12.9)	244	(15.9)	9	(0.6)	15	(1.0)	1,531	(100.0)
5T	8,681	(66.1)	819	(6.2)	3,556	(27.1)	46	(0.4)	34	(0.3)	13,136	(100.0)
COMBINED												
2U	3,037	(54.2)	1,545	(27.5)	949	(16.9)	41	(0.7)	36	(0.6)	5,608	(100.0)
3T	1,336	(62.9)	249	(11.7)	500	(23.5)	21	(1.0)	18	(0.8)	2,124	(100.0)
4U	3,502	(60.9)	713	(12.4)	1,355	(23.6)	97	(1.7)	82	(1.4)	5,750	(100.0)
4D	3,509	(74.9)	649	(13.9)	431	(9.2)	47	(1.0)	49	(1.0)	4,685	(100.0)
5T	9,192	(65.1)	921	(6.5)	3,885	(27.5)	66	(0.5)	46	(0.3)	14,110	(100.0)

NOTE: Numbers in parentheses are row percentages.

TABLE 21. Distribution of manner of collision for multiple-vehicle nondriveway collisions on roadway segments for Minnesota and Michigan combined

Collision type	Percentage of accidents for each roadway type				
	2U	3T	4U	4D	5T
Rear-end	67.0	64.7	49.9	67.8	59.0
Head-on	4.7	3.7	4.3	1.3	5.3
Angle	10.7	15.0	17.2	10.0	17.0
Sideswipe, same direction	6.8	6.3	16.3	11.2	13.0
Sideswipe, opposite direction	5.8	2.1	2.8	1.3	2.4
Other multiple-vehicle collisions	5.0	8.2	9.5	8.4	3.3

TABLE 22. Distribution of collision type for single-vehicle collisions on roadway segments for Minnesota and Michigan combined

Collision type	Percentage of accidents by roadway type				
	2U	3T	4U	4D	5T
Collision with parked vehicle	37.7	16.5	43.5	11.9	25.6
Collision with animal	14.4	25.6	3.5	12.8	17.6
Collision with fixed object	20.4	28.9	25.8	30.2	23.5
Collision with other object	14.6	13.6	18.0	23.0	19.6
Other single-vehicle collision	1.0	0.6	3.9	5.1	4.9
Noncollision	11.9	14.8	5.3	17.0	8.8

TABLE 23. Summary of intersection accident and exposure data

Intersection type	Number of intersections	Average major-road ADT (veh/day)	Average minor-road ADT (veh/day)	Total exposure (10^6 entering vehs)[a]	Total number of accidents[a]	Average accident rate (per 10^6 entering vehs)
MINNESOTA						
3ST	36	16,523	1,157	1,161.6	161	0.14
3SG	34	24,597	5,331	1,857.0	602	0.32
4ST	48	17,868	956	1,648.9	382	0.23
4SG	64	21,270	5,502	3,127.0	1,516	0.48
NORTH CAROLINA						
3ST	47	12,691	2,173	1,784.9	896	0.50
3SG	42	21,354	3,908	2,710.9	2,404	0.89
4ST	48	14,074	1,409	1,898.8	1,038	0.55
4SG	44	20,796	9,133	3,364.6	4,522	1.34
COMBINED						
3ST	83	14,356	1,854	2,946.6	1,057	0.36
3SG	76	22,938	4,507	4,568.0	3,006	0.66
4ST	96	15,655	1,220	3,547.8	1,420	0.40
4SG	108	20,804	7,058	6,491.6	6,038	0.93

[a] In five-year period for Minnesota and in a seven-year period for Charlotte.

TABLE 24. Intersection accident distribution by intersection type and jurisdiction

Intersection type	Number (percentage) of accidents				
	Multiple-vehicle collisions	Single-vehicle collisions	Vehicle-pedestrian collisions	Vehicle-bicycle collisions	Total
MINNESOTA[a]					
3ST	140 (87.0)	19 (11.8)	1 (0.6)	1 (0.6)	161 (100.0)
3SG	547 (90.9)	44 (7.3)	2 (0.3)	9 (1.5)	602 (100.0)
4ST	318 (83.2)	50 (13.1)	10 (2.6)	4 (1.0)	382 (100.0)
4SG	1,327 (87.5)	125 (8.2)	34 (2.2)	30 (2.0)	1,516 (100.0)
NORTH CAROLINA[b]					
3ST	785 (87.6)	102 (11.4)	8 (0.9)	1 (0.1)	896 (100.0)
3SG	2,251 (93.6)	128 (5.3)	15 (0.6)	10 (0.4)	2,404 (100.0)
4ST	946 (91.1)	85 (8.2)	5 (0.5)	2 (0.2)	1,038 (100.0)
4SG	4,295 (95.0)	162 (3.6)	49 (1.1)	16 (0.4)	4,522 (100.0)
COMBINED					
3ST	925 (87.5)	121 (11.4)	9 (0.9)	2 (0.2)	1,057 (100.0)
3SG	2,798 (93.1)	172 (5.7)	17 (0.6)	19 (0.6)	3,006 (100.0)
4ST	1,264 (89.0)	135 (9.5)	15 (1.1)	6 (0.4)	1,420 (100.0)
4SG	5,622 (93.1)	287 (4.8)	83 (1.4)	46 (0.8)	6,038 (100.0)

[a] Includes five years of accident data.
[b] Includes seven years of accident data.
NOTE: Numbers in parentheses are row percentages.

TABLE 25. Distribution of manner of collision for multiple-vehicle intersection collisions in Minnesota and North Carolina combined

Collision type	Percentage of accidents for each intersection type			
	3ST	3SG	4ST	4SG
Rear-end	48.5	51.7	42.5	50.7
Head-on	0.5	0.6	0.9	0.7
Angle	38.1	33.0	44.8	34.4
Sideswipe, same direction	7.7	10.2	6.1	9.2
Sideswipe, opposite direction	0.4	0.4	0.8	0.4
Other multiple-vehicle collisions	4.8	4.1	4.9	4.7

TABLE 26. Distribution of collision type for single-vehicle accidents in Minnesota and North Carolina combined

Collision type	Percentage of accidents by intersection type			
	3ST	3SG	4ST	4SG
Collision with parked vehicle	0.0	1.2	4.4	4.9
Collision with animal	6.6	2.9	1.5	1.7
Collision with fixed object	14.0	28.5	10.4	24.7
Collision with other object	5.8	8.1	5.2	15.0
Other single-vehicle collision	20.7	12.2	40.0	24.0
Noncollision	52.9	47.1	38.5	29.6

Table 27 summarizes the independent variables that were included in the definition of roadway segment and intersection types or considered in the development of the base models for roadway segments or intersections presented in Chapter 5 of this report. Other variables of interest that were included in the project database (see Tables 15 and 16) may not have varied sufficiently within individual roadway segment or intersection types to be suitable for modeling.

Tables 28 and 29 present descriptive statistics for continuous and categorical independent variables, respectively, considered in modeling of roadway segments. Tables 30 and 31 present descriptive statistics for continuous and categorical variables, respectively, considered in modeling of intersections.

VALIDATION DATA

A validation data set has been developed with data for roadway segments from the Washington State DOT and data for intersections from the Florida DOT. The validation work based on these datasets is described in Chapter 8 of this report.

TABLE 27. Variables considered in safety prediction modeling

Variable	Multiple-vehicle nondriveway collisions	Single-vehicle collisions	Driveway-related accidents
Roadway Segments			
Traffic volume (ADT)	✓	✓	✓
Segment length	✓	✓	✓
Number of through lanes	✓	✓	✓
Presence of median	✓	✓	✓
Presence of TWLTL	✓	✓	
Lane width	✓	✓	✓
Shoulder width	✓	✓	
Shoulder type	✓	✓	
On-street parking	✓	✓	
Roadside hazard rating	✓	✓	
Speed category	✓	✓	
Number of driveways			✓
Driveway types			✓
Driveway density			✓
Intersections			
Number of legs	✓	✓	
Type of traffic control	✓	✓	
Major-road ADT	✓	✓	
Minor-road ADT	✓	✓	
Left-turn phasing	✓	✓	
Speed category	✓	✓	
Lane width	✓	✓	
Skew angle	✓	✓	
Character of development	✓	✓	

TABLE 28. Descriptive statistics for continuous variables for roadway segments

Variable/statistic	Minnesota					Michigan				
	2U	3T	4U	4D	5T	2U	3T	4U	4D	5T
ADT (veh/day)										
Mean	9,376	10,806	13,534	22,260	15,013	13,246	14,846	21,259	17,784	29,703
Median	8,607	10,226	13,787	18,986	14,979	13,666	14,303	21,180	24,232	27,775
Standard deviation	207	211	226	473	353	282	862	343	1,066	448
Minimum	715	3,437	1,361	949	5,685	2,389	2,644	9,401	4,793	6,273
Maximum	27,012	27,012	39,516	55,232	27,842	32,571	32,902	40,045	65,912	53,781
Shoulder width (ft)										
Mean	4.31	2.42	0.49	2.47	0.47	4.20	2.12	0.05	0.13	0.38
Median	3.00	0.00	0.00	0.00	0.00	3.00	0.00	0.00	0.00	0.00
Standard deviation	0.24	0.22	0.08	0.22	0.11	0.15	0.35	0.01	0.05	0.07
Minimum	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Maximum	12.00	13.00	10.00	13.00	8.00	13.00	14.00	3.00	3.00	12.00
Roadside hazard rating (1 to 7 scale)										
Mean	3.2	3.9	4.0	3.5	3.9	3.1	3.2	3.7	2.8	2.3
Median	3.0	4.0	4.0	4.0	4.0	3.0	3.0	3.5	3.0	2.0
Standard deviation	0.1	0.1	0.0	0.1	0.1	0.0	0.1	0.1	0.1	0.0
Minimum	1.0	1.0	1.5	1.0	2.0	1.0	1.0	1.0	1.0	1.0
Maximum	6.0	7.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
Number of driveways[a]										
Mean	3.0	4.7	3.9	1.0	6.0	4.3	3.9	4.9	3.5	5.9
Median	2.0	3.0	2.0	0.0	3.0	3.0	3.0	4.0	2.0	4.0
Standard deviation	0.2	0.6	0.3	0.1	1.5	0.2	0.3	0.2	0.4	0.2
Minimum	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Maximum	30.0	37.0	63.0	14.0	20.0	40.0	18.0	29.0	36.0	37.0
Driveway density (driveways/mi)[a]										
Mean	25.7	34.9	36.3	8.7	31.2	32.6	39.4	61.3	27.9	43.4
Median	16.8	24.7	29.3	0.0	22.1	23.3	29.8	58.8	17.2	37.0
Standard deviation	1.4	3.1	1.9	1.2	6.2	1.4	3.5	1.8	2.9	1.5
Minimum	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Maximum	119.0	148.0	175.0	114.8	82.3	322.0	156.5	196.7	188.7	241.9

[a] Includes driveways on both sides of the roadway.

TABLE 29. Percentage of roadway segments in specific categories by state and roadway type

Variable/category	Minnesota					Michigan				
	2U	3T	4U	4D	5T	2U	3T	4U	4D	5T
On-street parking										
None	73	79	65	90	97	95	97	91	100	99
One Side	< 1	9	6	3	< 1	2	3	5		1
Both Sides	27	12	29	7	2	4		4		
Speed category										
Low (30 mph or less)	58	71	87	42	79	39	66	85	33	17
Intermediate (35 - 45 mph)	36	26	13	49	18	46	26	13	67	66
High (50 mph or more)	6	3	< 1	9	3	15	8	2		17
Lane width[a]										
9 ft or less			4			10	27	22	9	
10 ft	6	12	33	1		14	22	38	7	3
11 ft	5	4	28	25	31	39	10	33	49	64
12 ft	43	57	21	34	69	25	28	6	24	20
13 ft or more	46	27	14	40		12	13		11	13
Shoulder width/type[a]										
Curb	38	55	94	72	89	18	66	98	96	94
No Shoulder	7	7		1		9	5			
1-3 ft	7	5		1		25	6	2	4	3
4-6 ft	12	15	1	1	9	11	2			
7-9 ft	22	13	2	11	1	29	12			3
10+ ft	14	5	3	14		8	9			1

[a] For sites where lane or shoulder widths were measured.

TABLE 30. Descriptive statistics for continuous variables for intersections

Variable/statistic	Minnesota				North Carolina			
	3ST	3SG	4ST	4SG	3ST	3SG	4ST	4SG
Major-road ADT (veh/day)								
Mean	15,789	24,730	17,868	21,369	12,691	21,354	14,074	20,796
Median	15,123	20,975	13,689	20,130	10,431	21,275	14,549	16,934
Standard deviation	1,450	2,028	1,533	1,214	1,389	1,301	1,195	1,584
Minimum	3,471	7,983	4,484	4,550	732	6,555	366	5,572
Maximum	45,662	58,056	46,784	67,636	34,130	43,413	33,947	51,031
Minor-road ADT (veh/day)								
Mean	1,157	5,331	956	5,502	2,173	3,908	1,409	9,133
Median	605	3,842	605	4,273	1,190	3,176	641	7,949
Standard deviation	266	710	143	634	306	397	232	1,063
Minimum	84	599	556	222	183	758	92	686
Maximum	9,080	16,325	5,664	22,464	9,242	10,466	5,856	33,392

TABLE 31. Percentage of intersections in specific categories by state and intersection type

	Minnesota				North Carolina			
	3ST	3SG	4ST	4SG	3ST	3SG	4ST	4SG
Speed category								
Low (30 mph or less)	38	42	60	67	57	50	60	57
Intermediate (35 to 45 mph)	35	25	25	12	40	40	35	34
High (50 mph or more)	28	33	15	21	2	10	4	9
Left-turn signal phasing for major road approaches								
Protected		53		34		38		48
Protected/permissive		28		28		26		20
None		19		37		36		32
Intersection angle								
80 degrees or more	85	83	85	94	72	88	88	91
Less than 80 degrees	15	17	15	6	28	12	13	9
Character development								
Commercial	28	47	34	75	13	31	6	52
Industrial/institutional		8	2	3	9	5	17	7
Mixed	10	8	17	12	2	5		5
Residential	45	17	34	1	74	57	75	30
Other	18	19	13	9	2	2	2	7

CHAPTER 5.

BASE MODELS AND ADJUSTMENT FACTORS

This chapter presents the base models and adjustment factors developed for the HSM safety prediction methodology for urban and suburban arterials. Base models and adjustment factors for both roadway segments and intersections are presented. Chapter 7 shows how these base models and adjustment factors are incorporated in the HSM methodology.

ROADWAY SEGMENTS

Base models and adjustment factors for roadway segments are presented for multiple-vehicle nondriveway collisions, single-vehicle collisions, driveway-related collisions, vehicle-pedestrian collisions, and vehicle-bicycle collisions. The five types of roadway segments considered are those identified in Chapter 4 of this report:

- two-lane undivided arterials (2U)
- three-lane arterials including a center TWLTL (3T)
- four-lane undivided arterials (4U)
- four-lane divided arterials (4D)
- five-lane arterials including a center TWLTL (5T)

Multiple-Vehicle Nondriveway Collisions

Negative binomial regression models for multiple-vehicle nondriveway collisions were initially developed in the form shown below:

$$N_{brmv} = \exp\,(a + b\ \ln ADT + \ln L + c\ SW + d\ OSP) \qquad \textbf{(26)}$$

where:

N_{brmv}	=	predicted number of multiple-vehicle nondriveway collisions per year for nominal or base conditions
ADT	=	average daily traffic volume (veh/day) on roadway segment
L	=	length of roadway segment (mi)
SW	=	shoulder width (ft); average shoulder width if the two directions of travel differ
OSP	=	on-street parking factor (= 0 if curb parking is present on either side of street; = 1 if curb parking is not present)
a,b,c,d	=	regression coefficients

The independent variables included in these models, when statistically significant, in addition to ADT, which was always included, are:

- shoulder width
- on-street parking

In most cases, ADT and the other independent variables are significant at the 5-percent significance level or better.

The model coefficients are presented in Tables 32 and 33 for roadway segments in Minnesota and Michigan, respectively.

Additional variables that were considered in the development of the models shown in Tables 32 and 33 included:

- lane width
- speed category (low, intermediate, high)

No consistent relationship was found between lane width and safety (*16*). Therefore, lane width was not included in the model form shown in Equation (26).

TABLE 32. Initial models for multiple-vehicle nondriveway collisions on Minnesota roadway segments

Road type	No. of sites	Regression coefficient (standard error): Intercept (a)		ADT (b)		Shoulder width (c)		On-street parking (d)		Over-dispersion parameter (k)	R^2_{LR}
Total accidents											
2U	577	–12.85	(1.36)	1.55	(0.15)	–0.04	(0.01)	–0.84	(0.16)	0.81	0.26
3T	380	–12.85	(2.34)	1.52	(0.25)	–0.09	(0.02)			0.74	0.17
4U	742	–14.39	(1.10)	1.67	(0.11)	–0.08	(0.02)			0.76	0.26
4D	540	–12.70	(1.23)	1.47	(0.12)	–0.07	(0.01)			0.74	0.29
5T	205	–8.21	(3.31)	1.01	(0.34)	–0.30	(0.10)			0.80	0.16
Fatal-and-injury accidents											
2U	577	–11.39	(1.93)	1.20	(0.21)	–0.04	(0.02)			1.15	0.07
3T	380	–13.90	(3.24)	1.50	(0.35)	–0.08	(0.03)			0.65	0.10
4U	742	–13.29	(1.46)	1.43	(0.15)	–0.06	(0.03)			0.72	0.12
4D	540	–13.90	(1.70)	1.48	(0.17)	–0.07	(0.02)			0.92	0.18
5T	205	–9.70	(3.87)	1.03	(0.40)	–0.24	(0.13)			0.24	0.11
Property-damage-only accidents											
2U	577	–14.45	(1.59)	1.70	(0.17)	–0.05	(0.02)	–0.95	(0.18)	0.78	0.25
3T	380	–12.82	(2.56)	1.48	(0.27)	–0.09	(0.02)			0.77	0.15
4U	742	–15.87	(1.38)	1.79	(0.14)	–0.09	(0.03)			0.97	0.22
4D	540	–12.92	(1.33)	1.45	(0.13)	–0.07	(0.01)			0.80	0.26
5T	205	–7.91	(3.80)	0.94	(0.39)	–0.31	(0.11)			0.94	0.12

NOTE: Models are in the form shown in Equation (26).

TABLE 33. Initial models for multiple-vehicle nondriveway collisions on Michigan roadway segments

Road type	No. of sites	Regression coefficient (standard error): Intercept (a)		ADT (b)		Shoulder width (c)		On-street parking (d)		Over-dispersion parameter (k)	R^2_{LR}
Total accidents											
2U	590	–14.75	(0.88)	1.73	(0.09)	–0.02	(0.01)			0.61	
3T	115	–10.03	(1.39)	1.29	(0.15)	–0.04	(0.02)			0.44	
4U	471	–5.23	(1.32)	0.79	(0.13)	–0.31	(0.14)	–0.29	(0.18)	0.67	0.09
4D	140	–8.84	(1.89)	1.11	(0.19)					0.92	
5T	562	–8.68	(1.08)	1.14	(0.11)	–0.08	(0.02)			0.73	0.18
Fatal-and-injury accidents											
2U	590	–17.11	(1.31)	1.81	(0.13)					0.43	0.30
3T	115	–14.81	(2.73)	1.59	(0.28)					0.47	0.33
4U	471	–7.82	(1.98)	0.88	(0.20)					0.88	0.04
4D	140	–8.13	(2.28)	0.88	(0.27)					0.82	0.10
5T	562	–9.60	(1.26)	1.09	(0.12)	–0.07	(0.03)			0.64	0.13
Property-damage-only accidents											
2U	590	–14.51	(0.95)	1.68	(0.10)	–0.02	(0.01)			0.67	0.36
3T	115	–9.64	(1.42)	1.23	(0.15)	–0.06	(0.02)			0.43	0.41
4U	471	–5.39	(1.42)	0.78	(0.14)	–0.38	(0.14)	–0.38	(0.19)	0.70	0.08
4D	140	–9.91	(1.96)	1.20	(0.20)					0.90	0.22
5T	562	–9.02	(1.12)	1.15	(0.11)	–0.08	(0.02)			0.76	0.17

NOTE: Models are in the form shown in Equation (26); all sites are located in Oakland County, MI.

The investigation of the effect of speed on safety on urban and suburban arterials considered three speed categories based on posted speed limit: low speed [48 km/h (30 mph) or less]; intermediate speed [56 to 72 km/h (35 to 45 mph)]; and high speed [80 km/h (50 mph) or more]. There was a statistically significant relationship between these speed categories and safety in a number of regression models but, in nearly all cases, this relationship was in the opposite direction to that expected (i.e., higher accident frequencies on low-speed than on high-speed arterials). Several alternative modeling approaches to obtain a speed effect that was not counterintuitive were tried, but none were successful. These alternative modeling approaches focused on identifying other independent variables that might be correlated with speed and might explain the counterintuitive speed effect that was found. One hypothesis investigated was that driveways within a roadway segment might create sufficient turbulence in traffic flow to affect multiple-vehicle nondriveway collisions. Therefore, variables representing number of driveways and driveway density were introduced into the models, but counterintuitive speed effects were still obtained. Because the effect of the speed categories on safety was opposite to the direction expected and because the consideration of speed category typically increased R^2_{LR} for the models by only about 0.01, a decision was reached not to include speed category in the models. The relationship of speed and safety merits further investigation for future HSM editions.

Figure 10 illustrates the regression relationship presented on the first line of Table 33 on a scatter plot of the data from which it was developed. Both the plotted curve and the observed data points represent a two-lane undivided arterial with zero shoulder width (e.g., a curb-and-gutter cross section) and no on-street parking.

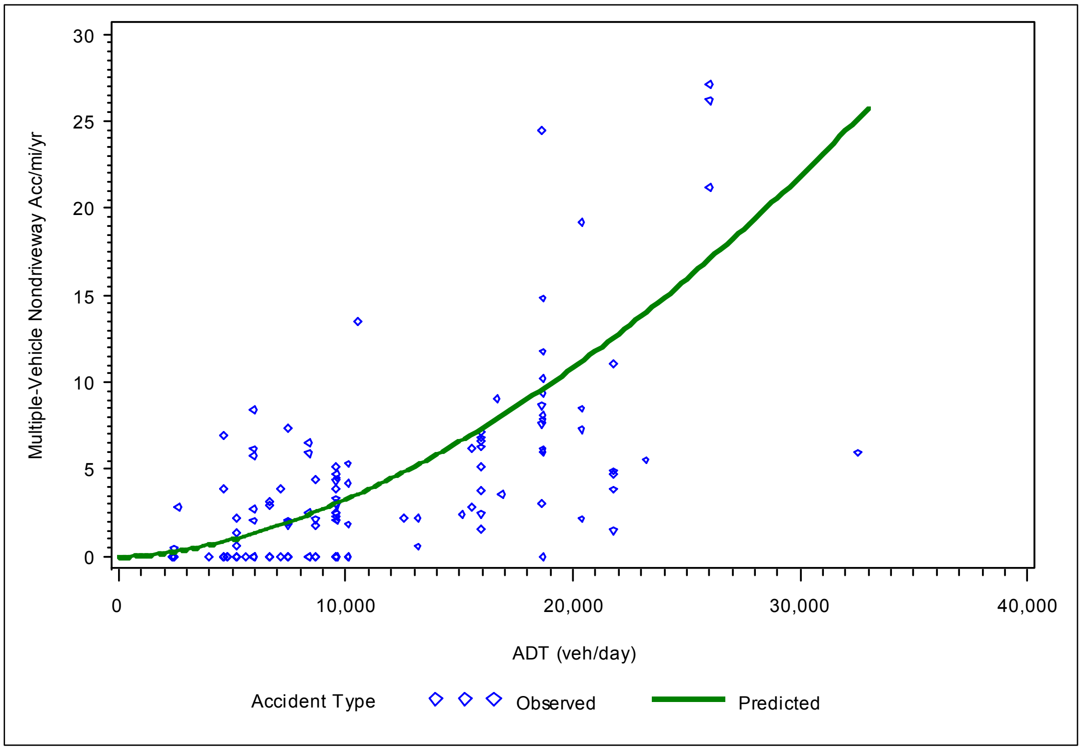

Figure 10. Example of regression relationship and observed data for two-lane roadways in Michigan with no shoulder and no on-street parking.

Table 34 presents comparable models to Tables 32 and 33 for the Minnesota and Michigan data combined. These models are in the same form as Equation (26), but were developed with a random state effect as explained in Chapter 3 of this report. Table 34 includes the magnitude of the variance of the state effect.

For application in the HSM methodology, the regression models for multiple-vehicle collisions shown in Table 34 were converted to nominal or base conditions and reduced to the form shown below:

$$N_{brmv} = \exp\,(a + b\ \ln ADT + \ln L) \qquad \textbf{(27)}$$

The nominal or base conditions include shoulder width of 1.8 m (6 ft) and no on-street parking. The random state effect combines the data for Minnesota and Michigan. Adjustment of the models for application in other states or in other years than those used in model development will be addressed through the calibration process. Table 35 presents the coefficients for the reduced models in the form shown in Equation (27). These models are illustrated in Figures 11 through 13.

TABLE 34. Initial models for multiple-vehicle nondriveway collisions on roadway segments for Minnesota and Michigan combined

Roadway type	Number of sites	Regression coefficient (standard error) Intercept (a)		ADT (b)		Shoulder width (c)		Curb parking (d)		State variance	Overdispersion parameter (k)	R_{LR}^2
Total accidents												
2U	1,167	–13.82	(0.84)	1.68	(0.08)	–0.03	(0.01)	–0.75	(0.12)	0.17	0.84	0.37
3T	480	–11.48	(1.32)	1.41	(0.13)	–0.07	(0.02)			0.24	0.66	0.46
4U	1,182	–10.84	(0.92)	1.33	(0.09)	–0.08	(0.02)	–0.18	(0.09	0.04	1.01	0.31
4D	680	–11.47	(1.17)	1.36	(0.12)	–0.07	(0.02)			0.03	1.32	0.35
5T	754	–9.39	(1.09)	1.17	(0.10)	–0.09	(0.02)			0.36	0.81	0.39
Fatal-and-injury accidents												
2U	1,167	–15.59	(1.10)	1.66	(0.12)	–0.03	(0.01)			0.03	0.65	0.02
3T	480	–15.71	(1.91)	1.69	(0.20)	–0.04	(0.02)			0.00	0.59	0.13
4U	1,182	–11.61	(1.08)	1.25	(0.11)	–0.06	(0.03)			0.00	0.99	0.13
4D	680	–11.96	(1.40)	1.28	(0.14)	–0.06	(0.02)			0.02	1.31	0.24
5T	754	–10.24	(1.17)	1.12	(0.11)	–0.08	(0.03)			0.23	0.62	0.23
Property-damage-only accidents												
2U	1,167	–14.08	(0.92)	1.69	(0.09)	–0.03	(0.01)	–0.86	(0.13)	0.25	0.87	0.33
3T	480	–11.01	(1.36)	1.33	(0.14)	–0.08	(0.02)			0.37	0.59	0.47
4U	1,182	–11.61	(1.01)	1.38	(0.10)	–0.10	(0.03)	–0.24	(0.10)	0.05	1.08	0.29
4D	680	–11.94	(1.25)	1.38	(0.12)	–0.07	(0.02)			0.11	1.34	0.41
5T	754	–9.66	(1.16)	1.17	(0.11)	–0.09	(0.03)			0.43	0.88	0.35

NOTE: Models are in the form shown in Equation (26).

TABLE 35. Base models for multiple-vehicle nondriveway collisions on roadway segments

Road type	Regression coefficients: Intercept (a)	Regression coefficients: ADT (b)	Overdispersion parameter (k)
Total accidents			
2U	–14.75	1.68	0.84
3T	–11.92	1.41	0.66
4U	–11.53	1.33	1.01
4D	–11.88	1.36	1.32
5T	–9.93	1.17	0.81
Fatal-and-injury accidents			
2U	–15.75	1.66	0.65
3T	–15.97	1.69	0.59
4U	–11.98	1.25	0.99
4D	–12.30	1.28	1.31
5T	–10.70	1.12	0.62
Property-damage-only accidents			
2U	–15.15	1.69	0.87
3T	–11.47	1.33	0.59
4U	–12.43	1.38	1.08
4D	–12.35	1.38	1.34
5T	–10.20	1.17	0.88

NOTE: Models are in the form shown in Equation (27).

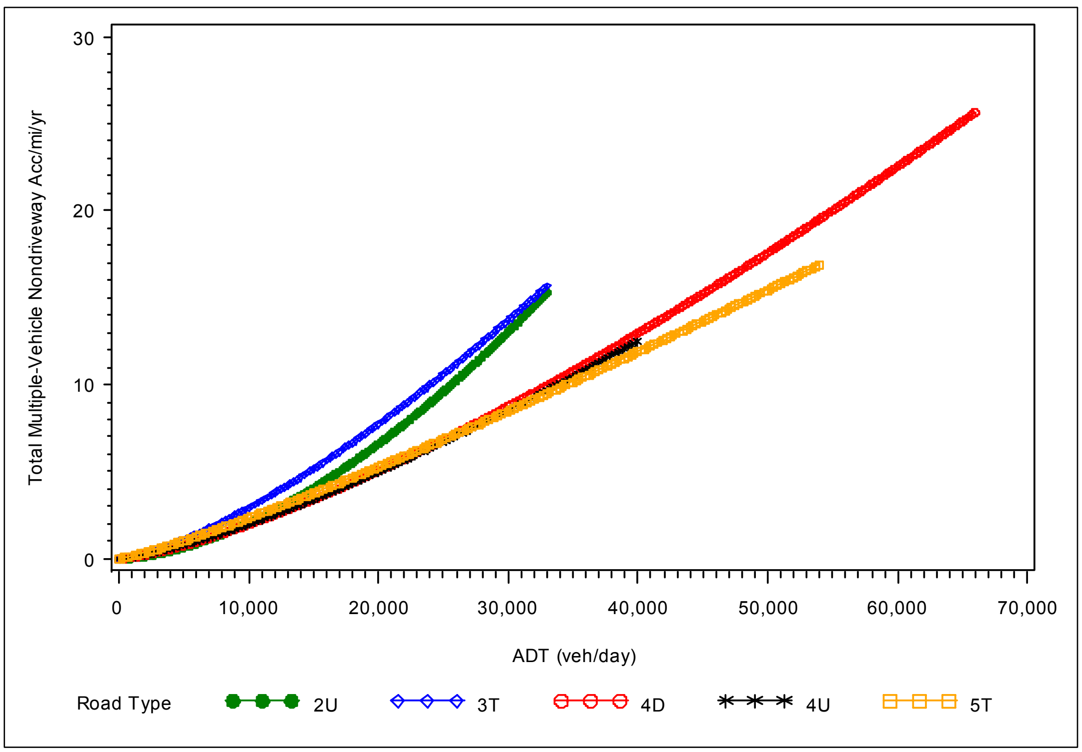

Figure 11. Plots for base models for total multiple-vehicle nondriveway collisions on roadway segments by roadway type.

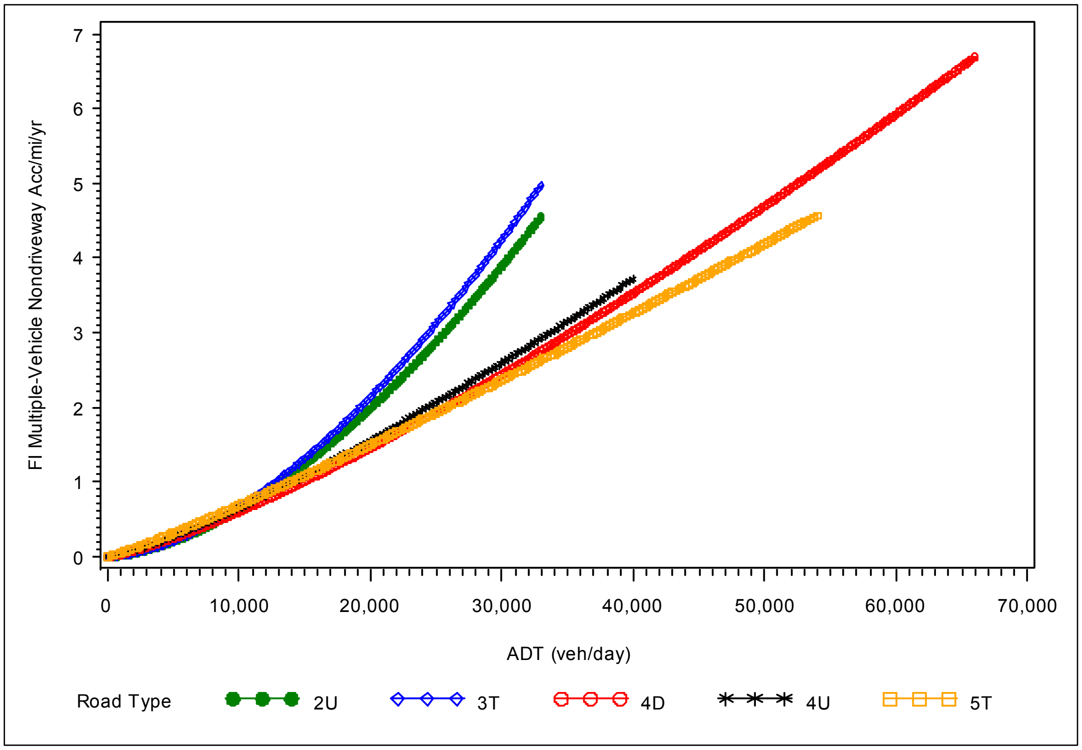

Figure 12. Plots for base models for fatal-and-injury multiple-vehicle nondriveway collisions on roadway segments by roadway type.

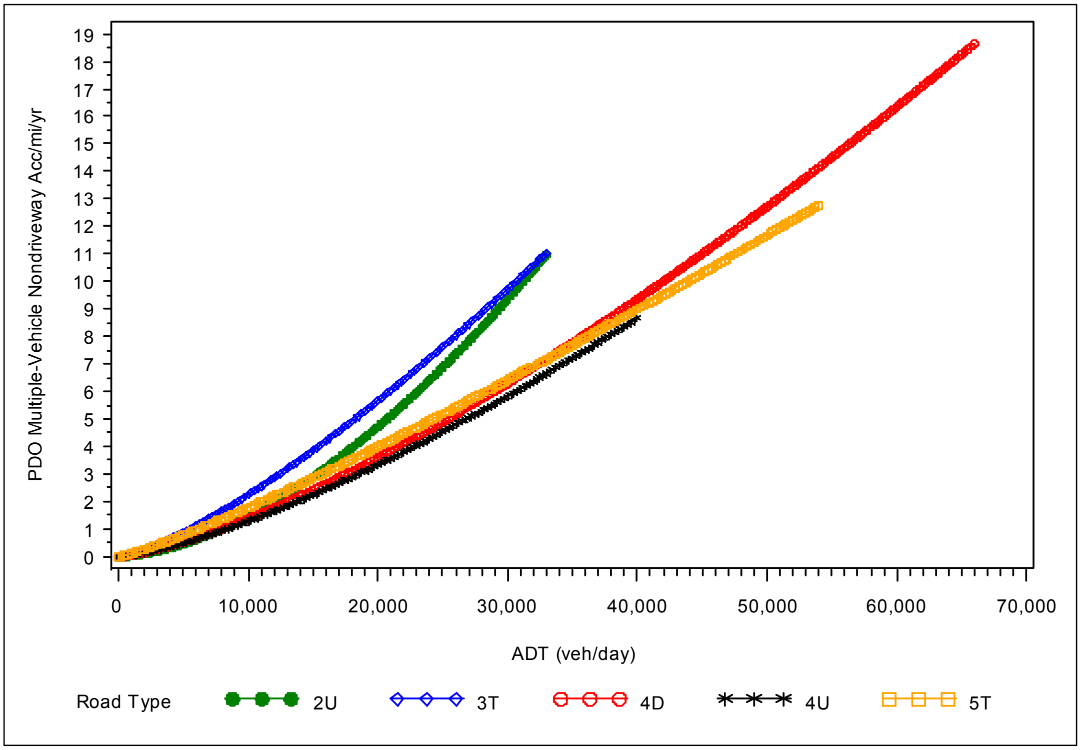

Figure 13. Plots for base models for property-damage-only multiple-vehicle nondriveway collisions on roadway segments by roadway type.

The values of R^2_{LR} for the base models, which represent the improvement of the model over an intercept-only model, range from 0.10 to 0.37. The only models with relatively low values of R^2_{LR} are those for fatal-and-injury accidents on roadway types with relatively few accidents.

The severity distribution for multiple-vehicle nondriveway collisions is shown in Table 36 based on combined data for Minnesota and Michigan.

TABLE 36. Severity distribution of multiple-vehicle collisions for roadway segments

Road type	No. of sites	Percentage of total accidents: Fatal and injury	Percentage of total accidents: Property damage only
Minnesota sites			
2U	577	29.4	70.6
3T	380	29.5	70.5
4U	742	30.7	69.3
4D	540	36.3	63.7
5T	205	30.6	69.4
Michigan sites			
2U	590	24.7	75.3
3T	115	20.4	79.6
4U	471	25.6	74.4
4D	140	19.4	80.6
5T	562	23.0	77.0
Combined sites			
2U	1,167	27.1	72.9
3T	495	25.0	75.0
4U	1,213	28.1	71.9
4D	680	27.9	72.1
5T	767	26.8	73.2

Single-Vehicle Collisions

Negative binomial regression models for single-vehicle collisions were initially developed in the form shown below:

$$N_{brsv} = \exp\,(a + b\ \ln ADT + \ln L + c\ SW + d\ OSP + e\ RHR) \qquad \textbf{(28)}$$

where:

N_{brsv} = predicted number of single-vehicle collisions per year for nominal for base conditions

RHR = roadside hazard rating for roadway segment (1 to 7 scale); average roadside hazard rating if the roadsides for the two directions of travel differ

a,...,e = regression coefficients

The independent variables included in these models, when statistically significant, in addition to ADT, which was always included, are:

- shoulder width
- on-street parking
- roadside hazard rating

The model coefficients are presented in Tables 37 and 38 for roadway segments in Minnesota and Michigan, respectively. As in the case of multiple-vehicle nondriveway collisions, no statistically significant effect was found for lane width and only counterintuitive effects were found for speed category.

Table 39 presents comparable models to Tables 37 and 38 for the Minnesota and Michigan data combined. These models are in the same form as Equation (28), but were developed with a random state effect as explained in Chapter 3 of this report. Table 39 includes the magnitude of the variance for the state effect.

For application in the HSM methodology, the regression models for single-vehicle collisions were converted to nominal or base conditions and reduced to the form shown below:

$$N_{brsv} = \exp\left(a + b\ \ln ADT + \ln L\right) \qquad \textbf{(29)}$$

The nominal or base conditions include shoulder width of 1.8 m (6 ft), no on-street parking, and a roadside hazard rating of 3. Table 40 presents the coefficients for these reduced models. These models are illustrated in Figures 14 through 16.

Statistically significant models for single-vehicle collisions were not found for all cases. Where no formal model is available, single-vehicle collisions can be estimated by a multiplicative factor as a proportion of multiple-vehicle nondriveway collisions, as follows:

$$N_{brsv} = N_{brmv}\ f_{rsv} \qquad \textbf{(30)}$$

where:

f_{rsv} = factor for single-vehicle collisions on a roadway segment as a proportion of multiple-vehicle nondriveway collisions

Table 41 presents values of f_{rsv} that can be applied where no model for single-vehicle collisions is available. A value for f_{rsv} equal to 0.24 implies that the average frequency of single collisions is 24 percent of the average frequency of multiple-vehicle nondriveway collisions. The magnitude of the values in Table 41 is such that the use of predictive models in the form of Equation (29) would be preferable to the use of tabulated values like those in Table 41. While the values of R^2_{LR} for the models presented in Tables 39 and 40 are quite low, use of these models allows single-vehicle collision frequencies to vary as a function of ADT. If factors like those shown in Table 41 were used, that ADT dependency would be lost. Therefore, a decision was reached to use the models shown in Tables 39 and 40 in the HSM methodology.

The severity distribution for single-vehicle collisions on roadway segments is presented in Table 42.

TABLE 37. Initial models for single-vehicle collisions on Minnesota roadway segments

Road type	No. of sites	Intercept (a)	ADT (b)	Shoulder width (c)	On-street parking (d)	Roadside hazard rating (e)	Over-dispersion parameter (k)	R^2_{LR}
		Regression coefficient (standard error)						
Total accidents								
2U	577	–3.35 (0.95)	0.43 (0.11)		–0.99 (0.17)	0.13 (0.05)	0.77	0.16
3T	380	–5.10 (2.18)	0.52 (0.24)				1.36	0.01
4U	742	–8.44 (1.30)	0.91 (0.14)		–0.73 (0.12)	0.12 (0.05)	0.64	0.14
4D	540	–6.10 (1.16)	0.60 (0.11)		–0.38 (0.20)	0.19 (0.05)	0.65	0.08
5T	205	–6.93 (3.15)	0.71 (0.33)				0.75	0.02
Fatal-and-injury accidents								
2U	577							
3T	380							
4U	742							
4D	540	–8.42 (1.79)	0.66 (0.17)			0.23 (0.07)	0.26	0.04
5T	205							
Property-damage-only accidents								
2U	577	–4.74 (1.06)	0.56 (0.12)		–1.11 (0.18)	0.14 (0.06)	0.74	0.17
3T	380	–6.97 (2.57)	0.68 (0.28)				1.71	0.02
4U	742	–9.48 (1.52)	0.99 (0.16)		–0.75 (0.14)	0.12 (0.05)	0.79	0.12
4D	540	–6.26 (1.34)	0.59 (0.13)		–0.48 (0.23)	0.18 (0.06)	0.82	0.06
5T	205	–7.07 (3.81)	0.69 (0.40)				1.10	0.02

NOTE: Models are in the form shown in Equation (28).

TABLE 38. Initial models for single-vehicle collisions on Michigan roadway segments

		Regression coefficient (standard error)						
Road type	No. of sites	Intercept (a)	ADT (b)	Shoulder width (c)	On-street parking (d)	Roadside hazard rating (e)	Over-dispersion parameter (k)	R^2_{LR}
Total accidents								
2U	590	–4.99 (0.83)	0.61 (0.09)				0.64	0.08
3T	115	–4.59 (2.22)	0.48 (0.23)				0.57	0.04
4U	471							
4D	140							
5T	562	–4.52 (1.25)	0.51 (0.12)	–0.05 (0.03)			0.47	0.03
Fatal-and-injury accidents								
2U	590	–3.95 (1.34)	0.32 (0.14)				0.11	0.01
3T	115	–11.04 (4.88)	1.01 (0.50)				0.65	0.05
4U	471							
4D	140							
5T	562							
Property-damage-only accidents								
2U	590	–5.60 (0.90)	0.66 (0.09)				0.69	0.08
3T	115	–3.48 (2.12)	0.34 (0.25)				0.62	0.02
4U	471							
4D	140							
5T	562	–5.66 (1.35)	0.60 (0.13)	–0.05 (0.03)			0.51	0.04

NOTE: Models are in the form shown in Equation (28); all sites are located in Oakland County, MI.

TABLE 39. Initial models for single-vehicle collisions on roadway segments in Minnesota and Michigan combined

Road type	Number of sites	Regression coefficient (standard error): Intercept (a)	AADT (b)	Curb parking (c): None	On-street parking (d)	State variance	Over-dispersion parameter (k)	R^2_{LR}
Total accidents								
2U	1,167	–4.37 (0.72)	0.56 (0.07)	–0.74 (0.12)	0.11 (0.03)	0.14	0.83	0.13
3T	495	–5.26 (1.74)	0.54 (0.19)			0.00	1.37	0.05
4U	1,213	–7.14 (1.16)	0.81 (0.12)	–0.75 (0.11)		0.04	0.91	0.10
4D	680	–4.30 (1.07)	0.47 (0.10)	–0.41 (0.21)	0.12 (0.04)	0.17	0.86	0.19
5T	767	–5.05 (1.19)	0.54 (0.12)			0.11	0.52	
Fatal-and-injury accidents								
2U	1,167	–3.32 (1.03)	0.23 (0.11)	–0.29 (0.21)	0.11 (0.03)	0.14	0.83	0.13
3T	495	–5.89 (2.64)	0.47 (0.28)			0.00	1.37	0.05
4U	1,213	–6.60 (1.83)	0.61 (0.19)	–0.67 (0.18)		0.04	0.91	0.10
4D	680	–8.47 (1.57)	0.66 (0.15)		0.12 (0.04)	0.17	0.86	0.19
5T	767	–4.66 (1.71)	0.35 (0.17)			0.11	0.52	
Property-damage-only accidents								
2U	1,167	–5.30 (0.79)	0.64 (0.08)	–0.85 (0.13)	0.12 (0.04)	0.17	0.87	0.17
3T	495	–5.81 (2.06)	0.56 (0.22)			0.00	1.93	0.06
4U	1,213	–7.61 (1.27)	0.84 (0.13)	–0.79 (0.12)		0.02	0.97	0.08
4D	680	–4.16 (1.22)	0.45 (0.11)	–0.51 (0.24)	0.10 (0.05)	0.29	1.06	0.17
5T	767	–6.06 (1.31)	0.61 (0.13)			0.16	0.55	

NOTE: Models are in the form shown in Equation (28).

TABLE 40. Base models for single-vehicle collisions on roadway segments

Road type	Regression coefficients: Intercept (a)	Regression coefficients: ADT (b)	Overdispersion parameter (k)
Total accidents			
2U	–5.00	0.56	0.83
3T	–5.26	0.54	1.37
4U	–7.89	0.81	0.91
4D	–4.59	0.47	0.86
5T	–5.05	0.54	0.52
Fatal-and-injury accidents			
2U	–3.49	0.23	0.50
3T	–5.89	0.47	1.06
4U	–7.27	0.61	0.54
4D	–8.25	0.66	0.28
5T	–4.66	0.35	0.36
Property-damage-only accidents			
2U	–6.04	0.64	0.87
3T	–5.81	0.56	1.93
4U	–8.40	0.84	0.97
4D	–4.58	0.45	1.06
5T	–6.06	0.61	0.55

NOTE: Models are in the form shown in Equation (29).

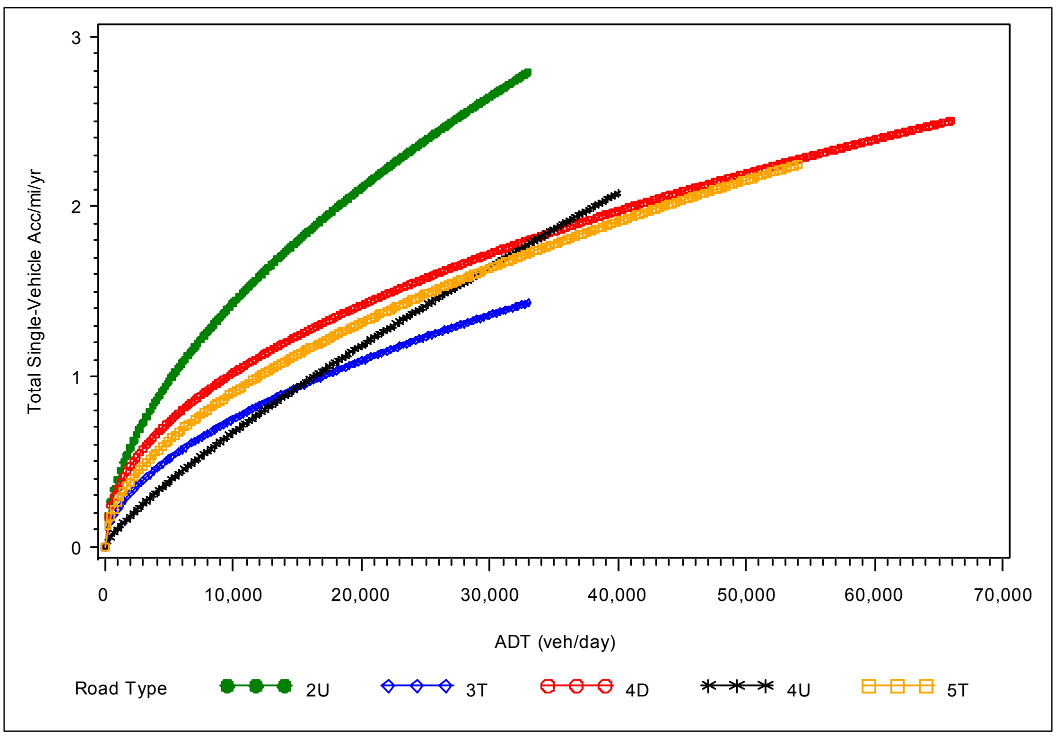

Figure 14. Plots for base models for total single-vehicle collisions on roadway segments by roadway type.

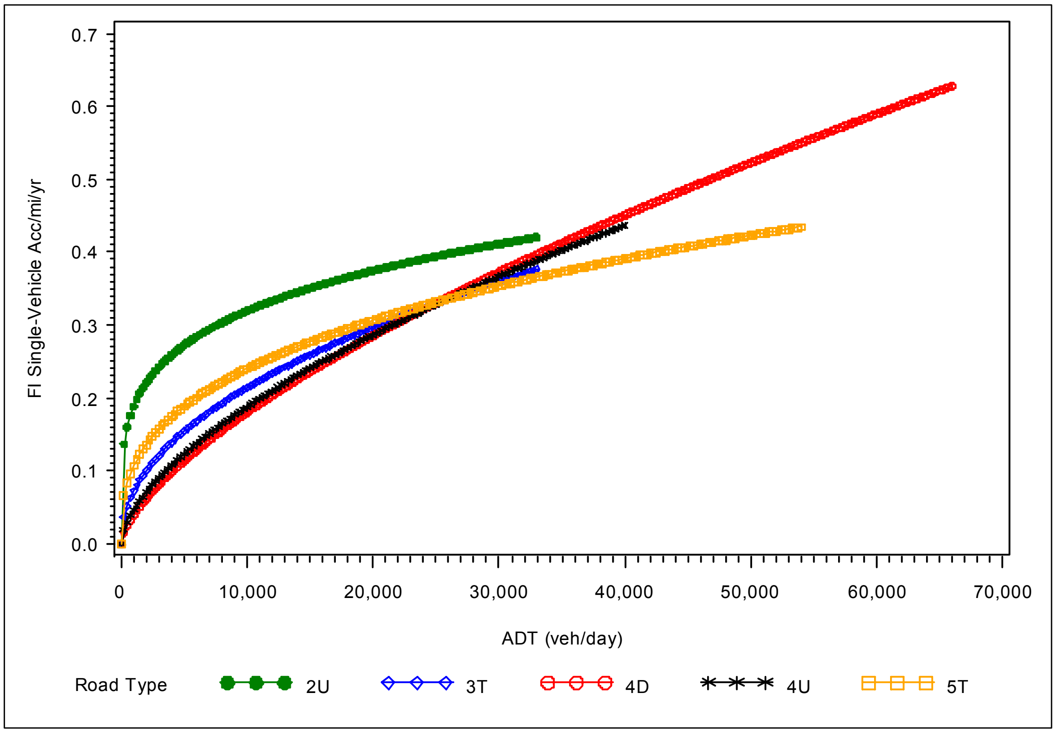

Figure 15. Plots for base models for fatal-and-injury single-vehicle collisions on roadway segments by roadway type.

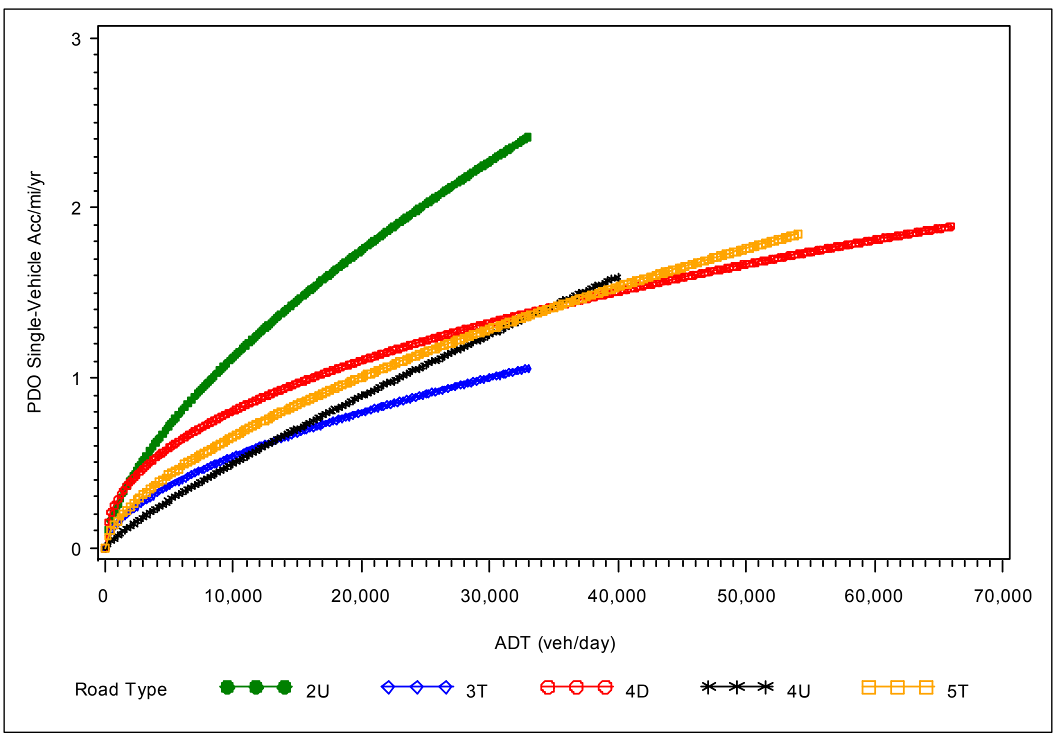

Figure 16. Plots for base models for property-damage-only single-vehicle collisions on roadway segments by roadway type.

TABLE 41. Single-vehicle collision factors for roadway segments

Road type	Single-vehicle collision factor (f_{rsv}) Minnesota	Michigan	Combined
Total accidents			
2U	0.41	0.17	0.29
3T	0.19	0.31	0.25
4U	0.20	0.10	0.15
4D	0.15	0.16	0.16
5T	0.17	0.09	0.13
Fatal-and-injury accidents			
2U	0.37	0.23	0.30
3T	0.18	0.12	0.15
4U	0.17	0.10	0.14
4D	0.13	0.12	0.13
5T	0.19	0.07	0.13
Property-damage-only accidents			
2U	0.43	0.33	0.38
3T	0.20	0.10	0.15
4U	0.21	0.12	0.17
4D	0.17	0.16	0.17
5T	0.17	0.09	0.13

TABLE 42. Severity distribution of single-vehicle collisions for roadway segments

Road type	No. of sites	Percentage of total accidents: Fatal and injury	Property damage only
Minnesota sites			
2U	577	24.3	75.7
3T	380	27.3	72.7
4U	742	25.3	74.7
4D	540	29.5	70.5
5T	205	31.9	68.1
Michigan sites			
2U	590	16.4	83.6
3T	115	25.3	74.7
4U	471	21.5	78.5
4D	140	14.2	85.8
5T	562	16.8	83.2
Combined sites			
2U	1,167	20.4	79.6
3T	495	26.3	73.7
4U	1,213	23.4	76.6
4D	680	21.8	78.2
5T	767	24.4	75.6

Driveway-Related Collisions

The models presented above for multiple-vehicle collisions addressed only collisions that were not related to driveways. Driveway-related accidents are also generally multiple-vehicle collisions, but are addressed separately because the frequency of driveway-related accidents on an arterial roadway segment depends on the number and type of driveways.

Seven specific driveway types were considered in modeling: These are:

- Major commercial driveways
- Minor commercial driveways
- Major industrial/institutional driveways
- Minor industrial/institutional driveways
- Major residential driveways
- Minor residential driveways
- Other driveways

Major driveways are those that serve sites with 50 or more parking spaces. Minor driveways are those that serve sites with less than 50 parking spaces. Commercial driveways provide access to establishments that serve retail customers. Residential driveways serve single- and multiple-family dwellings. Industrial/institutional driveways serve factories, warehouses, schools, hospitals, churches, offices, public facilities, and other places of employment.

The modeling of driveway-related collisions was based on the driveway types listed above. Driveway volume was not considered in modeling for two reasons. First, no data on driveway volumes were available for the study sites. Second, even if a model with driveway volumes had been developed, driveway volumes are seldom available to highway agencies that will use the HSM. It would not be desirable to create an HSM methodology that cannot be applied by most users.

The base models for driveway-related collisions are of the form:

$$N_{brdwy} = a(ADT/10{,}000)^{b} \, (c\, D_{majc} + d\, D_{minc} + e\, D_{maji} + f\, D_{mini} + g\, D_{majr} + h\, D_{minr} + i\, D_{oth}) \qquad \textbf{(31)}$$

where:

N_{brdwy}	=	predicted number of driveway-related collisions per year on a roadway segment
D_{majc}	=	number of major commercial driveways within the roadway segment
D_{minc}	=	number of minor commercial driveways within the roadway segment
D_{maji}	=	number of major industrial/institutional driveways
D_{mini}	=	number of minor industrial/institutional driveways
D_{majr}	=	number of major residential driveways
D_{minr}	=	number of minor residential driveways
D_{oth}	=	number of other driveways
a, ..., i	=	regression coefficients

Table 43 presents values for model coefficient a, which represents the intercept; coefficient b, which represents the exponent of the traffic volume factor; and coefficients c through i, that are proportional to the average annual accident frequencies for driveways of specific types. The factors in Table 43 were derived with multinomial regression from Minnesota data. The model form shown in Equation (31), confirmed by the models shown in Table 43, indicates that collision frequency for a driveway depends on the traffic volume on the arterial. The collision frequency for a driveway should also logically depend on the driveway volume. However, because no driveway volume data are available, the effect of driveway volume is represented indirectly by the major and minor driveway categories. For example, the comparison between major and minor commercial driveways and major and minor industrial driveways for two-lane arterials in Table 43 indicates that, on the average, major driveways experience 5.3 times as many accidents as minor driveways. As explained in Chapter 3 of this report, the modeling approach used for driveway accidents constrained the value of the regression coefficients, c through i, to be not less than 1. In those cases where the accident data were too sparse for the model to converge, the value of the coefficient was set equal to 1.

The model shown in Table 43 for two-lane undivided arterials appears relatively good except for the coefficient for major residential driveways (*g*), which was not able to be fit due to scarcity of data. This coefficient was estimated as 5.3 times the coefficient for minor residential driveways, based on the average noted above.

The models for four-lane undivided and four-lane divided arterials in Table 43 appear to provide a reasonable representation of the effect of ADT on driveway accidents, but have some inconsistent coefficient values (e.g., accident frequencies for minor commercial driveways that are greater than those for major commercial driveways). A decision was reached to use the intercept and ADT coefficients derived from the data and to assume that the coefficients for individual driveway types were proportional to those for two-lane undivided arterials.

The driveway coefficients for four-lane divided highways are relatively low. Particularly for minor driveways, these probably represent driveways that are not opposite a median opening which, therefore, have right-in, right-out operation. No formal adjustment for the presence or absence of a median opening at a driveway on a divided highway has yet been developed, but this is being considered.

The models for three-lane arterials with center two-way left lanes, and especially for five-lane arterials with center two-way left-turn lanes, each fit the data poorly, had inconsistent coefficients, or both. It was decided that the available data were not sufficient to support models for these roadway type, so models were derived from those from the equivalent undivided facilities using an interim AMF of 0.65 for two-way left-turn lanes.

The coefficients of the models shown in Equation (31) have been modified as shown above and have been recast by multiplying coefficients *c* through *i* by coefficient *a* from Equation (31), and normalizing all of the coefficients to an ADT of 15,000 veh/day, which is close to the average ADT for all five roadway types. In this form, the coefficients N_j shown in

TABLE 43. Initial models for driveway-related collisions based on Minnesota data

Road type	Total number of driveways	Regression coefficients for intercept, traffic volume, and specific driveway types: Intercept (a)	ADT (b)	Major commercial (c)	Minor commercial (d)	Major industrial/ institutional (e)	Minor industrial/ institutional (f)	Major residential (g)	Minor residential (h)	Other (i)	Over-dispersion parameter (k)
2U	1,193	0.0165	1.000	10.18	3.25	11.07	1.47	1.00	1.00	1.63	0.81
3T	651	0.0025	1.000	28.14	46.75	19.47	10.28	64.89	1.00	17.99	1.10
4U	1,442	0.0123	1.172	3.56	4.21	22.35	1.77	2.43	1.00	0.00	0.81
4D	236	0.0033	1.106	7.45	8.88	1.00	1.00	1.00	1.00	1.00	1.39
5T	103	0.0165	2.053	4.48	1.00	1.00	1.00	1.00	1.00	1.00	0.10

NOTE: Models are in the form shown in Equation (31). Coefficients c through g and i are expressed in a form proportional to coefficient h. Thus, a coefficient value of 1.0 means the accident frequency for the driveway type in question did not differ from the accident frequency for minor residential driveways.

Table 44 each represent the average number of accidents per driveway per year for specific driveway types, an arterial of the specified type for an ADT of 15,000 veh/day. The normalization or centering to an ADT of 15,000 veh/day was done to facilitate comparisons between values of the driveway coefficients across roadway types. Equation (32) can be used to estimate the total driveway-related accident frequency per year for any specific mix of driveway types on a specific roadway type at any specific ADT level:

$$N_{brdwy} = \sum_{\text{all driveway types}} n_j \, N_j \left(\frac{ADT}{15{,}000} \right)^b \qquad (32)$$

where:

n_j = number of driveways of type j within the roadway segment

N_j = base number of accidents per driveway per year for driveway type j from Table 44

The severity proportions shown in Table 44 can be used to subdivide N_{brdwy} into components for fatal-and-injury and property-damage-only accidents.

TABLE 44. Driveway factors for Minnesota roadway segments

Driveway type	Roadway type: Two-lane undivided	Three-lane with TWLTL	Four-lane undivided	Four-lane divided	Five-lane with TWLTL
Average annual accident frequency per driveway (N_j)					
Major commercial	0.252	0.164	0.202	0.053	0.131
Minor commercial	0.080	0.052	0.064	0.017	0.042
Major industrial/institutional	0.274	0.178	0.220	0.057	0.143
Minor industrial/institutional	0.036	0.024	0.029	0.008	0.019
Major residential	0.132	0.086	0.106	0.028	0.069
Minor residential	0.025	0.016	0.020	0.005	0.013
Other	0.040	0.026	0.032	0.008	0.021
ADT coefficient (b)					
All driveways	1.000	1.000	1.172	1.106	1.172
Proportion of fatal-and-injury accidents (f_{dwy})					
All driveways	0.323	0.243	0.342	0.284	0.269
Proportion of property-damage-only accidents					
All driveways	0.677	0.757	0.658	0.716	0.731

NOTE: Major driveways are those that serve sites with 50 or more parking spaces. Minor driveways are those that serve sites with less than 50 parking spaces.

Vehicle-Pedestrian Collisions

Table 45 presents adjustment factors for the average frequency of vehicle-pedestrian collisions on arterial roadway segments. The table includes values for Minnesota, Michigan, and both data sets combined. Accident counts for multiple-vehicle nondriveway collisions, single-vehicle collisions, and driveway-related collisions can be multiplied by the factor shown in Table 45 to reflect the average frequency of vehicle-pedestrian collisions on arterial roadway segments. The factors in Table 45 incorporate vehicle-pedestrian collisions, as well as any bicycle-pedestrian collisions that may have occurred.

TABLE 45. Pedestrian safety adjustment factors for roadway segments in Minnesota and Michigan

Road type	Pedestrian safety adjustment factor (f_{pedr})		
	Minnesota	Michigan	Combined
2U	0.018	0.003	0.011
3T	0.017	0.001	0.009
4U	0.027	0.008	0.017
4D	0.012	0.006	0.009
5T	0.021	0.004	0.012

The pedestrian factors in Table 45 for Minnesota and Michigan differ substantially. This probably implies that the pedestrian volumes are greater on the roadway segments studied in Minnesota than in the area studied in Michigan, although there are no data available to demonstrate this. Table 46 suggests a combined table for use in the HSM methodology based on the assumption that the Minnesota data represent a mix of urban and suburban conditions, while the Michigan data are nearly entirely suburban.

Table 46 will eventually be replaced with a pedestrian safety prediction methodology which is under development in a newly-added portion of NCHRP Project 17-26.

TABLE 46. Pedestrian safety adjustment factors for urban and suburban areas

Road type	Pedestrian safety adjustment factor (f_{pedr})	
	Urban	Suburban
2U	0.033	0.003
3T	0.034	0.001
4U	0.045	0.008
4D	0.019	0.006
5T	0.039	0.004

NOTE: These factors apply to the methodology for predicting total accidents (all severity levels combined). All vehicle-pedestrian collisions resulting from this adjustment factor should be treated as fatal-and-injury accidents and none as property-damage-only accidents.

Vehicle-Bicycle Collisions

Table 47 presents adjustment factors for the average frequency of vehicle-bicycle collisions on arterial roadway segments. The table includes values for Minnesota, Michigan, and both data sets combined. The factors shown in Table 47 are multiplied by the frequency of all other collision types, except vehicle-pedestrian collisions, to estimate the average frequency of vehicle-bicycle collisions.

TABLE 47. Bicycle safety adjustment factors for roadway segments

Road type	Bicycle safety adjustment factor (f_{biker})		
	Minnesota	Michigan	Combined
2U	0.010	0.005	0.008
3T	0.014	0.002	0.008
4U	0.019	0.010	0.015
4D	0.011	0.011	0.011
5T	0.013	0.003	0.008

Analogous to the discussion of the pedestrian factors in Table 45, the bicycle factors in Table 47 differ substantially between Minnesota and Michigan. This may reflect more bicycling in Minnesota than in Michigan. Table 48 has been developed in a manner analogous to Table 46 on the assumption that the Minnesota data represent a mix of urban and suburban conditions, while the Michigan data are nearly entirely suburban.

Table 48 is intended for use in the HSM methodology for the first edition, as there is no work currently planned for a bicycle safety prediction methodology analogous to the pedestrian safety prediction methodology currently under development.

TABLE 48. Bicycle safety adjustment factors for roadway segments in urban and suburban areas

Road type	Bicycle safety adjustment factor (f_{biker})	
	Urban	Suburban
2U	0.015	0.005
3T	0.026	0.002
4U	0.029	0.010
4D	0.011	0.011
5T	0.023	0.003

NOTE: These factors apply to the methodology for predicting total accidents (all severity levels combined). All vehicle-bicycle collisions resulting from this adjustment factor should be treated as fatal-and-injury accidents and none as property-damage-only accidents.

INTERSECTIONS

Base models and adjustment factors for intersections are presented for multiple-vehicle collisions, single-vehicle collisions, vehicle-pedestrian collisions, and vehicle-bicycle collisions. The four types of intersections considered are those identified in Chapter 4 of this report:

- three-leg intersections with STOP control on the minor-road approach (3ST)
- three-leg signalized intersections (3SG)
- four-leg intersections with STOP control on the minor-road approaches (4ST)
- four-leg signalized intersections (4SG)

Multiple-Vehicle Collisions

Negative binomial regression models for multiple-vehicle collisions were developed in the form shown below:

$$N_{bimv} = \exp\left(a + b\ \ln ADT_{maj} + c\ \ln ADT_{min}\right) \tag{33}$$

where:

N_{bimv} = predicted number of multiple-vehicle collisions per year for nominal or base conditions
ADT_{maj} = average daily traffic volume (veh/day) for major road
ADT_{min} = average daily traffic volume (veh/day) for minor road
a, b, c = regression coefficients

No independent variables other than major- and minor-road traffic volume were statistically significant. The model coefficients are presented in Tables 49 and 50. Some models with significance levels that exceed 0.20 were used because no better model was available. Such cases are identified in the Notes column of each table.

TABLE 49. Initial models for multiple-vehicle collisions at Minnesota intersections

Intersection type	No. of sites	Regression coefficient (standard error): Intercept (a)	Regression coefficient (standard error): ADT_{maj} (b)	Regression coefficient (standard error): ADT_{min} (c)	Over-dispersion parameter (k)	R^2_{LR}	Notes
Total accidents							
3ST	40	–21.27 (5.05)	1.70 (0.42)	0.65 (0.28)	1.10	0.37	
3SG	36	–11.74 (2.79)	1.14 (0.25)	0.17 (0.12)	0.41	0.38	
4ST	48	–10.39 (2.09)	0.93 (0.17)	0.24 (0.15)	0.20	0.42	
4SG	67	–8.44 (2.08)	0.81 (0.22)	0.22 (0.08)	0.51	0.32	
Fatal-and-injury accidents							
3ST	40	–18.45 (5.34)	1.38 (0.44)	0.54 (0.30)	0.85	0.26	
3SG	36	–10.71 (2.74)	0.98 (0.25)	0.12 (0.12)	0.26	0.34	0.30[a]
4ST	48	–12.55 (2.68)	0.96 (0.22)	0.36 (0.19)	0.17	0.34	
4SG	67	–12.66 (2.20)	1.10 (0.23)	0.25 (0.08)	0.36	0.41	
Property-damage-only accidents							
3ST	40	–23.47 (6.11)	1.87 (0.52)	0.67 (0.31)	1.26	0.34	
3SG	36	–12.46 (3.04)	1.16 (0.28)	0.18 (0.13)	0.46	0.35	
4ST	48	–9.51 (2.51)	0.87 (0.21)	0.13 (0.19)	0.29	0.29	0.49[a]
4SG	67	–7.56 (2.14)	1.69 (0.23)	0.21 (0.09)	0.55	0.25	

[a] Minor-road ADT significance level.
NOTE: All models are in the form shown in Equation (33).

TABLE 50. Initial models for multiple-vehicle collisions at North Carolina intersections

Intersection type	No. of sites	Regression coefficient (standard error) Intercept (a)	ADT$_{maj}$ (b)	ADT$_{min}$ (c)	Over-dispersion parameter (k)	R^2_{LR}	Notes
Total accidents							
3ST	47	–12.22 (1.54)	1.08 (0.14)	0.37 (0.14)	0.51	0.65	
3SG	42	–11.68 (3.67)	1.05 (0.32)	0.39 (0.24)	0.87	0.24	
4ST	48	–7.95 (1.49)	0.76 (0.15)	0.26 (0.13)	0.66	0.40	
4SG	44	–13.69 (2.64)	1.38 (0.28)	0.28 (0.17)	0.69	0.44	
Fatal-and-injury accidents							
3ST	47	–13.11 (2.09)	1.18 (0.19)	0.22 (0.16)	0.55	0.54	
3SG	42	–10.60 (3.33)	0.99 (0.29)	0.21 (0.22)	0.67	0.22	0.35[a]
4ST	48	–10.23 (2.00)	0.90 (0.21)	0.25 (0.15)	0.79	0.36	
4SG	44	–13.04 (2.27)	1.33 (0.24)	0.15 (0.14)	0.46	0.46	0.29[a]
Property-damage-only accidents							
3ST	47	–14.45 (1.85)	1.15 (0.16)	0.50 (0.14)	0.47	0.64	
3SG	42	–13.40 (3.58)	1.10 (0.31)	0.49 (0.23)	0.77	0.28	
4ST	48	–8.07 (1.49)	0.71 (0.15)	0.26 (0.11)	0.51	0.41	
4SG	44	–14.87 (2.46)	1.39 (0.26)	0.36 (0.16)	0.60	0.50	

[a] Minor-road ADT significance level.

NOTE: All models are in the form shown in Equation (33); all intersections are located in Charlotte, NC.

Table 51 presents comparable models to Tables 49 and 50 for the Minnesota and North Carolina data combined. These models are in the same form as Equation (33), but were developed with a random state effect as explained in Chapter 3 of this report. Table 51 includes the magnitude of the variance for the state effect. These models are illustrated in Figures 17 through 19. The plots for all intersection types are based on an assumed value of 3,000 veh/day for minor-road traffic volume.

TABLE 51. Base models for multiple-vehicle collisions at intersections in Minnesota and North Carolina combined

Intersection Types	Number of Sites	Regression coefficient (standard error) Intercept (a)	ADT$_{maj}$ (b)	ADT$_{min}$ (c)	State variance	Over-dispersion parameter (k)	R_{LR}^2
Total accidents							
3ST	97	–13.39 1.81	1.11 0.16	0.41 0.13	0.57	0.80	0.46
3SG	78	–11.63 1.73	1.11 0.15	0.26 0.09	0.52	0.33	0.53
4ST	96	–8.97 1.26	0.82 0.11	0.25 0.09	0.37	0.40	0.53
4SG	111	–10.63 1.44	1.07 0.14	0.23 0.06	0.44	0.39	0.61
Fatal-and-injury accidents							
3ST	97	–14.04 2.15	1.16 0.18	0.30 0.14	0.67	0.69	0.40
3SG	78	–11.08 1.85	1.02 0.17	0.17 0.10	0.42	0.30	0.48
4ST	96	–11.20 1.66	0.93 0.15	0.28 0.10	0.53	0.48	0.43
4SG	111	–12.78 1.50	1.18 0.15	0.22 0.07	0.58	0.33	0.56
Property-damage-only accidents							
3ST	97	–15.41 2.04	1.20 0.18	0.51 0.14	0.44	0.77	0.30
3SG	78	–12.74 1.88	1.14 0.17	0.30 0.10	0.57	0.36	0.50
4ST	96	–8.81 1.33	0.77 0.12	0.23 0.09	0.28	0.40	0.43
4SG	111	–10.66 1.52	1.02 0.15	0.24 0.07	0.38	0.44	0.58

NOTE: All models are in the form shown in Equation (33).

The values of R^2_{LR} for the models shown in Table 51 are relatively high, with the proportion in the variation in accident frequency explained by the models ranging from 0.30 to 0.61.

The severity distribution for multiple-vehicle intersection collisions is shown in Table 52 based on combined data for Minnesota and North Carolina.

TABLE 52. Severity distribution of multiple-vehicle collisions for intersections

Intersection type	No. of sites	Percentage of total accidents: Fatal and injury	Percentage of total accidents: Property damage only
Minnesota sites			
3ST	40	35.0	65.0
3SG	36	37.2	62.8
4ST	48	35.2	64.8
4SG	67	32.9	67.1
Michigan sites			
3ST	47	36.9	63.1
3SG	42	34.2	65.8
4ST	48	41.5	58.5
4SG	44	33.6	66.4
Combined sites			
3ST	87	36.0	64.0
3SG	78	35.7	64.3
4ST	96	38.4	61.6
4SG	111	33.3	66.7

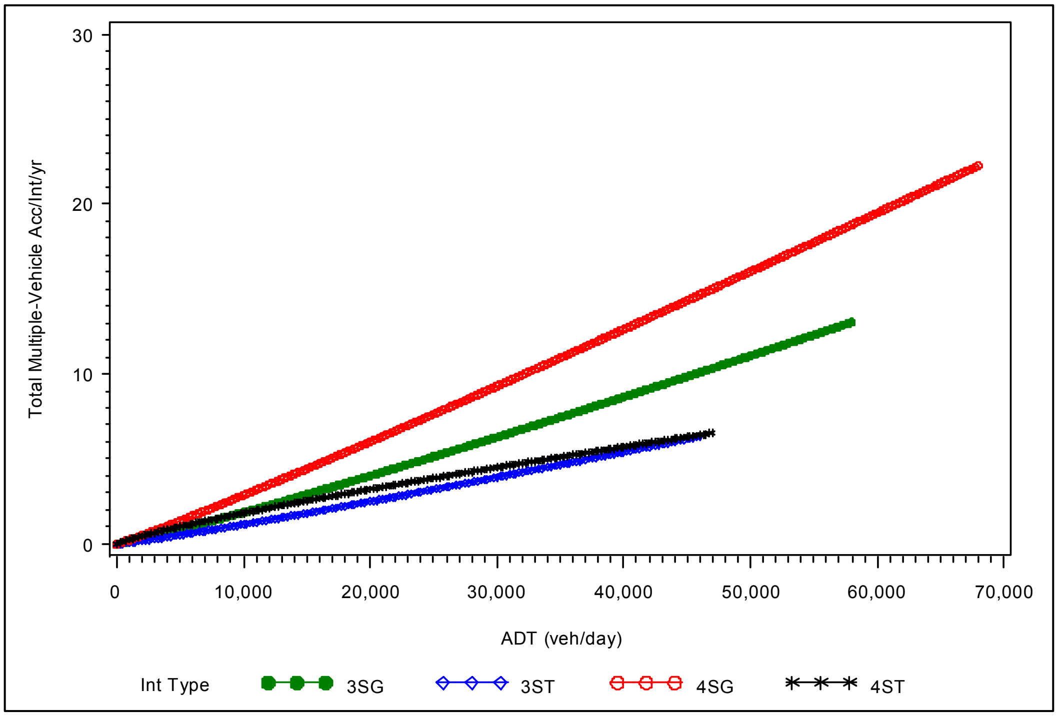

Figure 17. Plots for base models for total multiple-vehicle collisions at intersections by intersection type.

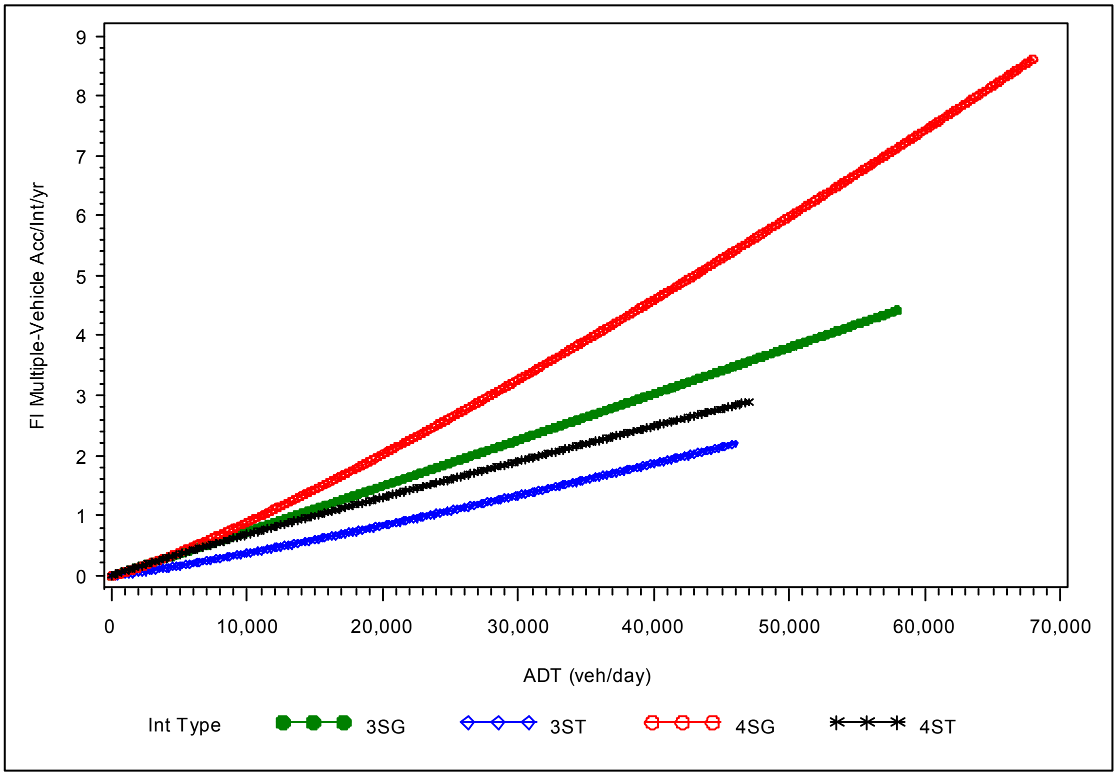

Figure 18. Plots for base models for fatal-and-injury multiple-vehicle collisions at intersections by intersection type.

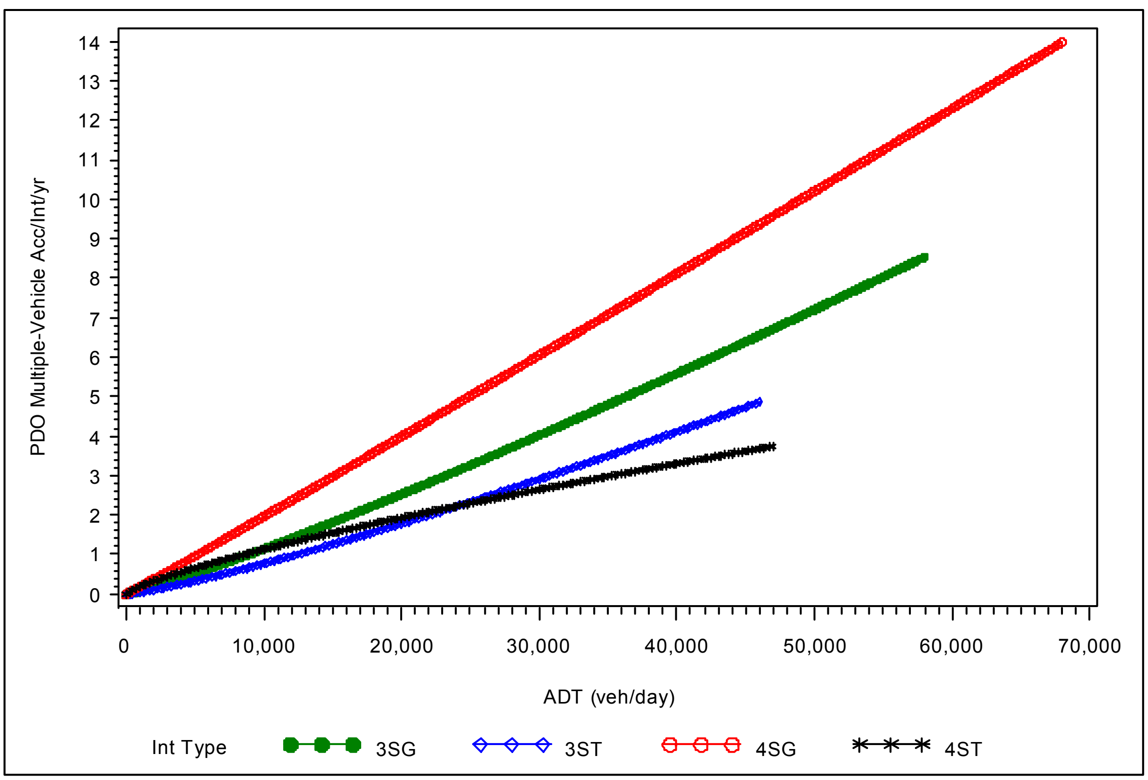

Figure 19. Plots for base models for property-damage-only multiple-vehicle collisions at intersections by intersection type.

Single-Vehicle Collisions

Negative binomial regression models for single-vehicle collisions were initially developed in the form shown below:

$$N_{bisv} = \exp\,(a + b\,\ln ADT_{maj} + c\,\ln ADT_{min}) \qquad \textbf{(34)}$$

where:

N_{bisv} = predicted number of single-vehicle collisions per year for nominal or base conditions

a,b,c = regression coefficients

No independent variables other than major- and minor-road traffic volume were statistically significant for single-vehicle collisions except for one model where intersection skew angle was statistically significant.

The model coefficients are presented in Tables 53 and 54 for Minnesota and North Carolina, respectively.

Table 55 presents comparable models to Tables 53 and 54 for the Minnesota and North Carolina data combined. These models are in the same form as Equation (34), but were developed with a random state effect as explained in Chapter 3 of this report. Table 55 includes the magnitude of the variance for the state effect. These models are illustrated in Figures 20 through 22. The plots for all intersection types are based on an assumed value of 3,000 veh/day for minor-road traffic volume.

Statistically significant models for single-vehicle collisions were found for all but one case. Where no formal model is available, single-vehicle collisions can be estimated by a multiplicative factor as a proportion of multiple-vehicle collisions, as follows:

$$N_{bisv} = N_{bimv}\, f_{isv} \qquad \textbf{(35)}$$

where:

f_{isv} = factor for single-vehicle collisions at intersections as a proportion of multiple-vehicle collisions

TABLE 53. Initial models for single-vehicle collisions at Minnesota intersections

Intersection type	No. of sites	Regression coefficient (standard error): Intercept (a)	Regression coefficient (standard error): ADT_{maj} (b)	Regression coefficient (standard error): ADT_{min} (c)	Over-dispersion parameter (k)	R^2_{LR}	Notes
Total accidents							
3ST	40						
3SG	36	–12.75 (4.00)	0.74 (0.37)	0.47 (0.19)	0.18	0.27	
4ST	48	–16.29 (4.03)	1.07 (0.34)	0.62 (0.26)	0.31	0.26	[a]
4SG	67	–8.34 (2.59)	0.46 (0.26)	0.35 (0.11)	0.30	0.22	
Fatal-and-injury accidents							
3ST	40						
3SG	36						
4ST	48	–18.26 (5.66)	1.18 (0.48)	0.60 (0.37)	0.15	0.21	[a]
4SG	67	–13.42 (4.00)	0.85 (0.41)	0.32 (0.18)	0.09	0.15	
Property-damage-only accidents							
3ST	40						
3SG	36	–12.51 (5.14)	0.89 (0.47)	0.21 (0.24)	0.42	0.12	0.36[b]
4ST	48	–14.94 (4.41)	0.92 (0.37)	0.56 (0.28)	0.14	0.16	
4SG	67	–7.07 (2.78)	0.31 (0.29)	0.33 (0.12)	0.28	0.17	0.27[c]

[a] Initial model contained a statistically significant effect of intersection skew angle; this base model incorporates the coefficient for a right-angle intersection.
[b] Minor-road ADT significance level if greater than 0.20.
[c] Major-road ADT significance level if greater than 0.20.
NOTE: All models are in the form shown in Equation (34).

TABLE 54. Initial models for single-vehicle collisions at North Carolina intersections

Intersection type	No. of sites	Regression coefficient (standard error) Intercept (a)	ADT$_{maj}$ (b)	ADT$_{min}$ (c)	Over-dispersion parameter (k)	R^2_{LR}	Notes
Total accidents							
3ST	47	–6.11 (2.04)	0.21 (0.21)	0.40 (0.24)	1.11	0.12	0.31[a]
3SG	42	–6.89 (3.90)	0.28 (0.36)	0.40 (0.22)	0.53	0.09	0.43[a]
4ST	48						
4SG	44	–11.33 (2.07)	1.03 (0.21)	0.05 (0.12)	0.11	0.39	0.64[b]
Fatal-and-injury accidents							
3ST	47						
3SG	42	–8.93 (6.12)	0.45 (0.58)	0.26 (0.33)	0.64	0.03	0.44[a], 0.43[b]
4ST	48						
4SG	44	–6.65 (3.26)	0.30 (0.35)	0.18 (0.22)	0.03	0.05	0.38[a], 0.41[b]
Property-damage-only accidents							
3ST	47	–8.39 (2.33)	0.33 (0.22)	0.50 (0.23)	0.93	0.17	
3SG	42	–7.12 (4.16)	0.22 (0.37)	0.47 (0.25)	0.57	0.09	0.56[a]
4ST	48	–5.43 (2.16)	0.25 (0.22)	0.19 (0.17)	0.49	0.07	0.26[a], 0.28[b]
4SG	44	–14.29 (2.66)	1.32 (0.27)	0.03 (0.14)	0.21	0.39	0.85[b]

[a] Major-road ADT significance level if greater than 0.20.
[b] Minor-road ADT significance level if greater than 0.20.

NOTE: All models are in the form shown in Equation (34); all intersections are located in Charlotte, NC.

TABLE 55. Base models for single-vehicle collisions at intersections in Minnesota and North Carolina combined

Intersection types	Number of sites	Regression coefficient (standard error): Intercept (a)	Regression coefficient (standard error): ADT_{maj} (b)	Regression coefficient (standard error): ADT_{min} (c)	State variance	Over-dispersion parameter (k)	R_{LR}^2	Notes
Total accidents								
3ST	97	–6.84 (2.07)	0.16 (0.19)	0.51 (0.18)	0.30	1.14	0.16	0.40[a]
3SG	78	–8.52 (2.59)	0.42 (0.24)	0.40 (0.15)	0.20	0.36	0.15	
4ST	96	–5.40 (1.82)	0.33 (0.17)	0.12 (0.14)	0.02	0.65		0.41[b]
4SG	111	–9.85 (1.85)	0.68 (0.19)	0.27 (0.08)	< 0.01	0.36		
Fatal-and-injury accidents								
3ST	97							
3SG	78	–9.25 (3.81)	0.27 (0.36)	0.51 (0.22)	< 0.01	0.24		0.46[a]
4ST	96							
4SG	111	–8.89 (2.62)	0.43 (0.27)	0.29 (0.13)	< 0.01	0.09		
Property-damage-only accidents								
3ST	97	–8.39 (2.45)	0.25 (0.22)	0.55 (0.20)	0.30	1.29		0.27[a]
3SG	78	–8.58 (3.10)	0.45 (0.29)	0.33 (0.17)	0.32	0.53	0.09	
4ST	96	–7.11 (2.17)	0.36 (0.20)	0.25 (0.15)	0.03	0.54		
4SG	111	–10.98 (2.11)	0.78 (0.22)	0.25 (0.10)	< 0.01	0.44		

[a] Major-road ADT significance level if greater than 0.20.
[b] Minor-road ADT significance level if greater than 0.20.
NOTE: All models are in the form shown in Equation (34).

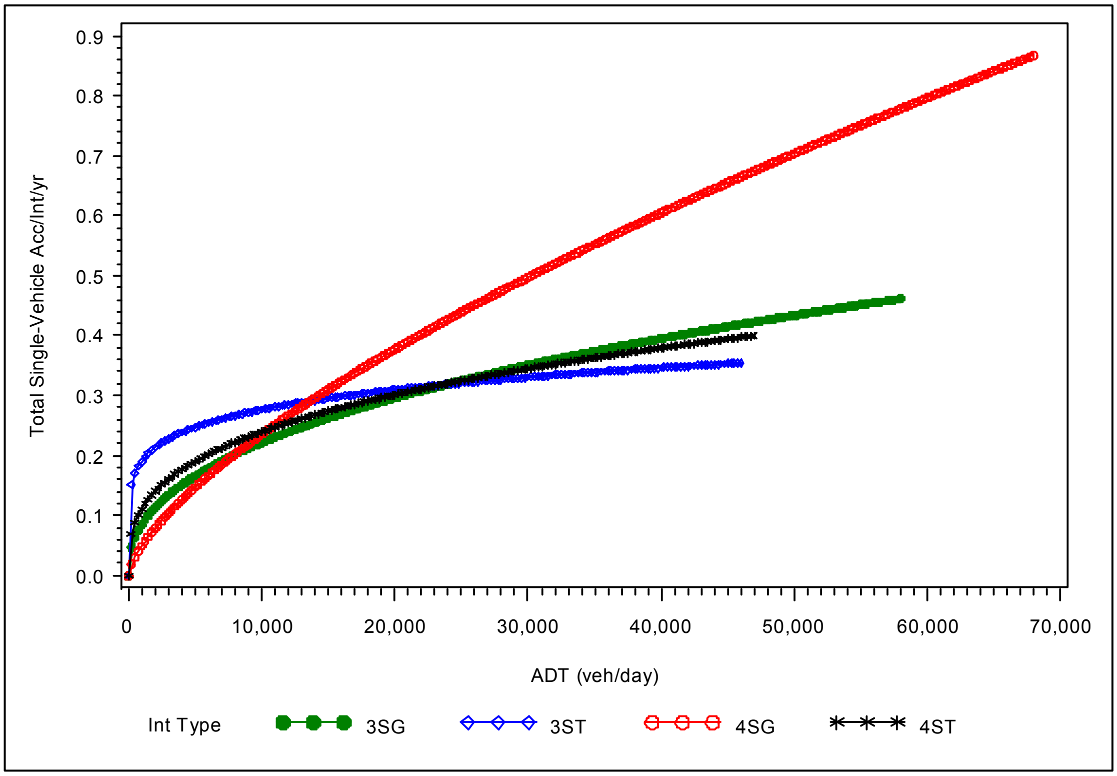

Figure 20. Plots for base models for total single-vehicle collisions at intersections by intersection type.

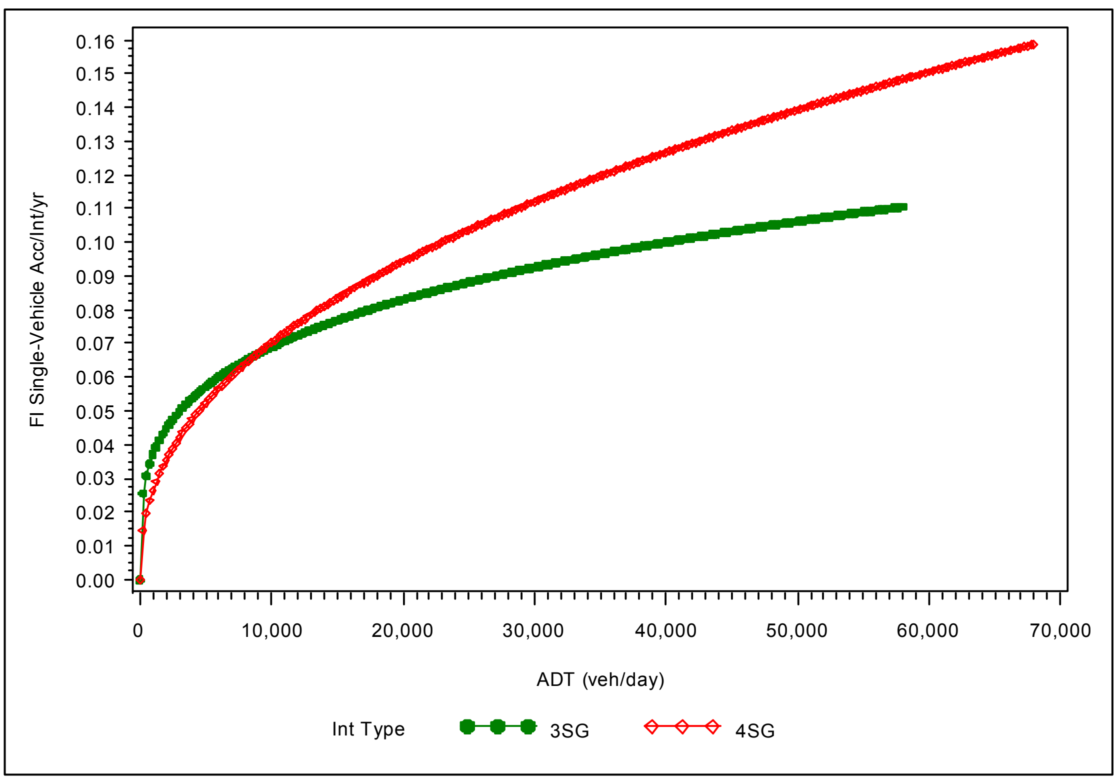

Figure 21. Plots for base models for fatal-and-injury single-vehicle collisions at intersections by intersection type.

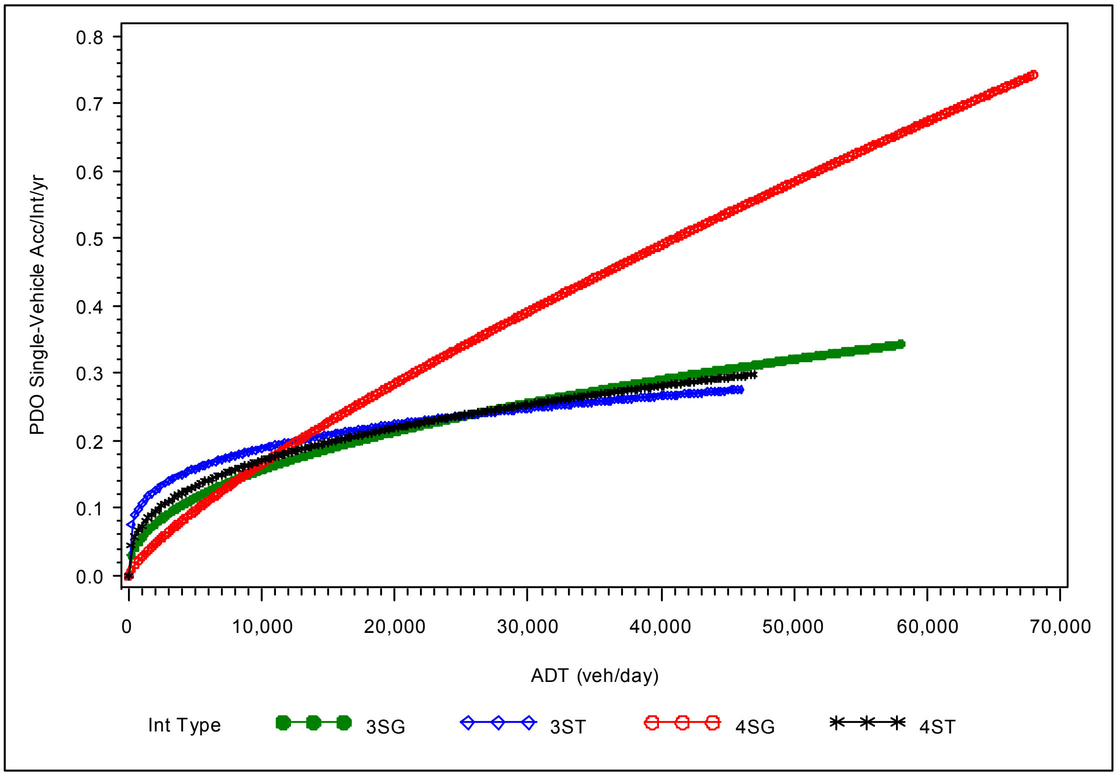

Figure 22. Plots for base models for property-damage-only single-vehicle collisions at intersections by intersection type.

Table 56 presents values of f_{isv} that can be applied where no model for single-vehicle collisions is available. The values of f_{isv} in Table 56 are smaller than the values of f_{rsv} in Table 41, so the use of tabulated factors appears even more appropriate for intersections than for roadway segments. This approach is definitely needed for fatal and injury accidents at three- and four-leg stop-controlled intersections, because no models were obtained for these cases in Table 55.

TABLE 56. Single-vehicle collision factors for intersections

Intersection type	Single-vehicle accident factor (f_{isv})		
	Minnesota	North Carolina	Combined
Total accidents			
3ST	0.12	0.12	0.12
3SG	0.07	0.05	0.06
4ST	0.14	0.08	0.11
4SG	0.09	0.04	0.07
Fatal-and-injury accidents			
3ST	0.13	0.10	0.12
3SG	0.07	0.04	0.06
4ST	0.16	0.07	0.12
4SG	0.07	0.03	0.05
Property-damage-only accidents			
3ST	0.12	0.13	0.13
3SG	0.08	0.06	0.07
4ST	0.12	0.09	0.11
4SG	0.10	0.04	0.07

The severity distribution for single-vehicle collisions at intersections is presented in Table 57.

TABLE 57. Severity distribution of single-vehicle collisions for intersections

Intersection type	No. of sites	Percentage of total accidents: Fatal and injury	Percentage of total accidents: Property damage only
Minnesota sites			
3ST	40	36.8	63.2
3SG	36	39.1	60.9
4ST	48	42.0	58.0
4SG	67	24.8	75.2
Michigan sites			
3ST	47	30.4	69.6
3SG	42	22.7	77.3
4ST	48	36.5	63.5
4SG	44	24.1	75.9
Combined sites			
3ST	87	33.6	66.4
3SG	78	30.9	69.1
4ST	96	39.2	60.8
4SG	111	24.4	75.6

Vehicle-Pedestrian Collisions

Table 58 presents adjustment factors for the average frequency of vehicle-pedestrian collisions at arterial intersections. The table includes values for Minnesota, North Carolina, and both data sets combined. The accident counts reflect the average frequency of vehicle-pedestrian collisions at intersections. The factors in Table 58 incorporate vehicle-pedestrian collisions and any bicycle-pedestrian collisions that occur. Both the Minnesota and North Carolina data represent a mix of urban and suburban conditions. It is recommended that the combined column shown in Table 58 be used in the initial HSM methodology for both urban and suburban conditions.

Table 58 will eventually be replaced with a pedestrian safety prediction methodology which is just beginning development in a newly-added portion of NCHRP Project 17-26.

TABLE 58. Pedestrian safety adjustment factors for intersections

Intersection type	Pedestrian safety adjustment factor (f_{pedi}): Minnesota	North Carolina	Combined
3ST	0.006	0.009	0.008
3SG	0.003	0.006	0.005
4ST	0.027	0.005	0.016
4SG	0.023	0.011	0.017

NOTE: These factors apply to the methodology for predicting total accidents (all severity levels combined). All vehicle-pedestrian collisions resulting from this adjustment factor should be treated as fatal-and-injury accidents and none as property-damage-only accidents.

Vehicle-Bicycle Collisions

Table 59 presents adjustment factors for the average frequency of vehicle-bicycle collisions at arterial intersections. The table includes values for Minnesota, North Carolina, and both data sets combined. The factor shown in Table 59 is multiplied by the frequency of all other accident types to estimate the average frequency of vehicle-bicycle collisions.

The values of f_{pedi} and f_{bikei} have been determined consistently so that, together, they account for the average frequency of vehicle-pedestrian and vehicle-bicycle collisions on arterial intersections of various types. Both the Minnesota and North Carolina data represent a mix of urban and suburban conditions. It is recommended that the combined column shown in Table 59 be used in the HSM methodology for both urban and suburban conditions.

Table 59 is intended for use in HSM methodology, as there is no current plan for a bicycle safety prediction methodology analogous to the pedestrian safety prediction methodology currently under development.

TABLE 59. Bicycle safety adjustment factor for intersections

Intersection type	Bicycle safety adjustment factor (f_{bikei})		
	Minnesota	North Carolina	Combined
3ST	0.006	0.001	0.004
3SG	0.015	0.004	0.010
4ST	0.010	0.002	0.006
4SG	0.021	0.004	0.013

NOTE: These factors apply to the methodology for predicting total accidents (all severity levels combined). All vehicle-bicycle collisions resulting from this adjustment factor should be treated as fatal-and-injury accidents and none as property-damage-only accidents.

CHAPTER 6.

ACCIDENT MODIFICATION FACTORS

This chapter presents the accident modification factors (AMFs) developed for use in the HSM methodology for urban and suburban arterials and explains the derivation of these AMFs.

The AMFs for use in the HSM methodology were chosen by an expert panel convened jointly by the research team for the current project and NCHRP Project 17-25, *Crash Reduction Factors for Traffic Engineering and ITS Improvements*. The expert panel included members of both research teams and other invited researchers and practitioners. The members of the expert panel included:

- James Bonneson, Texas Transportation Institute
- Forest M. Council, BMI-SG/VHB
- Kim Eccles, BMI-SG/VHB
- David Harkey, Highway Safety Research Center, University of North Carolina
- Douglas W. Harwood, Midwest Research Institute
- Ezra Hauer, Consultant
- Bhagwant Persaud, Ryerson University
- Stan Polanis, City of Winston-Salem, North Carolina
- Ragahavan Srinivasan, Highway Safety Research, University of North Carolina
- Thomas Welch, Iowa Department of Transportation

The expert panel assessed the literature for a broad range of design and traffic control elements for urban and suburban arterials and made recommendations concerning:

- which design and traffic control elements have safety effects that are sufficiently well understood to be used as AMFs in the HSM methodology
- for those design and traffic control elements whose safety effects are well understood, which sources in the literature best document those safety effects, and form the basis for a reliable AMF

The process used by the expert panel to assess AMFs was as follows:

- a comprehensive search and review of literature related to potential AMFs was conducted.
- a notebook containing extracts from all relevant literature was assembled and was reviewed by individual expert panel members.
- the expert panel was convened and each AMF and each study related to that AMF were reviewed and discussed

- Based on the review and discussion at the meeting, the expert panel reached consensus and selected one or more studies as providing the most credible AMF, which was recommended for potential use in the HSM. Criteria considered by the expert panel in assessing studies as potential sources of AMFs included quality of documentation; sample size; study design, including compensation for regression to the mean; and standard error or goodness of fit.

In some cases, further analyses were conducted by the research team to further develop the expert panel's selection of AMFs.

The recommended AMFs for roadway segments and intersections are presented below.

ROADWAY SEGMENTS

Recommended AMFs for roadway segments presented below include:

- on-street parking
- roadside fixed objects
- lighting

It was originally intended to include an AMF for median width on arterial roadway segments based on the results of Kniuman et al. (*128*) and Hadi (*13*). However, on further consideration of this issue, the research team decided that a satisfactory AMF could not be developed for median width on roadway segments.

The AMFs for roadway segments are presented below.

On-Street Parking (AMF_{1r})

An AMF for on-street parking was developed from recent work by Bonneson (*129*) for TexDOT. Bonneson formulated an AMF for on-street parking as follows:

$$AMF_{1r} = 1 + p_{pk}\ (f_{pk} - 1) \tag{36}$$

$$f_{pk} = (1.10 + 0.365\ I_{cs} + 0.609\ p_{b/0})\ [(f_{ap/pp} - 1.0)\ p_{ap} + 1.0] \tag{37}$$

where:

AMF_{1r}	=	accident modification factor for on-street parking
p_{pk}	=	proportion of curb length with on-street parking (= 0.5 L_{pk}/L)
L_{pk}	=	curb length with on-street parking (mi)
L	=	roadway segment length (mi)
I_{cs}	=	indicator variable for cross-section (= 1 for two-lane street; 0 otherwise)

$P_{b/0}$ = for that part of the street with parking, the proportion that has business or office as adjacent land use
$f_{ap/pp}$ = ratio of crashes on streets with angle parking to crashes on streets with parallel parking
P_{ap} = for that part of the street with parking, the proportion with angle parking

Bonneson has derived the value of 2.34 for $f_{ap/pp}$.

Based on Equation (37), Table 60 presents computed values for f_{pk} for various types of sites. The values of f_{pk} can be used in Equation (36) to determine an AMF value for on-street parking. The value of p_{pk} in Equation (36) represents the proportion of curb length with on-street parking; since on-street parking may be permitted on one or both sides of the street, the maximum length of curb parking (L_{pk}) used in determining P_{pk} for any site is twice the roadway segment length (L). Table 60 shows the AMF value that would apply if on-street parking were permitted on both sides of the street throughout a site (i.e., P_{pk} = 1.0).

Roadside Fixed Objects (AMF_{2r})

The expert panel decided that the best source of information on the safety effects of roadside obstacles was the model for prediction of utility pole accidents developed by Zegeer and Cynecki (*130*). The Zegeer and Cynecki model for utility pole accidents is:

$$N_{fo} = \frac{9.84 \times 10^{-5}\ ADT + 0.0354\ D_{fo}}{O_{fo}^{0.6}} - 0.04 \qquad (38)$$

where:

N_{fo} = predicted number of fixed object accidents per mi per year
ADT = average daily traffic volume (veh/day)
D_{fo} = fixed object density (objects/mi)
O_{fo} = fixed object offset (distance from edge of traveled way to objects) (ft)

The expert panel decided that the safety effect of utility poles in the Zegeer and Cynecki model could be used, as well, to represent the safety effect of other roadside fixed objects with the same density and offset.

The dashed lines in Figure 23 illustrate the relationships between fixed objects accidents and fixed object density represented by the Zegeer and Cynecki model. The figure shows one concern with the Zegeer and Cynecki model that is potentially inconsistent with theory. The dashed lines in the figure show a non-zero y-intercept, such that a non-zero number of fixed object accidents is predicted even when the fixed object density is zero. A further consideration is that there should be a theoretical upper limit on fixed object density at which the fixed objects are so close together that they no longer have independent effects on safety. In other words, if two fixed objects are very close together, they might be better counted as one fixed object rather than two.

TABLE 60. Determination of the accident modification factor for on-street parking for specific types of roadway segments

Roadway cross section	Land use	Type of parking	I_{cs}	$P_{b/0}$	$f_{ap/pp}$	P_{ap}	f_{pk} [a]	P_{pk}	AMF_{1r} [b]
Multilane	Residential	Parallel	0	0.0	2.34	0.0	1.100	1.0	1.00
	Commercial/industrial	Parallel	0	1.0	2.34	0.0	1.709	1.0	1.709
Two-lane	Residential	Parallel	1	0.0	2.34	0.0	1.465	1.0	1.465
	Commercial/industrial	Parallel	1	1.0	2.34	0.0	2.074	1.0	2.074
Multilane	Residential	Angle	0	0.0	2.34	1.0	2.574	1.0	2.574
	Commercial/industrial	Angle	0	1.0	2.34	1.0	3.999	1.0	3.999
Two-lane	Residential	Angle	1	0.0	2.34	1.0	3.428	1.0	3.428
	Commercial/industrial	Angle	1	1.0	2.34	1.0	4.853	1.0	4.853

[a] Determined with Equation (37).
[b] Determined with Equation (36).

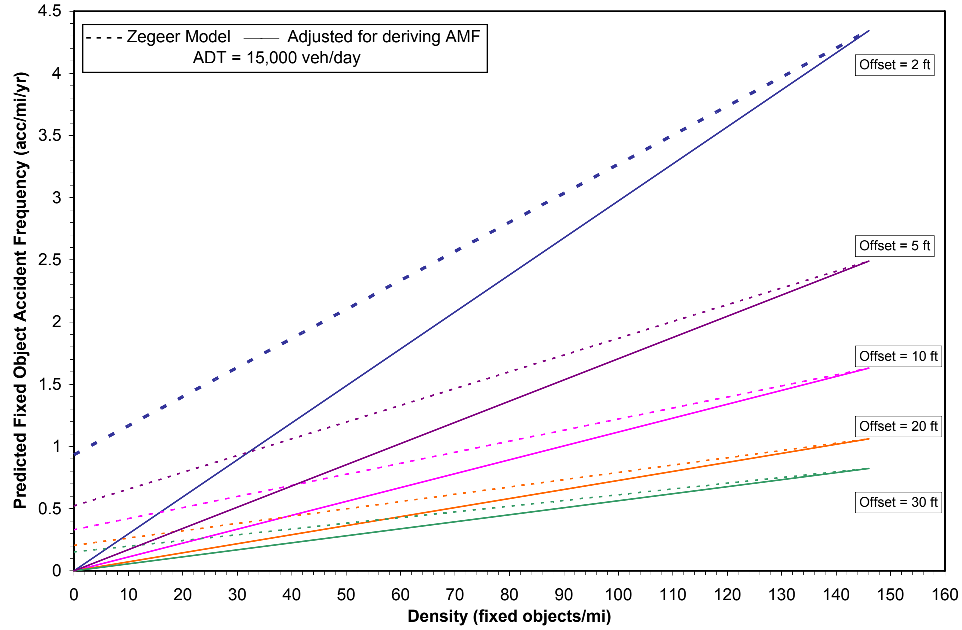

Figure 23. Relationships of fixed-object collisions to fixed-object density and offset [adapted from Zegeer and Cynecki (130)].

The maximum utility pole density for any site in the Zegeer and Cynecki study is 90.7 objects/km (146 objects/mi) for both sides of the road combined, which corresponds to an average spacing between objects of 22 m (72 ft); it would not seem appropriate to assume any closer object spacing in applying the Zegeer and Cynecki results. The points at the right end of each line in Figure 23 represents this maximum value of fixed object density. The dashed lines in Figure 23 are based on the Zegeer and Cynecki model. The dashed lines have nonzero values for the y-intercept. It would be more consistent with theory for the lines in the figure to pass through the origin than through a nonzero intercept. The solid lines in Figure 23 show this more consistent relationship which has been used in developing the AMF.

An AMF for fixed object density and offset can be represented as the ratio of the predicted fixed object accidents for the actual fixed object density and offset on a roadway segment to the predicted fixed object collisions for a typical average fixed object density for urban roads (24.8 fixed objects/km or 40 fixed objects/mi) and a typical average offset for fixed objects on urban roads (1.5 m or 5 ft). Ignoring the 0.04 term in Equation (38), which makes a minimal contribution to the estimate, this AMF can be represented by:

$$AMF_{2r} = \frac{\left(\left(9.84 \times 10^{-5}\ ADT + 0.0354\ (105.6) / O_{fo}^{0.6}\right)\left(\frac{D_{fo}}{105.6}\right)\right.}{\left(\left(9.84 \times 10^{-5}\ ADT + 0.0354\ (105.6) / 5^{0.6}\right)\left(\frac{40}{105.6}\right)\right.} \qquad \mathbf{(39)}$$

The ADT term in Equation (39) has very little influence on the value of AMF_{2r}. All of Equation (34) except the D_{fo} term can be reduced to a factor (f_{offset}) that is a factor of fixed object offset alone (see Table 61). The AMF as shown in Equation (39) applies to fixed-object accidents, which constitute the proportion of total accidents (excluding vehicle-pedestrian and vehicle-bicycle collisions) shown in Table 62. Thus, the AMF for roadside fixed objects for application in the HSM methodology can be represented as:

$$AMF_{2r} = f_{offset}\, D_{fo}\, p_{fo} + (1 - p_{fo}) \qquad \mathbf{(40)}$$

where:

f_{offset} = fixed-object offset factor from Table 61
p_{fo} = fixed-object collisions as a proportion of total accidents from Table 62

TABLE 61. Fixed-object offset factor (f_{offset})

Offset to fixed objects (O_{fo}) (ft)	Fixed-object offset factor (f_{offset})
2	0.232
5	0.133
10	0.087
15	0.068
20	0.057
25	0.049
30	0.044

TABLE 62. Proportion of fixed-object collisions (p_{fo})

Road type	Proportion of fixed-object collisions (p_{fo})
2U	0.059
3T	0.034
4U	0.037
4D	0.036
5T	0.016

In estimating the density of fixed objects, point objects that are within 22 m (72 ft) of one another longitudinally along the road should be counted as a single object. Continuous objects that are not behind point objects should be counted as one point object for each 22 m (72 ft) of length. Only point or continuous objects within the offset distance used to derive the value of f_{offset} should be used in determining D_{fo} and only objects of 100 mm (4 in) or more in diameter and are not of breakaway design should be counted.

Lighting (AMF_{3r})

The AMF for lighting is derived from the work of Elvik and Vaa (*131*) who combined the results of 38 international studies through meta analysis and found the following accident reductions for lighting of roadway segments that had not previously been lighted:

Accident severity level	Best estimate of nighttime accident reduction
Fatal	64%
Injury	28%
Property-damage-only	17%

Table 63 illustrates the derivation of an AMF for total accidents based on these estimates using crash distribution data for Minnesota and Michigan combined. The AMFs for the individual road types are so similar that a single combined AMF of 0.96 for all road types seems appropriate. In other words, installation of lighting appears to reduce roadway segment accidents by approximately 4 percent.

TABLE 63. Example of computing the AMF for lighting of roadway segments

Road type	AMF by accident severity level[a]			Proportion of total nighttime accidents[b]			AMF for nighttime accidents[c]	Proportion of accidents that occur at night[b]	AMF for total accidents (AMF_{3r})
	Fatal	Injury	PDO	Fatal	Injury	PDO			
2U	0.360	0.720	0.830	0.008	0.240	0.750	0.798	0.266	0.946
3T	0.360	0.720	0.830	0.030	0.300	0.670	0.783	0.279	0.939
4U	0.360	0.720	0.830	0.001	0.290	0.709	0.797	0.230	0.953
4D	0.360	0.720	0.830	0.001	0.290	0.709	0.797	0.204	0.959
5T	0.360	0.720	0.830	0.001	0.273	0.726	0.800	0.391	0.922

[a] Based on Elvik and Vaa (*131*).
[b] Based on combined data for unlighted roadway segments in Minnesota and Michigan.
[c] Weighted average of Elvik and Vaa estimates for all severity levels combined.

Because the crash distribution estimates shown in Table 63 may vary among jurisdictions, it is recommended that, whenever possible, the lighting AMF should be computed explicitly for each jurisdiction that applies the HSM. The lighting AMF can be computed as:

$$AMF_{3r} = 1 - \left[(1 - 0.36\,p_{fnr} - 0.72\,p_{inr} - 0.83\,p_{nr})\,p_{nr}\right] \qquad \textbf{(41)}$$

where:

p_{fnr} = proportion of total nighttime accidents for unlighted roadway segments that involve a fatality

p_{inr} = proportion of total nighttime accidents for unlighted roadway segments that involve only a nonfatal injury

p_{pnr} = proportion of total nighttime accidents for unlighted roadway segments that involve property damage only

p_{nr} = proportion of total accidents for unlighted roadway segments that occur at night

When estimates of p_{fnr}, p_{inr}, p_{pnr}, and p_{nr} are not available for a jurisdiction, then the values presented in Table 63 should be used as defaults.

INTERSECTIONS

Recommended AMFs for intersections presented below include:

- left-turn lanes
- left-turn signal phasing
- right-turn lanes
- right turn on red
- lighting

The expert panel recommended the inclusion of all of these AMFs. The inclusion of an AMF for red-light cameras was designated as optional. The research team decided to include this AMF based on comments from the TRB Task Force.

The AMFs for intersections are presented below.

Left-Turn Lanes (AMF_{1i})

AMFs for left-turn lanes at intersections are based on the work of Harwood et al. (*81*). The values of the recommended AMFs have been presented earlier in Table 4. These AMFs apply to left-turn lanes installed on major-road intersection approaches. The AMF for left-turn lanes does not include consideration of left-turn lane length. The safety effect of left-turn lane lengths has not been documented and, in any case, is probably dependent on hourly or shorter-term traffic volumes which are not likely to be available, in most cases.

Left-Turn Signal Phasing (AMF_{2i})

The expert panel decided to adopt the AMFs derived by Hauer (*132*) with adjustments to the AMFs for the converting permissive to protected/permissive phasing and converting protected/permissive to protected phasing based on the findings of Lyon et al. (*133*). Table 64 presents the recommended AMFs. These AMFs are applicable only to signalized intersections. It should be noted that the research indicated no difference in safety between protected/permissive or permissive/protected operation.

Right-Turn Lanes (AMF_{3i})

AMFs for right-turn lanes are based on the work of Harwood et al. (*81*). The values recommended have been presented earlier in Table 7. These AMFs apply to right-turn lanes installed on major-road intersection approaches.

TABLE 64. AMFs for left-turn signal phasing

Original left-turn phasing	Modified left-turn phasing	Accident type	AMF for specific accident type	Proportion of specified accident type	AMF for total accidents (AMF_{2i})
Permissive	Protected	Left turn Other	0.30[a] 1.00[b]	0.09 0.91	0.937
Permissive	Protective/ permissive or permissive/ protected	Left turn Other	0.84 1.00[b]	0.09 0.91	0.986
Protected/ permissive or permissive/ protected	Protected	Left turn Other	0.36 1.00[b]	0.09 0.91	0.942
Lagging	Leading	Left turn	1.00	0.09	1.000

[a] Most likely depends on number of opposing lanes.
[b] Insufficient and contradictory evidence.

Right Turn on Red (AMF_{4i})

The AMF for prohibiting right-turn-on-red (RTOR) operation at signals is based on the work of Clark (*134*). The Clark study was reviewed by Bahar et al. (*135*) in the interim report of NCHRP Project 17-27. Bahar et al. noted that Clark's results can be expressed as AMFs of 1.13 for South Carolina data and 1.05 for Alabama data for permitting RTOR, with a combined AMF of 1.067. If RTOR is permitted on a single approach to a four-leg signalized intersection, the AMF would be $1.067^{0.25}$, or 1.016. This AMF applies to total intersection accidents.

In the HSM methodology, it is more useful to present an AMF for prohibiting RTOR, since permitting RTOR is the default condition under existing state laws. The AMF for prohibiting RTOR on a single approach to a signalized intersection would be 1/1.064 = 0.984. Since RTOR may be prohibited for several approaches to a given intersection, the AMF can be expressed as:

$$AMF_{4i} = (0.984)^{nprohib} \qquad (42)$$

where:

n^{prohib} = number of signalized intersection approaches for which RTOR is prohibited

Since AMF_{4i} is applied in the HSM methodology to intersection accidents excluding vehicle-pedestrian and vehicle-bicycle collisions, the expert panel recommended that the AMF in Equation (42) be reviewed to determine whether part of the safety benefit included was a reduction in vehicle-pedestrian collisions that should be considered not in this AMF, but in the pedestrian safety prediction methodology currently under development. It was found that a portion of the safety benefit represented by Equation (42) was due to reduction of collisions between right-turning vehicles and pedestrians, but that the proportion was so small that the value 0.984 did not change (i.e., the effect was in the fourth significant digit). Therefore, Equation (42) was not changed.

Lighting (AMF_{5i})

The AMF for intersection lighting is derived from the work of Elvik and Vaa (*131*) for roadway segments, discussed above. The expert panel decided that the effectiveness measures for roadway segments were the best available information to apply to intersections. Table 65 illustrates the derivation of the AMF for intersection lighting using crash distribution data for Minnesota and North Carolina combined.

Because the crash distribution estimates shown in Table 65 may vary among jurisdictions, it is recommended that, whenever possible, the lighting AMF should be computed explicitly for each jurisdiction that applies the HSM. The lighting AMF can be computed as:

$$AMF_{5i} = 1 - [(1 - 0.36\,p_{fni} - 0.72\ p_{ini} - 0.83\ p_{pni})\ p_{ni}] \quad \textbf{(43)}$$

where:

p_{fni} = proportion of total nighttime accidents for unlighted intersections that involve a fatality

p_{ini} = proportion of total nighttime accidents for unlighted intersections that involve only a nonfatal injury

p_{pni} = proportion of total nighttime accidents for unlighted intersections that involve property damage only

p_{ni} = proportion of total accidents for unlighted intersections that occur at night

When estimates of p_{fni}, p_{ini}, p_{pni}, and p_{ni} are not available for a jurisdiction, then the values presented in Table 65 should be used as defaults.

TABLE 65. Example of computing the AMF for lighting of intersections

Intersection type	AMF by accident severity level[a]			Proportion of total nighttime accidents[b]			AMF for nighttime accidents[c]	Proportion of accidents that occur at night[b]	AMF for total accidents (AMF_{5i})
	Fatal	Injury	PDO	Fatal	Injury	PDO			
3ST	0.360	0.720	0.830	0.001	0.334	0.665	0.793	0.192	0.960
3SG	0.360	0.720	0.830	0.001	0.393	0.606	0.786	0.206	0.956
4ST	0.360	0.720	0.830	0.040	0.360	0.600	0.772	0.191	0.956
4SG	0.360	0.720	0.830	0.003	0.328	0.670	0.793	0.200	0.959

[a] Based on Elvik and Vaa (*131*) findings for roadway segments.
[b] Based on combined data for unlighted intersections in Minnesota and North Carolina.
[c] Weighted average of Elvik and Vaa estimates for all severity levels combined.

CHAPTER 7.

SUMMARY OF HSM METHODOLOGY

This chapter summarizes the HSM methodology for urban and suburban arterials by putting together the building blocks of the methodology whose development was documented in Chapters 5 and 6. The methodology presented here is summarized in the draft version of HSM Chapter 10 presented in Appendix B of this report.

OVERALL STRUCTURE OF METHODOLOGY

The overall structure of the methodology is to make safety predictions for individual arterial roadway segments and intersections and to combine these predictions in the following manner across all roadway segments and intersections that make up an arterial road section or project of interest:

$$N_t = \sum_{\text{all segments}} N_{rs} + \sum_{\text{all intersections}} N_{int} \quad (44)$$

where:

N_t = predicted accident frequency for an entire project or an extended highway section;
N_{rs} = predicted number of total roadway segment accidents per year;
N_{int} = predicted number of total intersection-related accidents per year

DEFINITION OF ROADWAY SEGMENTS AND INTERSECTIONS

Roadway segments and intersections are defined in a manner similar to the prototype chapter for two-lane highways.

Roadway segment models are used to predict all accidents that occur on portions of roadway segments that are more than 76 m (250 ft) from an intersection and nonintersection-related accidents that occur on portions of roadway segments that are within 76 m (250 ft) of an intersection. Each roadway segment should be generally homogeneous with respect to:

- Average daily traffic volume (veh/day)
- Number of through lanes
- Presence/type of median
- Presence/type of on-street parking
- Roadside fixed object density

Also, a new roadway segment should start at any of the following locations:

- Intersection (use center of intersection as segment boundary)
- Beginning or end of a horizontal curve

Intersection models are used to predict all intersection-related accidents that occur within 76 m (250 ft) of a particular intersection.

ROADWAY SEGMENT METHODOLOGY

General Form of Roadway Segment Methodology

Safety predictions for a particular roadway segment will be developed as a combination of base models, calibration factors, and accident modification factors (AMFs) using the following general approach:

$$N_{rs} = (N_{br} + N_{pedr} + N_{biker})\ C_r \qquad \textbf{(45)}$$

$$N_{br} = N_{brbase}\ (AMF_{1r}\ AMF_{2r}\ AMF_{3r}) \qquad \textbf{(46)}$$

where:

N_{rs}	=	predicted number of total roadway segment accidents per year
N_{br}	=	predicted number of roadway segment accidents per year excluding vehicle-pedestrian and vehicle-bicycle collisions
N_{brbase}	=	predicted number of total roadway segment accidents per year for nominal or base conditions excluding vehicle-pedestrian and vehicle-bicycle collisions
N_{pedr}	=	predicted number of vehicle-pedestrian collisions per year
N_{biker}	=	predicted number of vehicle-bicycle collisions per year
AMF_{1r} … AMF_{3r}	=	accident modification factors for roadway segments
C_r	=	calibration factor for roadway segments developed for use for a particular geographical area [see Equation (59)]

Types of Roadway Segments Considered

The safety prediction methodology addresses five types of roadway segments:

- Two-lane undivided arterials (2U)
- Three-lane arterials including a center TWLTL (3T)
- Four-lane undivided arterials (4U)
- Four-lane divided arterials (i.e., including a raised or depressed median) (4D)
- Five-lane arterials including a center TWLTL (5T)

The procedures for four-lane arterials may also be applied to six-lane arterials. A review of literature by an expert panel for NCHRP Projects 17-25 and 17-26 did not find any indication of safety differences between four- and six-lane arterials.

Base Models and Adjustment Factors for Roadway Segments (N_{brmv}, N_{brsv}, N_{brdwy}, N_{pedr}, and N_{biker})

Base models are used for roadway segments to make separate predictions for multiple-vehicle nondriveway collisions, single-vehicle collision and noncollision accidents, and driveway-related collisions. Tabulated factors are used instead of base models to account for pedestrian and bicycle collisions. The tabulated pedestrian factors will eventually be replaced with a pedestrian safety prediction methodology that is under development in a recently added portion of NCHRP Project 17-26.

Equation (45) shows that total roadway segment accidents are determined as the sum of three components: N_{br}, N_{pedr}, and N_{biker}. The following equation shows that the base model portion of N_{br}, designated as N_{brbase}, is further broken down into three components:

$$N_{brbase} = N_{brmv} + N_{brsv} + N_{brdwy} \tag{47}$$

where:

N_{brmv} = predicted number of multiple-vehicle nondriveway collisions per year for nominal or base conditions
N_{brsv} = predicted number of single-vehicle collisions per year for nominal or base conditions
N_{brdwy} = predicted number of driveway-related collisions per year

Thus, base models and adjustment factors are applied to determine five components of total roadway segment accident experience corresponding to five accident types of interest: N_{brmv}, N_{brsv}, N_{brdwy}, N_{pedr}, and N_{biker}.

Base Models for Multiple-Vehicle Nondriveway Collisions on Roadway Segments (N_{brmv})

Base models for multiple-vehicle nondriveway collisions on roadway segments are applied as follows:

$$N_{brmv} = \exp\,(a + b\ \ln ADT + \ln L) \tag{48}$$

where:

ADT	=	average daily traffic volume (veh/day) on roadway segment
L	=	length of roadway segment (mi)
a,b	=	regression coefficients

Table 66 presents the values of the coefficients a and b used in applying Equation (48).

TABLE 66. Base models for multiple-vehicle nondriveway collisions on roadway segments

Road type	Coefficients used in Equation (48) Intercept (a)	Coefficients used in Equation (48) ADT (b)	Overdispersion parameter (k)
Total accidents			
2U	–14.75	1.68	0.84
3T	–11.92	1.41	0.66
4U	–11.53	1.33	1.01
4D	–11.88	1.36	1.32
5T	–9.93	1.17	0.81
Fatal-and-injury accidents			
2U	–15.75	1.66	0.65
3T	–15.97	1.69	0.59
4U	–11.98	1.25	0.99
4D	–12.30	1.28	1.31
5T	–10.70	1.12	0.62
Property-damage-only accidents			
2U	–15.15	1.69	0.87
3T	–11.47	1.33	0.59
4U	–12.43	1.38	1.08
4D	–12.35	1.38	1.34
5T	–10.20	1.17	0.88

Equation (48) is first applied to determine N_{brmv} using the coefficients for total accidents in Table 66. The values of $N'_{brmv(FI)}$ and $N'_{brmv(PDO)}$ are then determined with Equation (48) using the coefficients for fatal-and-injury and property-damage-only accidents, respectively, in Table 66. The following adjustments are made to the values of $N'_{brmv(FI)}$ and $N'_{brmv(PDO)}$ to ensure that they sum to N_{brmv}:

$$N_{brmv(FI)} = N_{brmv}\left(\frac{N'_{brmv(FI)}}{N'_{brmv(FI)} + N'_{brmv(PDO)}}\right) \qquad (49)$$

$$N_{brmv(PDO)} = N_{brmv} - N_{brmv(FI)} \qquad (50)$$

The percentages in Table 67 are used to break down $N_{brmv(FI)}$ and $N_{brmv(PDO)}$ by collision type. The distributions shown in Table 67 are based on observed collision data for all study sites in Minnesota and Michigan. The overall distributions are used in the HSM methodology and are not predicted based on the characteristics of the specific site being evaluated.

TABLE 67. Distribution of multiple-vehicle nondriveway collisions for roadway segments by collision type

Collision type	Proportion of accidents by severity level for specific road types									
	2U		3T		4U		4D		5T	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Rear-end collision	0.665	0.671	0.708	0.628	0.602	0.461	0.778	0.632	0.670	0.566
Head-on collision	0.096	0.030	0.070	0.027	0.085	0.028	0.025	0.008	0.090	0.042
Angle collision	0.106	0.107	0.140	0.153	0.185	0.167	0.093	0.104	0.160	0.173
Sideswipe, same direction	0.023	0.084	0.006	0.081	0.057	0.203	0.036	0.146	0.035	0.159
Sideswipe, opposite direction	0.062	0.057	0.018	0.022	0.021	0.031	0.005	0.016	0.019	0.025
Other multiple-vehicle collisions	0.048	0.051	0.058	0.089	0.053	0.110	0.063	0.094	0.026	0.035

Base Models for Single-Vehicle Collisions for Roadway Segments (N_{brsv})

Base models for single-vehicle collisions for roadway segments are applied as follows:

$$N_{brsv} = \exp(a + b\ \ln ADT + \ln L) \tag{51}$$

Table 68 presents the values of the coefficients and factors used in Equation (51) for each roadway type. Equation (51) is first applied to determine N_{brsv} using the coefficients for total accidents in Table 69. The values of $N'_{brsv(FI)}$ and $N'_{brsv(PDO)}$ are then determined with Equation (51) using the coefficients for fatal-and-injury and property-damage-only accidents, respectively, in Table 68. The following adjustments are made to the values of $N'_{brsv(FI)}$ and $N'_{brsv(PDO)}$ to assure that they sum to N_{brsv}:

$$N_{brsv(FI)} = N_{brsv}\left(\frac{N'_{brsv(FI)}}{N'_{brsv(FI)} + N'_{brsv(PDO)}}\right) \tag{52}$$

$$N_{brsv(PDO)} = N_{brsv} - N_{brmv(FI)} \tag{53}$$

The percentages in Table 69 are used to break down $N_{brsv(FI)}$ and $N_{brsv(PDO)}$ by accident type. The distributions shown in Table 69 are based on observed collision data for all study sites in Minnesota and Michigan. The overall distributions are used in the HSM methodology and are not predicted based on the characteristics of the specific site being evaluated.

TABLE 68. Base models for single-vehicle collisions on roadway segments

Road type	Coefficients used in Equation (51): Intercept (a)	Coefficients used in Equation (51): ADT (b)	Overdispersion parameter (k)
Total accidents			
2U	–5.00	0.56	0.81
3T	–5.26	0.54	1.37
4U	–7.89	0.81	0.91
4D	–4.59	0.47	0.86
5T	–5.05	0.54	0.52
Fatal-and-injury accidents			
2U	–3.49	0.23	0.50
3T	–5.89	0.47	1.06
4U	–7.27	0.61	0.54
4D	–8.25	0.66	0.28
5T	–4.66	0.35	0.36
Property-damage-only accidents			
2U	–6.04	0.64	0.87
3T	–5.81	0.56	1.93
4U	–8.40	0.84	0.97
4D	–4.58	0.45	1.06
5T	–6.06	0.61	0.55

TABLE 69. Distribution of single-vehicle collisions for roadway segments by collision type

Collision type	Proportion of accidents by severity level for specific road types									
	2U		3T		4U		4D		5T	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Collision with parked vehicle	0.164	0.443	0.143	0.173	0.285	0.484	0.061	0.144	0.267	0.250
Collision with animal	0.033	0.179	0.082	0.323	0.008	0.044	0.023	0.172	0.033	0.236
Collision with fixed object	0.327	0.166	0.367	0.260	0.324	0.236	0.385	0.264	0.233	0.236
Collision with other object	0.230	0.120	0.184	0.118	0.236	0.162	0.205	0.241	0.167	0.208
Other single-vehicle collision	0.000	0.013	0.000	0.008	0.033	0.041	0.076	0.041	0.067	0.042
Noncollision	0.246	0.079	0.224	0.118	0.114	0.033	0.250	0.138	0.233	0.028

Base Models for Multiple-Vehicle Driveway-Related Collisions (N_{brdwy})

The models presented above for multiple-vehicle collisions addressed only collisions that are not related to driveways. Driveway-related accidents are also generally multiple-vehicle collisions, but are addressed separately because the frequency of driveway-related accidents on a roadway segment depends on the number and type of driveways.

The total number of driveway-related collisions within a roadway segment is determined as:

$$N_{brdwy} = \sum_{\text{all driveway types}} n_j \, N_j \left(\frac{ADT}{15{,}000} \right)^{b} \tag{54}$$

where:

N_j = number of accidents per year for an individual driveway of driveway type j from Table 70.
n_j = number of driveways within roadway segment of driveway type j
b = coefficient for traffic volume adjustment from Table 70

The N_j term in Equation (54) includes all driveways on both sides of the road.

Seven specific driveway types have been considered in modeling. These are:

- Major commercial driveways
- Minor commercial driveways
- Major industrial/institutional driveways
- Minor industrial/institutional driveways
- Major residential driveways
- Minor residential driveways
- Other driveways

Major driveways are those that serve sites with 50 or more parking spaces. Minor driveways are those that serve sites with less than 50 parking spaces. Commercial driveways provide access to establishments that serve retail customers. Residential driveways serve single- and multiple-family dwellings. Industrial/institutional driveways serve factories, warehouses, schools, hospitals, churches, public facilities, and other places of employment.

Driveway-related collisions can be broken down by severity level as follows:

$$N_{brdwy(FI)} = N_{brdwy}\, f_{dwy} \quad \textbf{(55)}$$

$$N_{brdwy(PDO)} = N_{brdwy} - N_{brdwy(FI)} \quad \textbf{(56)}$$

The values of N_j and f_{dwy} are shown in Table 70.

Adjustment Factor to Estimate Vehicle-Pedestrian Collisions (N_{pedr})

The number of vehicle-pedestrian collisions per year for a roadway segment is estimated as:

$$N_{pedr} = N_{br}\, f_{pedr} \quad \textbf{(57)}$$

where:

f_{pedr} = pedestrian safety adjustment factor.

TABLE 70. Driveway factors for roadway segments

Driveway type	Coefficients for specific roadway types: Two-lane undivided	Three-lane with TWLTL	Four-lane undivided	Four-lane divided	Five-lane with TWLTL
Number of driveway-related collisions per driveway per year (N_j)					
Major commercial	0.252	0.164	0.202	0.053	0.131
Minor commercial	0.080	0.052	0.064	0.017	0.042
Major industrial/institutional	0.274	0.178	0.220	0.057	0.143
Minor industrial/institutional	0.036	0.024	0.029	0.008	0.019
Major residential	0.132	0.086	0.106	0.028	0.069
Minor residential	0.025	0.016	0.020	0.005	0.013
Other	0.040	0.026	0.032	0.008	0.021
Regression coefficient for ADT (b)					
All driveways	1.000	1.000	1.172	1.106	1.172
Proportion of fatal-and-injury accidents (f_{dwy})					
All driveways	0.323	0.243	0.342	0.284	0.269
Proportion of property-damage-only accidents					
All driveways	0.677	0.757	0.658	0.716	0.731

NOTE: Includes only unsignalized driveways; signalized driveways are analyzed as signalized intersections. Major driveways serve 50 or more parking spaces; minor driveways serve less than 50 parking spaces.

Table 71 presents the values of f_{pedr} for use in Equation (57). All vehicle-pedestrian collisions are considered to be fatal-and-injury accidents.

TABLE 71. Pedestrian safety adjustment factors for urban and suburban areas

Road type	Pedestrian safety adjustment factor (f_{pedr}): Urban	Suburban
2U	0.033	0.003
3T	0.034	0.001
4U	0.045	0.008
4D	0.019	0.006
5T	0.039	0.004

Note: These factors apply to the methodology for predicting total accidents (all severity levels combined). All pedestrian collisions resulting from this adjustment factor should be treated as fatal-and-injury accidents and none as property-damage-only accidents.

Table 71 will eventually be replaced with a pedestrian safety prediction methodology which is under development in Phase III of NCHRP Project 17-26.

Adjustment Factor to Estimate Vehicle-Bicycle Collisions (N_{biker})

The number of vehicle-bicycle collisions per year for a roadway segment is estimated as:

$$N_{biker} = N_{br}\, f_{biker} \tag{58}$$

where:

f_{biker} = bicycle safety adjustment factor.

Table 72 presents the values of f_{biker} for use in Equation (58). All vehicle-bicycle collisions are considered to be fatal-and-injury accidents.

TABLE 72. Bicycle safety adjustment factors (f_{biker}) for roadway segments

Road type	Bicycle safety adjustment factor (f_{biker})	
	Urban	Suburban
2U	0.015	0.005
3T	0.026	0.002
4U	0.029	0.010
4D	0.011	0.011
5T	0.023	0.003

Calibration Factor for Roadway Segments (C_r)

The calibration factor (C_r) in Equation (45) is intended to be developed for specific roadway segment types by each agency using the HSM methodologies. Typically, new calibration factors are determined for each year of data.

The calibration factor is needed because the base models are developed from data for particular highway agencies (in this case, highway agencies in Minnesota and Michigan), while the HSM methodology may be applied by a highway agency in any part of the U.S. The calibration process is intended to compensate for differences in climate, driver population, and (in the case of rural methodologies) animal population in different regions of the U.S. Calibration also adapts the methodologies to years other than the years for which the models were developed.

The calibration process can generally be implemented by identifying a set of sites whose characteristics are similar to the nominal or base conditions. The HSM methodology is then applied to the site, accident history data are obtained for the site, and the calibration factor is determined as the ratio of observed accidents to predicted accidents, as follows:

$$C_r = \frac{\text{Observed accident count for a group of roadway segments}}{\text{Predicted accident count for the same group of roadway segments}} \tag{59}$$

The description of the calibration process presented in the prototype chapter can be rewritten and simplified. It is likely that a single calibration process can be provided for HSM Chapters 8, 9, and 10. Potentially, this process would need to appear only once in the HSM and could be referred to from other chapters.

Accident Modification Factors

The safety prediction methodology for arterial roadway segments will include up to four accident modification factors (AMFs):

- on-street parking
- roadside fixed objects/roadside hazard rating
- lighting

The AMFs were developed based on assessments made by an expert panel convened jointly by NCHRP Projects 17-25 and 17-26. Each of these AMFs is discussed below.

On-Street Parking (AMF_{1r})

The AMF for on-street parking is based on recent work by Bonneson (*129*) for TexDOT. The AMF is determined as:

$$AMF_{1r} = 1 + p_{pk}\,(f_{pk} - 1) \qquad \textbf{(60)}$$

where:

f_{pk} = factor from Table 73
p_{pk} = proportion of curb length with on-street parking = ($0.5\ L_{pk}/L$)
L_{pk} = curb length with on-street parking (mi)

Table 73 is based on Table 60. The two-lane values from Table 60 have been applied to all road types with two through lanes (2U and 3T) and the multilane values have been applied to all road types with more than two through lanes (4U, 4D, and 5T).

TABLE 73. Values of f_{pk} used in determining the accident modification factor for on-street parking

	Type of parking and land use			
	Parallel parking		Angle parking	
Road type	Residential/other	Commercial or industrial/institutional	Residential/other	Commercial or industrial/institutional
2U	1.465	2.074	3.428	4.853
3T	1.465	2.074	3.428	4.853
4U	1.100	1.709	2.574	3.999
4D	1.100	1.709	2.574	3.999
5T	1.100	1.709	2.574	3.999

Roadside Fixed Objects (AMF_{2r})

An AMF for roadside fixed objects has been adapted from the work of Zegeer and Cynecki (*130*) on predicting utility pole accidents. The AMF is determined with the following equation:

$$AMF_{2r} = f_{offset}\, D_{fo}\, p_{fo} + (1 - p_{fo}) \qquad (61)$$

where:

f_{offset} = fixed-object offset factor from Table 74
D_{fo} = fixed-object density (fixed objects/mi)
p_{fo} = fixed-object collisions as a proportion of total accidents from Table 75

This AMF applies to total roadway segment accidents.

TABLE 74. Fixed-object offset factor (f_{offset})

Offset to fixed objects (O_{fo}) (ft)	Fixed-object offset factor (f_{offset})
2	0.232
5	0.133
10	0.087
15	0.068
20	0.057
25	0.049
30	0.044

TABLE 75. Proportion of fixed-object collisions (p_{fo})

Road type	Proportion of fixed-object collisions (p_{fo})
2U	0.059
3T	0.034
4U	0.037
4D	0.036
5T	0.016

In estimating the density of fixed objects (D_{fo}), point objects that are within 21 m (70 ft) of one another longitudinally along the road should be counted as a single object. This distance of 21 m (70 ft) is rounded from the distance of 22 m (72 ft) shown in Section 5 of this report. Continuous objects that are not behind point objects should be counted as one point object for each 21 m (70 ft) of length. Only point or continuous objects within the offset distance used to derive the value of f_{offset} should be used in determining D_{fo}.

Lighting (AMF_{3r})

Based on the work of Elvik and Vaa (*131*), the AMF for lighting of roadway segments that have not previously been lighted is determined as:

$$AMF_{3r} = 1 - \left((1 - 0.36\ p_{fnr} - 0.72\ p_{inr} - 0.83\ p_{pnr})\ p_{nr}\right) \tag{62}$$

where:

p_{fnr} = proportion of total nighttime accidents for unlighted roadway segments that involve a fatality
p_{inr} = proportion of total nighttime accidents for unlighted roadway segments that involve only a nonfatal injury
p_{pnr} = proportion of total nighttime accidents for unlighted roadway segments that involve property damage only
p_{nr} = proportion of total accidents for unlighted roadway segments that occur at night

This AMF applies to total roadway segment accidents. Table 76 presents default values for the nighttime accident proportions p_{fnr}, p_{inr}, p_{pnr}, and p_{nr}.

TABLE 76. Nighttime accident proportions for unlighted roadway segments

Road type	Proportion of total nighttime accidents by severity level[a]			Proportion of accidents that occur at night[a]
	Fatal p_{fnr}	Injury p_{inr}	PDO p_{pnr}	p_{nr}
2U	0.008	0.240	0.750	0.266
3T	0.030	0.300	0.670	0.279
4U	0.001	0.290	0.709	0.230
4D	0.001	0.290	0.709	0.204
5T	0.001	0.273	0.726	0.391

[a] Based on combined data for Minnesota and Michigan.
NOTE: HSM users are encouraged to replace these estimates with locally derived values.

INTERSECTION METHODOLOGY

General Form of Intersection Methodology

Safety predictions for a particular intersection will be developed as a combination of base models, calibration factors, and accident modification factors (AMFs) using the following general approach:

$$N_{int} = (N_{bi} + N_{pedi} + N_{bikei})\ C_i \tag{63}$$

$$N_{bi} = N_{bibase}\ (AMF_{1i}\ AMF_{2i}\ AMF_{3i}\ AMF_{4i}\ AMF_{5i}) \tag{64}$$

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 (excluding vehicle-pedestrian and vehicle-bicycle collisions)
N_{bibase}	=	predicted number of total intersection-related accidents per year for nominal or base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions)
N_{pedi}	=	predicted number of vehicle-pedestrian collisions per year
N_{bikei}	=	predicted number of vehicle-bicycle collisions per year
C_i	=	calibration factor for at-grade intersections developed for use for a particular geographical area [see Equation (75)]
AMF_{1i} … AMF_{5i}	=	accident modification factors for intersections

Types of Intersections Considered

The safety prediction methodology addresses four types of intersections on arterial roadways:

- Three-leg intersections with STOP control on the minor-road approach (3ST)
- Three-leg signalized intersections (3SG)
- Four-leg intersections with STOP control on the minor-road approach (4ST)
- Four-leg signalized intersections (4SG)

Base Models and Adjustment Factors for Intersections (N_{bimv}, N_{bisv}, N_{pedi}, and N_{bikei})

Base models are used to make predictions for multiple-vehicle intersection-related accidents. Single-vehicle accidents constitute a relatively small proportion of the accidents at intersections; statistically significant base models were developed in some cases, but a decision may be reached to predict single-vehicle collisions for most intersections with tabulated factors based on the average proportion of accidents. Tabulated factors are also used to account for pedestrian and bicycle collisions. The tabulated pedestrian factors will eventually be replaced with a pedestrian safety prediction methodology that is under development in Phase III of NCHRP Project 17-26.

Equation (63) shows that total intersection accidents are determined as the sum of three components: N_{bi}, N_{pedi}, and N_{bikei}. The following equation shows that the base model portion of N_{bi} designated as N_{bibase}, is further broken down into two components:

$$N_{bibase} = N_{bimv} + N_{bisv} \tag{65}$$

where:

N_{bimv} = predicted number of multiple-vehicle collisions per year for nominal or base conditions
N_{bisv} = predicted number of single-vehicle collisions per year for nominal or base conditions

Thus, the base models and adjustment factors are applied to determine four components of total intersection accident experience: N_{bimv}, N_{bisv}, N_{pedi}, and N_{bikei}.

Base Models for Multiple-Vehicle Collisions (N_{bimv})

Base models for multiple-vehicle intersection-related collisions are applied as follows:

$$N_{bimv} = \exp\left(a + b \ln ADT_{maj} + c \ln ADT_{min}\right) \quad \textbf{(66)}$$

where:

ADT_{maj} = average daily traffic volume (veh/day) for major road
ADT_{min} = average daily traffic volume (veh/day) for minor road
a, b, c = regression coefficients

Table 77 presents the values of the coefficients a, b, and c used in applying Equation (66).

TABLE 77. Base models for multiple-vehicle collisions at intersections

Intersection types	Coefficients used in Equation (66) Intercept (a)	ADT_{maj} (b)	ADT_{min} (c)	Overdispersion parameter (k)
Total accidents				
3ST	–13.39	1.11	0.41	0.80
3SG	–11.63	1.11	0.26	0.33
4ST	–8.97	0.82	0.25	0.40
4SG	–10.63	1.07	0.23	0.39
Fatal-and-injury accidents				
3ST	–14.04	1.16	0.30	0.69
3SG	–11.08	1.02	0.17	0.30
4ST	–11.20	0.93	0.28	0.48
4SG	–12.78	1.18	0.22	0.33
Property-damage-only accidents				
3ST	–15.41	1.20	0.51	0.77
3SG	–12.74	1.14	0.30	0.36
4ST	–8.81	0.77	0.23	0.40
4SG	–10.66	1.02	0.24	0.44

Equation (66) is first applied to determine N_{bimv} using the coefficients for total accidents in Table 77. The values of $N'_{bimv(FI)}$ and $N'_{bimv(PDO)}$ are then determined with Equation (66) using the coefficients for fatal-and-injury and property-damage-only accidents, respectively, in Table 77. The following adjustments are made to the values of $N'_{bimv(FI)}$ and $N'_{bimv(PDO)}$ to assure that they sum to N_{bimv}:

$$N_{bimv(FI)} = N_{bimv}\left(\frac{N'_{bimv(FI)}}{N'_{bimv(FI)} + N'_{bimv(PDO)}}\right) \tag{67}$$

$$N_{bimv(PDO)} = N_{bimv} - N_{bimv(FI)} \tag{68}$$

The percentages in Table 78 are used to break down $N_{bimv(FI)}$ and $N_{bimv(PDO)}$ by manner of collision. The distributions shown in Table 78 are based on observed collision data for all study sites in Minnesota and North Carolina. The overall distributions are used in the HSM methodology and are not predicted based on the characteristics of the specific site being evaluated.

TABLE 78. Distribution of multiple-vehicle collisions for intersections by collision type

Collision type	Proportion of accidents by severity level for specific intersection types							
	3ST		3SG		4ST		4SG	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Rear-end collision	0.494	0.481	0.530	0.511	0.422	0.428	0.506	0.507
Head-on collision	0.009	0.003	0.010	0.004	0.016	0.005	0.012	0.004
Angle collision	0.429	0.353	0.387	0.300	0.522	0.398	0.421	0.305
Sideswipe, same direction	0.044	0.095	0.049	0.129	0.026	0.084	0.029	0.123
Sideswipe, opposite direction	0.000	0.007	0.004	0.004	0.006	0.009	0.003	0.005
Other multiple-vehicle collisions	0.024	0.061	0.020	0.052	0.008	0.076	0.029	0.056

Base Models for Single-Vehicle Collisions (N_{bisv})

Base models for single-vehicle collisions at intersections are applied as follows:

$$N_{bisv} = \exp\left(a + b\ \ln ADT_{maj} + c\ \ln ADT_{min}\right) \tag{69}$$

Table 79 presents the values of the coefficients and factors used in Equation (69) for each roadway type. Equation (69) is first applied to determine N_{bisv} using the coefficients for total accidents in Table 79. The values of $N'_{bisv(FI)}$ and $N'_{bisv(PDO)}$ are then determined with Equation (69) using the coefficients for fatal-and-injury and property-damage-only accidents, respectively, in Table 79. The following adjustments are made to the values of $N'_{bisv(FI)}$ and $N'_{bisv(PDO)}$ to assure that they sum to N_{bisv}:

$$N_{bisv(FI)} = N_{bisv}\left(\frac{N'_{bisv(FI)}}{N'_{bisv(FI)} + N'_{bisv(PDO)}}\right) \tag{70}$$

$$N_{bisv(PDO)} = N_{bisv} - N_{bimv(FI)} \tag{71}$$

Since there are no models for fatal-and-injury accidents at three- and four-leg STOP-controlled intersections in Table 79, Equation (69) must be replaced with the following equation in these cases:

$$N_{bisv(FI)} = N_{bisv}\, f_{bisv} \qquad (72)$$

where:

f_{bisv} = proportion of fatal-and-injury accidents for combined sites from Table 57

TABLE 79. Base models for single-vehicle collisions at intersections

Intersection type	Coefficients used in Equation (69): Intercept (a)	ADT$_{maj}$ (b)	ADT$_{min}$ (c)	Overdispersion parameter (k)
Total accidents				
3ST	–6.84	0.16	0.51	1.14
3SG	–8.52	0.42	0.40	0.36
4ST	–5.40	0.33	0.12	0.65
4SG	–9.85	0.68	0.27	0.36
Fatal-and-injury accidents				
3ST				
3SG	–9.25	0.27	0.51	0.24
4ST				
4SG	–8.89	0.43	0.29	0.09
Property-damage-only accidents				
3ST	–8.39	0.25	0.55	1.29
3SG	–8.58	0.45	0.33	0.53
4ST	–7.11	0.36	0.25	0.54
4SG	–10.98	0.78	0.25	0.44

The percentages in Table 80 are used to break down $N_{bisv(FI)}$ and $N_{bisv(PDO)}$ by collision type. The distributions shown in Table 80 are based on observed collision data for all study sites in Minnesota and North Carolina. The overall distributions are used in the HSM methodology and are not predicted based on the characteristics of the specific site being evaluated.

TABLE 80. Distribution of single-vehicle collisions for intersections by collision type

Collision type	Proportion of accidents by severity level for specific intersection types: 3ST FI	3ST PDO	3SG FI	3SG PDO	4ST FI	4ST PDO	4SG FI	4SG PDO
Collision with parked vehicle	0.000	0.016	0.000	0.000	0.000	0.065	0.019	0.060
Collision with animal	0.022	0.031	0.026	0.084	0.000	0.023	0.000	0.024
Collision with fixed object	0.156	0.331	0.132	0.145	0.141	0.282	0.038	0.145
Collision with other object	0.067	0.087	0.000	0.084	0.056	0.181	0.019	0.072
Other single-vehicle collision	0.276	0.071	0.105	0.253	0.451	0.171	0.442	0.374
Noncollision	0.479	0.464	0.737	0.434	0.352	0.278	0.482	0.325

Adjustment Factor to Estimate Vehicle-Pedestrian Collisions (N_{pedi})

The number of vehicle-pedestrian collisions per year for an intersection is estimated as:

$$N_{pedi} = N_{bi} \, f_{pedi} \tag{73}$$

where:

f_{pedi} = pedestrian safety adjustment factor.

Table 81 presents the values of f_{pedi} for use in Equation (73). All vehicle-pedestrian collisions are considered to be fatal-and-injury accidents.

Table 81 will eventually be replaced with a pedestrian safety prediction methodology which is under development in Phase III of NCHRP Project 17-26.

TABLE 81. Pedestrian safety adjustment factors for intersections

Intersection type	Pedestrian safety adjustment factor (f_{pedi})
3ST	0.008
3SG	0.005
4ST	0.016
4SG	0.017

NOTE: These factors apply to the methodology for predicting total accidents (all severity levels combined). All pedestrian collisions resulting from this adjustment factor should be treated as fatal-and-injury accidents and none as property-damage-only accidents.

Adjustment Factor to Estimate Vehicle-Bicycle Collisions (N_{bikei})

The number of vehicle-bicycle collisions per year for an intersection is estimated as:

$$N_{bikei} = N_{br} \, f_{bikei} \tag{74}$$

where:

f_{bikei} = bicycle safety adjustment factor.

Table 82 presents the values of f_{bikei} for use in Equation (74). All vehicle-bicycle collisions are considered to be fatal-and-injury accidents.

Table 82 is intended for use in HSM Chapter 10, as there is no current plan for a bicycle safety prediction methodology analogous to the pedestrian safety prediction methodology currently under development.

TABLE 82. Bicycle safety adjustment factors for intersections

Intersection type	Bicycle safety adjustment factor (f_{bikei})
3ST	0.004
3SG	0.010
4ST	0.006
4SG	0.013

NOTE: These factors apply to the methodology for predicting total accidents (all severity levels combined). All pedestrian collisions resulting from this adjustment factor should be treated as fatal-and-injury accidents and none as property-damage-only accidents.

Calibration Factor (C_i)

The calibration factor (C_i) in Equation (63) is intended to be developed for specific intersection types by each agency using the HSM methodologies. Typically, new calibration factors are determined for each year of data.

The calibration factor is needed because the base models are developed from data for particular highway agencies (in this case, highway agencies in Minnesota and North Carolina), while the HSM methodology may be applied by a highway agency in any part of the U.S. The calibration process is intended to compensate for differences in climate, driver, population, and (in the case of rural methodologies) animal population in different regions of the U.S.

The calibration process can generally be implemented by identifying a set of sites whose characteristics are similar to the nominal or base conditions. The HSM methodology is then applied to the site, accident history data are obtained for the site, and the calibration factor is determined as the ratio of observed accidents to predicted accidents, as follows:

$$C_i = \frac{\text{Observed accident count for a group of intersections}}{\text{Predicted accident count for the same group of intersections}} \tag{75}$$

The description of the calibration process presented in the prototype chapter can be rewritten and simplified. It is likely that a single calibration process can be provided for HSM Chapters 8, 9, and 10. Potentially, this process would need to appear only once in the HSM and could be referred to from other chapters.

Accident Modification Factors

The safety prediction methodology for arterial intersections will include up to five AMFs:

- left-turn lanes
- left-turn signal phasing
- right-turn lanes
- right-turn on red
- lighting

These AMFs were developed based on assessments made by an expert panel convened jointly by NCHRP Projects 17-25 and 17-26. Each of these AMFs is discussed below.

Left-Turn Lanes (AMF_{1i})

The AMF for left-turn lanes at intersections on urban and suburban arterials based on a before-after study performed for FHWA by Harwood et al. (*81*) is presented in Table 83. This AMF applies to total intersection accidents.

TABLE 83. AMFs for left-turn lanes at intersections on urban and suburban arterials (*81*)

Type of traffic control	Number of major-road approaches on which left-turn lanes are provided: One approach	Two approaches
THREE-LEG INTERSECTIONS		
Two-way STOP control	0.67	–
Signal Control	0.93	–
FOUR-LEG INTERSECTIONS		
Two-way STOP control	0.73	0.53
Signal control	0.90	0.81

Left-Turn Signal Phasing (AMF_{2i})

The AMF for left-turn signal phasing is based on the results of work by Hauer (*132*), as modified in a recent study by Lyon et al (*133*). Types of left-turn signal phasing considered include permissive, protected/permissive, permissive/protected, and protected. The AMF values are presented in Table 84. This AMF applies to total intersection accidents and is applicable only to signalized intersections. The base condition for the AMF is permissive left-turn signal phasing, which corresponds to an AMF value of 1.00. An AMF value of 1.00 should always be used for unsignalized intersections.

The AMF values in Table 84 are based on the research results presented in Table 64.

TABLE 84. AMFs for left-turn signal phasing at signalized intersections

Type of left-turn signal phasing	AMF_{2i}
Permissive	1.00
Protected/permissive or permissive/protected	0.99
Protected	0.94

NOTE: Use AMF_{2i} = 1.00 for all unsignalized intersections.

Right-Turn Lanes (AMF_{3i})

The AMF for right-turn lanes at intersections on urban and suburban arterials based on a before-after study performed for FHWA by Harwood et al. (*81*) is presented in Table 85. This AMF applies to total intersection accidents.

TABLE 85. AMFs for right-turn lanes at intersections on urban and suburban arterials (*81*)

	Number of major-road approaches on which right-turn lanes are provided	
Type of traffic control	One approach	Two approaches
Two-way STOP control	0.86	0.74
Signal control	0.96	0.92

Right Turn On Red (AMF_{4i})

The AMF for prohibiting right turn on red on one or more approaches to a signalized intersection has been derived from a study by Clark (*134*). The AMF is determined as:

$$AMF_{4i} = (0.984)^{nprohib} \tag{76}$$

where:

nprohib = number of signalized intersection approaches for which right turn on red is prohibited

This AMF applies to total intersection accidents. The AMF is applicable only to signalized intersections. An AMF value of 1.00 should be used for unsignalized intersections.

Lighting (AMF_{5i})

The AMF for lighting of intersections has been derived from an evaluation of roadway segment lighting by Elvik and Vaa (*131*). The AMF for intersections that have not previously been lighted is determined as:

$$AMF_{5i} = 1 - [(1 - 0.36\ P_{fni} - 0.72\ p_{ini} - 0.83\ p_{pni})\ p_{ni}] \quad (77)$$

where:

p_{fni} = proportion of total nighttime accidents for unlighted intersections that involve a fatality

p_{ini} = proportion of total nighttime accidents for unlighted intersections that involve only a nonfatal injury

p_{pni} = proportion of total nighttime accidents for unlighted intersections that involve property damage only

p_{ni} = proportion of total accidents for unlighted intersections that occur at night

This AMF applies to total intersection accidents. Table 86 presents default values for the nighttime accident proportions p_{fni}, p_{ini}, p_{pni}, and p_{ni}.

TABLE 86. Nighttime accident proportions for unlighted intersections

Intersection type	Proportion of total nighttime accidents by severity level[a]			Proportion of accidents that occur at night[a]
	Fatal p_{fni}	Injury p_{ini}	PDO p_{pni}	p_{ni}
3ST	0.001	0.334	0.665	0.192
3SG	0.001	0.393	0.606	0.206
4ST	0.040	0.360	0.600	0.191
4SG	0.003	0.328	0.670	0.200

[a] Based on combined data for Minnesota and North Carolina.
NOTE: HSM users are encouraged to replace these estimates with locally derived values.

COMBINING PREDICTED AND OBSERVED ACCIDENT COUNTS

The methodology presented above is suitable for predicting accident counts for roadway segments and intersections on urban and suburban arterials. Where observed accident history data are available, the predicted and observed accident counts can be combined using the Empirical Bayes (EB) method presented earlier in this report [see Equations (15) and (16)]. Special EB procedures apply for this HSM methodology because of the use of multiple base models with different over dispersion parameters; these special procedures are shown in Equations (17) to (25). It is likely that the EB method could be presented in just one place in HSM Part III and then referred to from other Part III chapters. It is recommended that the EB method be applied to individual roadway segments and intersections [i.e., to the individual values of N_{rs} and N_{int} before application of Equation (44)] whenever the observed accident count data are sufficiently disaggregated to support this. However, it is expected that, in many cases, accident data will be available for the project as a whole rather than for individual roadway segments and intersections. In this case, the EB method should be applied to an arterial facility or improvement project as a whole using Equations (17) to (25).

WORKSHEETS AND PRECISION IN COMPUTATIONS

The draft version of HSM Chapter 10 presented in Appendix B of this report includes seven worksheets for applying the methodology. These are:

- safety prediction worksheet for a roadway segment
- safety prediction worksheet for an intersection
- safety prediction worksheet for an entire facility or project without observed accident data
- safety prediction worksheet to incorporate observed accident data for a single roadway segment
- safety prediction worksheet to incorporate observed accident data for a single intersection
- safety prediction worksheet to incorporate observed accident frequencies for a facility or project with site-specific accident data
- safety prediction worksheet to incorporate observed accident frequencies for a facility or project without site-specific accident data

All computations of accident frequencies within these worksheets are conducted with values expressed to three decimal places. It should not be envisioned that the methodology is accurate to three decimal places, but this level of precision is needed for consistency in computations. It is recommended that, in the last stage of computations, final estimates of predicted accident frequency should be rounded to one decimal place.

CHAPTER 8.

VALIDATION OF THE HSM METHODOLOGY

This chapter presents the validation of the HSM methodology documented in Chapter 7 and in Appendix B. The validation was conducted by applying the HSM methodology to arterial roadway segments in Washington and arterial intersections in Florida and comparing the safety predictions obtained from the methodology to observed accident frequencies for the Washington and Florida sites. The validation studies for roadway segments and for intersections are each discussed below.

ROADWAY SEGMENT VALIDATION STUDY

The HSM methodology for roadway segments was validated through application to a set of arterial segment sites under the jurisdiction of the Washington State Department of Transportation. As in the project database described in Chapter 4, the validation database for roadway segments consisted of individual blocks from one public road intersection to the next along urban and suburban arterial highways and streets. All of the sites were located in the Seattle-Tacoma-Olympia area in King, Pierce, Snohomish, and Thurston counties. The roadway segment validation data set consisted of 864 blocks including 143.0 mi of roadway. The average block length in the dataset is 0.17 mi. These roadways experience a total of 6,422 accidents in a 3-year period.

Table 87 summarizes the roadway segment validation sites and includes the number of blocks, total length, average ADT, total exposure, total number of accidents, and average accident rate for each roadway type. The ADT, exposure, accident frequency, and accident rate data represent a 3-year period from 2002 to 2004. Table 88 summarizes the ADT distribution for the validation sites in each roadway type.

TABLE 87. Summary of roadway segment accident and exposure data for Washington validation dataset

Roadway type	Number of sites	Total length (mi)	Average ADT (veh/day)	Total exposure (10^6 veh-mi)[a]	Observed total number of accidents[a]	Average accident rate (per 10^6 veh-mi)
2U	286	59.9	13,416	948.7	1,220	1.29
3T	47	6.0	16,146	111.9	190	1.70
4U	106	12.5	24,975	334.5	527	1.58
4D	54	10.9	29,382	396.8	408	1.03
5T	371	53.7	30,359	1,759.6	4,077	2.32

[a] In 3-year period.

TABLE 88. Descriptive statistics for roadway segment traffic volumes in Washington validation dataset

ADT statistic (veh/day)	Roadway type				
	2U	3T	4U	4D	5T
Mean	13,416	16,146	24,975	29,382	30,359
Median	13,580	15,986	25,852	26,591	30,116
Standard deviation	329	690	656	1,025	337
Minimum	3,081	9,378	6,644	17,463	10,582
Maximum	27,599	26,102	35,930	49,538	43,926

Roadway segment characteristics and traffic volumes to apply the HSM methodology were obtained from roadway inventory files maintained by the Washington State Department of Transportation and from review of videologs. Crash data were obtained from the FHWA HSIS.

Validation Procedure

The first step in applying the HSM methodology to the Washington sites was to calibrate the methodology to Washington conditions, which should also be the first step for any agency using the HSM methodology. The HSM methodology was applied to each site and year and a yearly calibration factor was computed for each roadway type as the ratio of the total observed accident frequency to the total predicted accident frequency for that roadway type. The calibration factors (C_r) determined from the Washington data were:

Roadway type	Calibration factor (C_r) for total accidents by analysis year			
	2002	2003	2004	Average
2U	0.93	0.82	0.86	0.87
3T	0.74	1.08	1.01	0.94
4U	1.26	1.20	1.04	1.17
4D	1.15	0.95	0.96	1.02
5T	1.59	1.49	1.45	1.51

These calibration factors indicate that, except for 5T roadways, the average accident frequency at the Washington sites is relatively close to the average accident frequency at comparable Minnesota and Michigan sites from which the HSM methodology was derived. It is also evident that there can be substantial variation in the calibration factor from one year to the next for the same set of sites. Similar calibration factors were developed for fatal-and-injury and property-damage-only accidents.

In one sense, the calibration process assures reasonable overall validation results because the total predicted and observed accident frequencies for each roadway type are forced to be equal over the 3-year period. On the other hand, a calibration process of this type is essential when the HSM methodology is applied in any new jurisdiction. And, ultimately, the validation should be judged on the basis of agreement between predicted and observed accident frequencies for individual sites over several years and not for a group of sites as a whole. Calibration using only part of the validation dataset was tried, but the available sample sizes appeared too small to permit this. Therefore, calibration was performed with the entire validation data set.

The HSM methodology was then reapplied to each of the Washington sites, separately for each year, incorporating the appropriate yearly calibration factor, C_r. The predicted accident frequency for the 3-year period at each site was then compared to the observed accident frequency for the same 3-year period. The validation results are presented below.

Validation Results

Figures 24 through 28 show for the five roadway types, plots of the predicted and observed accident frequencies for each site normalized for site lengths and duration of study period (acc/mi/yr). The line of equality which would represent perfect agreement between predicted and observed accident frequencies is shown on each plot.

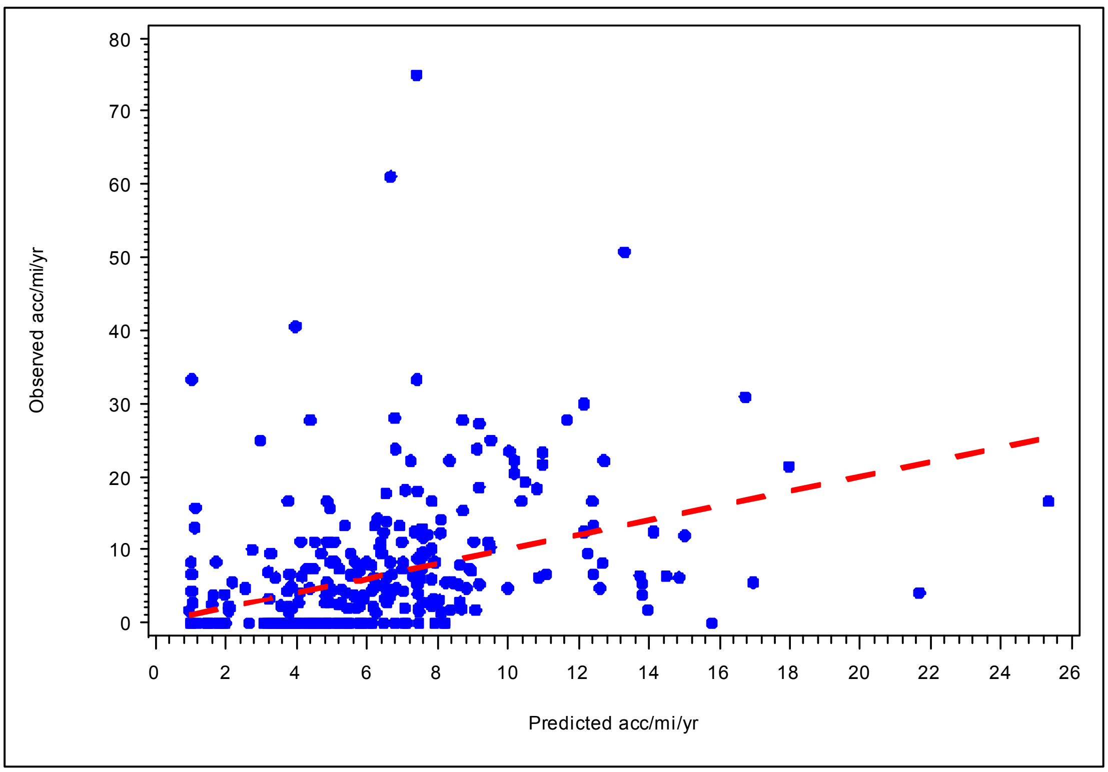

Figure 24. Comparison of total observed vs. predicted accident frequencies for 2U roadway segments in Washington

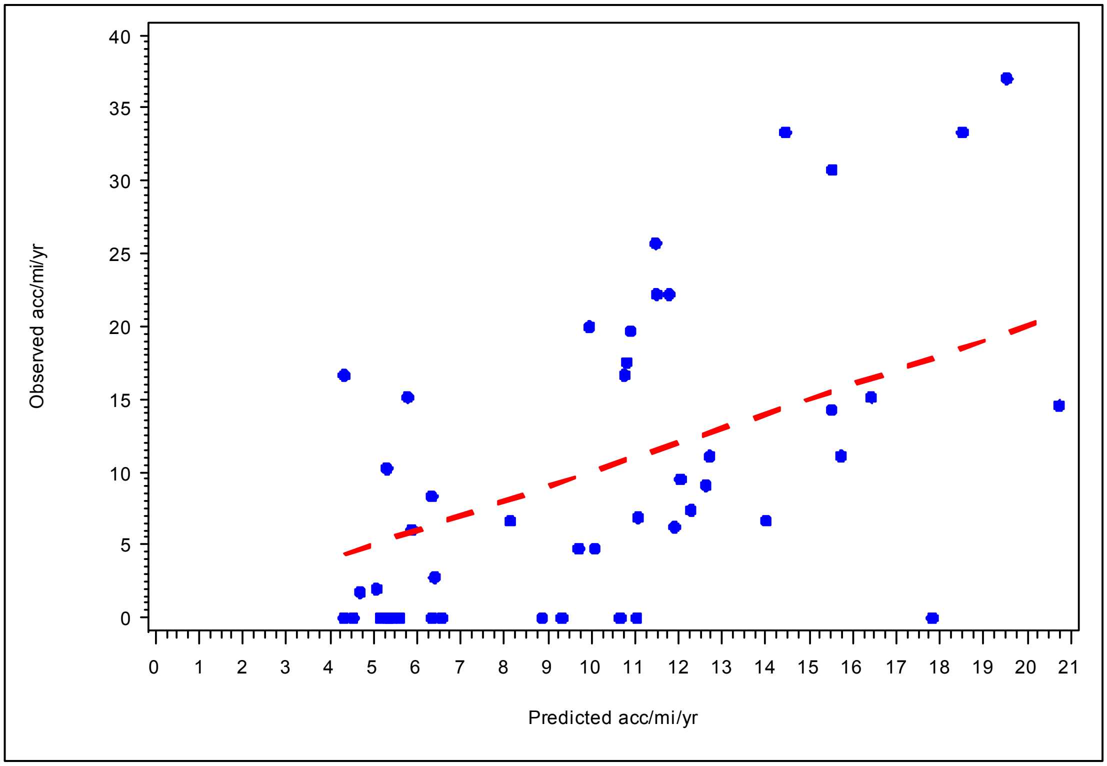

Figure 25. Comparison of total observed vs. predicted accident frequencies for 3T roadway segments in Washington

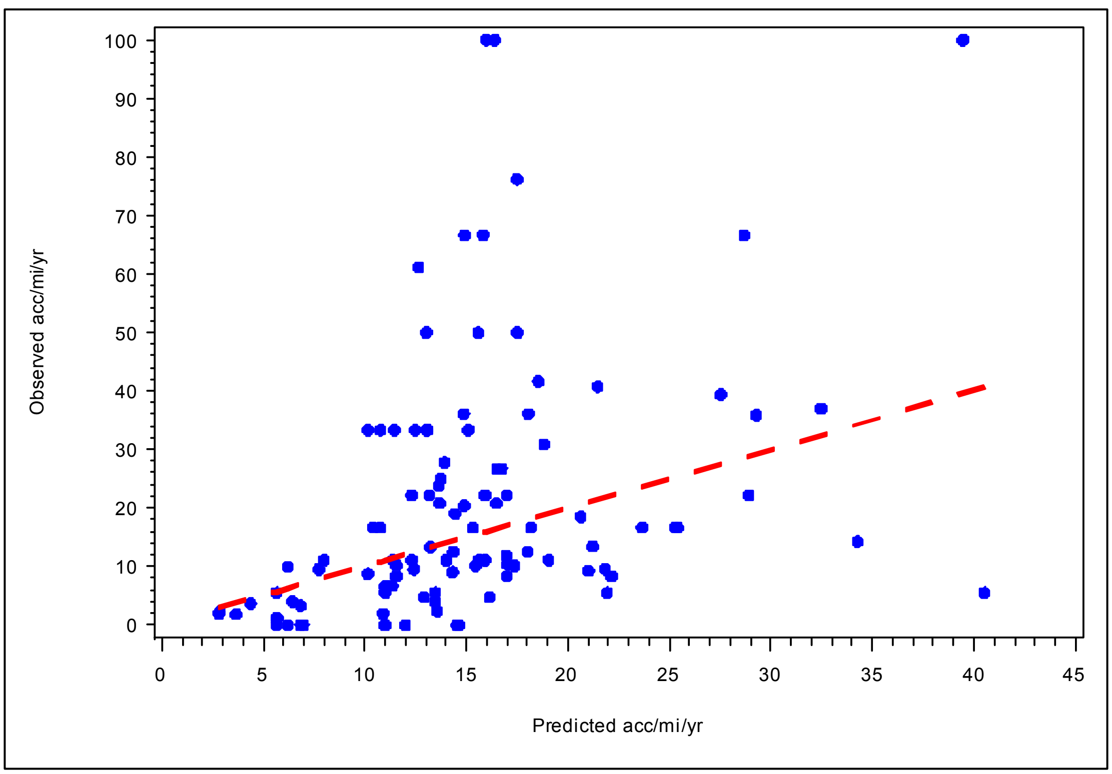

Figure 26. Comparison of total observed vs. predicted accident frequencies for 4U roadway segments in Washington

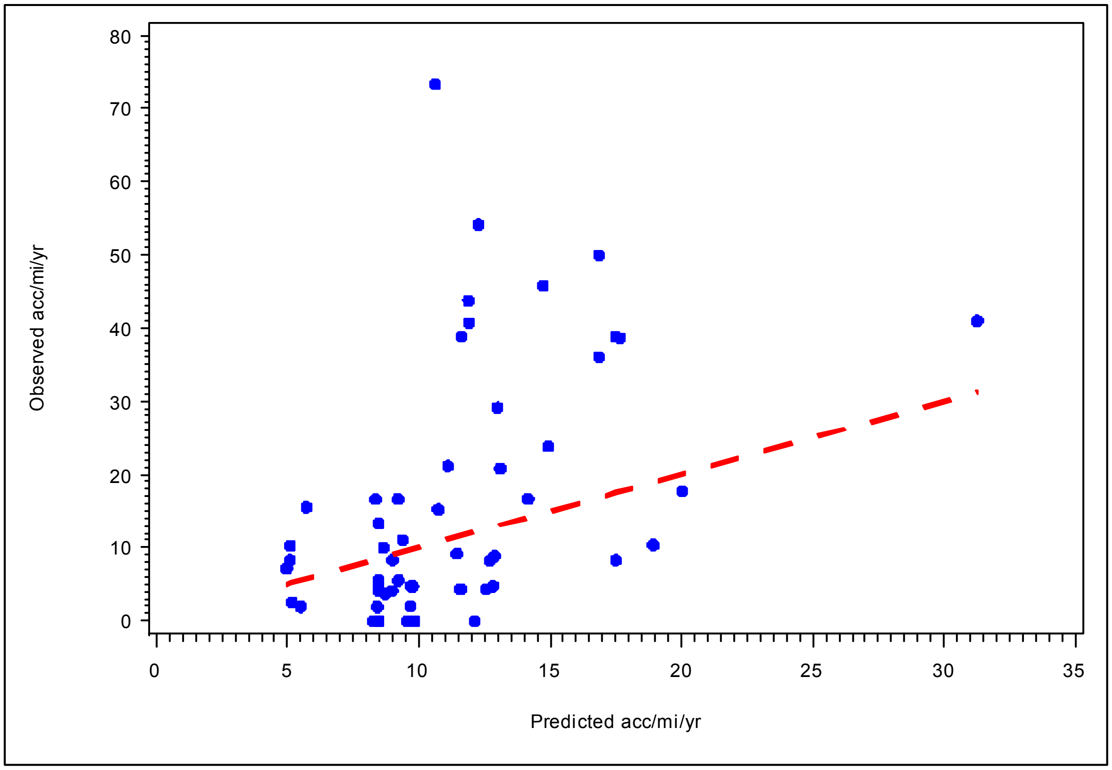

Figure 27. Comparison of total observed vs. predicted accident frequencies for 4D roadway segments in Washington

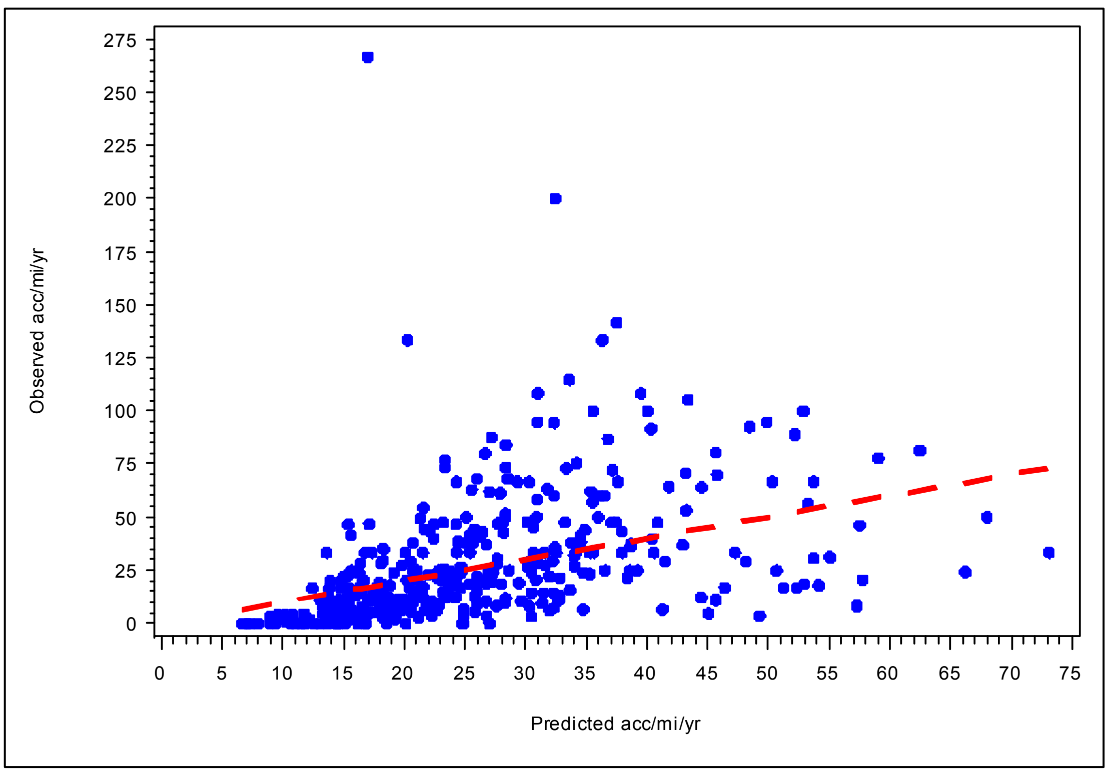

Figure 28. Comparison of total observed vs. predicted accident frequencies for 5T roadway segments in Washington

In reviewing the scatter plots, it should be recognized that perfect agreement between predicted and observed accident frequencies should not be expected, no matter how good the HSM methodology is. The predicted accident frequencies, at their best, represent an estimate of the long-term average accident frequency for similar sites. The observed accident frequency is simply one observation from a random process whose mean is estimated by the predicted accident frequency. There is no reason to expect that one observation from a random process should exactly equal the long-term mean.

The plots in Figures 24 through 28 each show a fair amount of spread around their line of equality. A few general comments about these plots are made.

- As in all observational studies, each group of roadway segments exhibits a number of extreme observed values. For example, an individual roadway segment may experience an unusually high number of accidents of any or all types; or a very short roadway segment may experience a relatively small number of accidents that, when expressed on a per-mile basis, corresponds to an unusually high accident frequency. Thus, there is inherently greater variability in the observed accident frequencies than in the predicted accident frequencies.
- The methodology cannot predict an accident frequency of zero because each SPF has a positive intercept. This is reasonable because no site can ever be expected to be accident-free in the long term. However, in any given time period, it is reasonable to expect that many sites, particularly lower volume sites, will experience zero observed accidents.
- For the sites with zero observed accident frequencies (i.e., points corresponding to zero on the y-axis), each plot shows a wide range of predicted accident frequencies on the x-axis. For example, in Figure 24 (2U roadway segments), the predicted total accident frequencies for sites with zero observed total accidents range from 1 to 15.8 acc/mi/yr. Similarly, in Figure 28 (5T roadway segments), predicted total accident frequencies for sites with zero observed total accidents range from 6.7 to 27.1 acc/mi/yr. This wide range simply reflects the variety in site characteristics of sites that, by chance, experienced no observed accidents over the 3-year period.

To assess the predictive capability of the methodology, a number of measures of the level of agreement between predicted and observed accident frequencies have been derived. These are:

- mean absolute error
- ratio of observed to predicted value (minimum, median, and maximum)
- Pearson correlation coefficient (lower and upper 95-percent confidence limits)
- percent of over- and underpredicted values
- average probability of more extreme accident frequency
- percentage of extreme observations at the 5-percent significance level

Table 89 presents the values of these validation measures for the Washington dataset. The first column in the table identifies the roadway type for the sites in question. The second and third columns provide sample size and average observed accident frequency (acc/mi/yr) for each roadway and accident type. The fourth through thirteenth columns of the table present the individual validation measures listed above. Each of these measures is discussed below.

TABLE 89. Validation measures for Washington roadway segments

Roadway type	Number of sites	Mean observed accident frequency (acc/mi/yr)	MAE[a] (acc/mi/yr)	Ratio of observed/predicted accident frequency			Pearson correlation coefficient		Percent over-predicted	Percent under-predicted	Average probability of more extreme accident frequency	Percent of extreme observations at 5% significance level
				Minimum	Median	Maximum	Lower 95% confidence limit	Upper 95% confidence limit				
Total accidents												
2U	286	7.39	5.69	0	0.75	32.15	0.53	0.68	59	41	0.27	5.6
3T	47	10.00	7.08	0	0.70	3.84	0.59	0.85	66	34	0.28	2.1
4U	106	20.73	13.20	0	0.88	6.25	0.51	0.74	56	44	0.27	4.7
4D	54	16.08	10.90	0	0.90	6.89	0.52	0.81	52	48	0.27	5.6
5T	371	28.43	17.25	0	0.79	15.66	0.62	0.73	62	38	0.23	5.9
Fatal-and-injury accidents												
2U	286	3.03	2.92	0	0.40	13.44	0.51	0.66	66	34	0.25	4.2
3T	47	4.09	3.51	0	0.57	9.95	0.56	0.84	64	36	0.29	2.1
4U	106	7.69	5.68	0	0.83	9.30	0.47	0.71	59	41	0.31	0.9
4D	54	5.65	5.08	0	0.57	5.80	0.48	0.79	57	43	0.26	3.7
5T	371	11.57	7.89	0	0.77	14.83	0.58	0.70	60	40	0.28	3.8
Property-damage-only accidents												
2U	286	4.36	3.66	0	0.69	58.86	0.48	0.64	61	39	0.26	7.7
3T	47	5.91	4.77	0	0.65	3.19	0.50	0.82	64	36	0.28	4.3
4U	106	13.04	10.09	0	0.91	6.65	0.44	0.70	53	47	0.25	3.8
4D	54	10.42	7.10	0	1.00	8.92	0.47	0.79	48	52	0.27	9.3
5T	371	16.86	11.05	0	0.72	16.29	0.60	0.71	62	38	0.23	7.8

[a] Mean absolute error.

Better agreement than shown in Figures 24 through 28 and Table 89 could have been obtained if the Empirical Bayes approach presented in Chapters 4 and 7, which is part of the recommended prediction methodology, had been used. The EB approach estimates the expected accident frequency as a weighted average of the predicted and observed accident frequencies. However, the EB approach was not used in the validation study; since the purpose of the validation study is to compare the predicted and observed values, it seemed inappropriate to move the predicted value toward the observed value by averaging them.

Mean Absolute Error (MAE)

The mean absolute error (MAE) is based on the absolute differences between predicted and observed accident frequencies (acc/mi/yr) and is computed as:

$$MAE = \frac{1}{n}\sum_{i=1}^{n} \left|N_i - O_i\right| \tag{78}$$

where N_i and O_i are the predicted and observed accident frequencies for site i and n is the validation sample size.

MAE is measured on the same scale as the original data, with smaller values indicating better predictive capability of the model. MAE is based on the absolute value of the differences between N_i and O_i, because positive and negative differences would cancel one another if the absolute value were not taken. Because the MAE is based on absolute differences, it does not provide an overall indication of whether the methodology over- or underpredicts, on average. The MAE measure, as all means, is very sensitive to large differences. Furthermore, differences of equal magnitude contribute equally to the MAE, whether these differences are observed at a low or high accident frequency (e.g., a difference of 2 vs. 1 or 31 vs. 30 yields the same difference of 1 acc/mi/yr).

A comparison of the mean observed accident frequencies and the MAE in Table 89 shows that the absolute errors in predictions are relatively high in each roadway category and for each accident type. One can interpret the MAE as an estimate of the standard deviation in accident frequencies and thus compare the square of the MAE (equivalent to the variance) to the mean accident frequency since accident frequencies are assumed to be NB distributed. The over dispersion in accident data will account for some of the discrepancy between these two measures, but not all.

The large mean absolute errors are also a reflection of the sensitivity of the mean to extreme values. This is illustrated in Figures 29 and 30 which show the variation of absolute errors [$|N_i - O_i|$] for total accidents for four-lane undivided and four-lane divided roadway segments, respectively, over the range of predicted accident frequencies. In Figure 29, although the absolute errors range up to 84 total acc/mi/yr, a large percentage of the errors (89 percent) are below 23 total acc/mi/yr. Similarly, in Figure 30, although the absolute errors range up to 63 total acc/mi/yr, a large percentage of the errors (87 percent) are below 21 total acc/mi/yr.

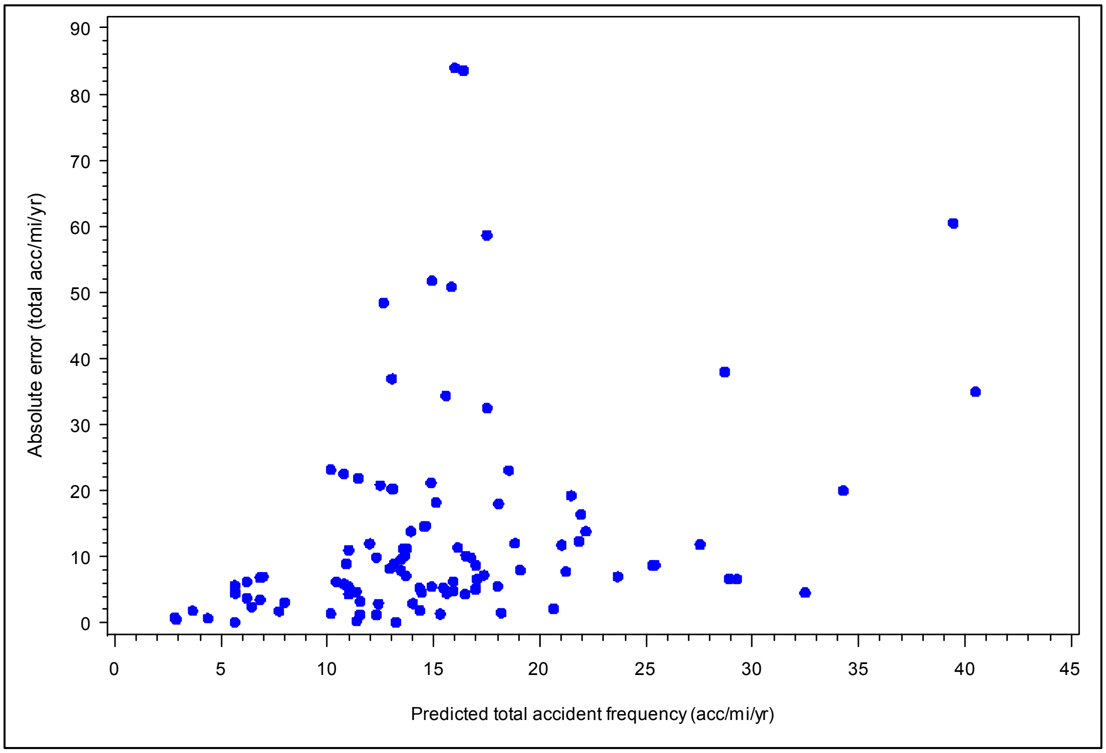

Figure 29. Variation of absolute error with predicted total accident frequency for 4U roadway segments in Washington

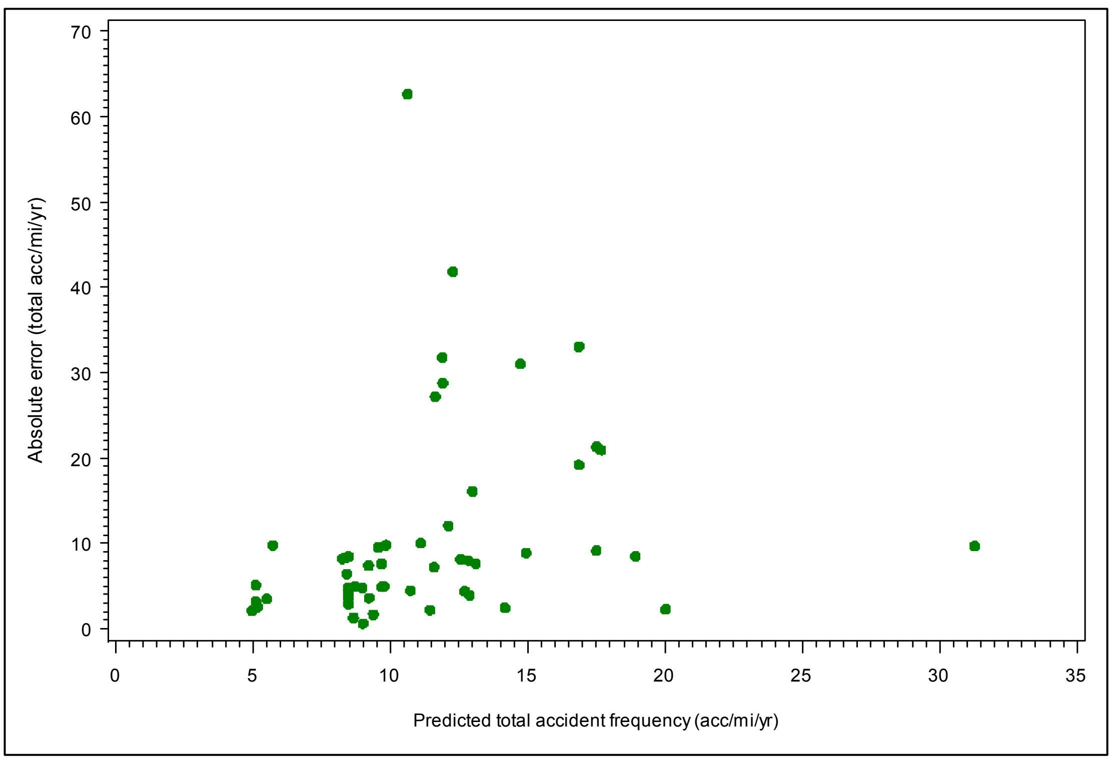

Figure 30. Variation of absolute error with predicted total accident frequency for 4D roadway segments in Washington

Some of the points in the upper range of absolute error in Figures 29 and 30 are clearly outliers, with very high observed accident frequencies and/or very short segment lengths. Accident patterns at several outlier sites have been reviewed. However, no outliers were removed from the data because, as discussed later in this section, the overall percentage of extreme observations was not higher than expected.

Ratio of Observed to Predicted Value

For each site, the ratio of observed to predicted accident frequency (R_i) is computed as O_i/N_i. The minimum, maximum, and median ratios across all sites of a given roadway type are then tabulated to summarize the distribution of the ratios. Because the distribution of these ratios is not symmetrical, the median is a more appropriate statistic than the mean to represent the central tendency of the data. Although an individual site ratio provides an indication of overprediction ($R_i < 1$) or underprediction ($R_i > 1$) at that particular site, the mean ratio is a biased estimate because overpredicted ratios are bounded by 0 and 1 while underpredicted ratios are only bounded by 1 on the low end and do not have an upper bound.

Minimum, median, and maximum ratios are shown in Table 89. The minimum in all cases is zero since at least one site in each category experienced no accidents over the 3-year validation period. Although the maximum ratios are quite high, especially for 2U and 5T roadways, the median ratio is the most interesting statistic to summarize these results. Except for property-damage-only accidents on 4D roadways (ratio=1.00), the median ratio is at most 0.91, an indication that for half the sites in each roadway category, the methodology overpredicts, which is to be expected for NB distributions. This finding is also in line with the percent of overpredicted results shown in the tenth column of Table 89.

Figures 31 and 32 illustrate the observed/predicted ratio as it varies with predicted accident frequency (acc/site/year) for 4U and 4D roadway segments, respectively. It is clear from these plots that the ratio varies most when the predicted accident frequency is low. For example, in Figure 31, the ratio of observed to predicted accident frequencies varies between 0 and 6.25 for predicted accident frequencies below 2.25 acc/site/yr. However, for predicted accident frequencies above 2.25 acc/site/yr, the ratio varies between 0 and 1.64. Similarly, as shown in Figure 32, the ratio varies only between 0 and 6.89 for predicted accident frequencies below 2.08 acc/site/yr, while for predicted accident frequencies above 2.08 acc/site/yr, the ratio varies between 0.21 and 2.19.

For predicted accident frequencies above 4 acc/site/yr at 4U roadways, the ratio varies between 0.7 and 1.6 (Figure 31). For predicted accident frequencies above 5 acc/site/yr at 4D roadways, the ratio varies between 0.3 and 1.6 (Figure 32).

The plots in Figures 31 and 32 illustrate a fundamental issue in the validation results. The plots illustrate that, for sites with high predicted accident frequencies, the observed and predicted accident frequencies agree quite well. However, for sites with low predicted accident frequencies, the observed accident frequencies vary widely in relative terms; in this case, the observed accident frequency can be relatively low or relatively high. This finding is illustrated

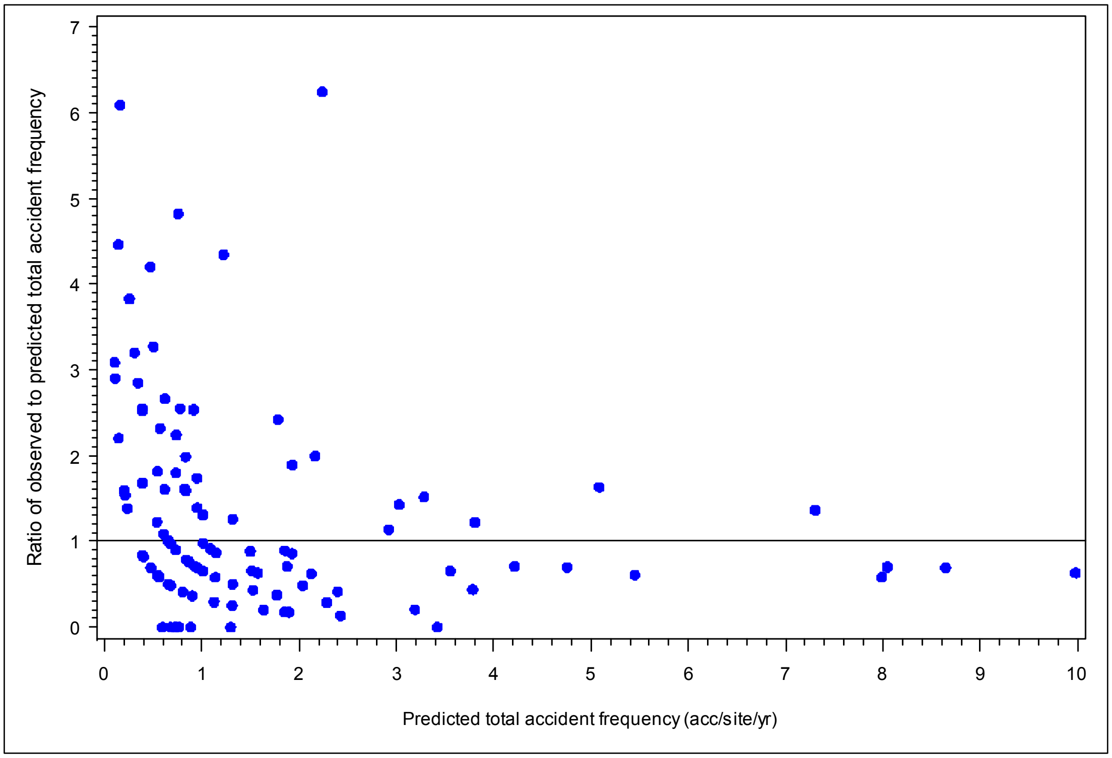

Figure 31. Variation of ratio of observed to predicted total accident frequency with predicted accident frequency for 4U roadway segments in Washington

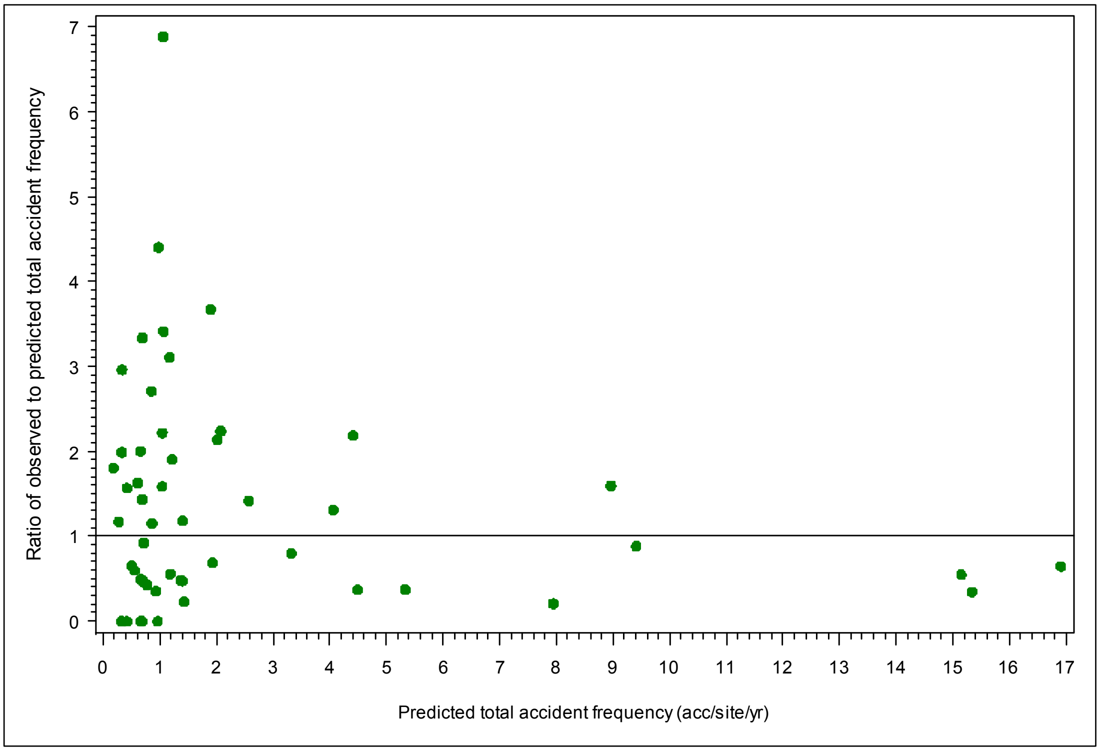

Figure 32. Variation of ratio of observed to predicted total accident frequency with predicted accident frequency for 4D roadway segments in Washington

by the large vertical range of data on the left side of the plots in Figures 31 and 32 and the limited vertical range of data on the right side of the same plots. In part, this result simply reflects the mathematics of small numbers; when the predicted value is low, even variations in observed accident frequency that are small in absolute terms may be large in relative terms. However, it is also likely that this represents an effect of variables that are not part of the predictive methodology. Driver behavioral variables, such as alcohol and safety belt usage, undoubtedly vary unevenly over the road network and such site-to-site variations are not accounted for by the HSM methodology. Clearly, the relative effect of such variations is greater at sites with low predicted accident frequencies than at sites high predicted accident frequencies.

Pearson Correlation Coefficient

The Pearson correlation coefficient, r, is a statistic that measures the correlation or degree of linear relationship between the observed and predicted accident frequencies (acc/site/year) and is calculated as:

$$r = \frac{\sum_{i=1}^{n} \left(O_i - \overline{O}\right)\left(N_i - \overline{N}\right)}{\sqrt{\sum_{i=1}^{n} \left(O_i - \overline{O}\right)^2 \sum_{i=1}^{n} \left(N_i - \overline{N}\right)^2}} \qquad (79)$$

where $\overline{N}$ and $\overline{O}$ represent the mean predicted and observed accident frequencies, respectively.

A value close to 1.0 for the correlation coefficient would indicate nearly perfect agreement between the observed and predicted accident frequencies, indicating that the methodology would have excellent predictive performance. A value close to zero for the correlation coefficient would indicate very poor predictive performance of the methodology. Thus, the correlation coefficient provides a measure of how tightly the points in Figures 24 through 28 fit around a straight line, but not necessarily the line of equality drawn in each figure. To best account for the effect of sample size on the correlation coefficient, the lower and upper 95-percent confidence limits for the correlation coefficient, rather than the correlation coefficient itself, are presented in Table 89. All of the correlation coefficients presented in Table 89 are significantly different from zero at the 95-percent confidence level, indicating that predicted and observed accident frequencies are significantly correlated. The correlation coefficients are relatively high as confirmed by their 95-percent confidence intervals. For total accidents, the lower 95-percent confidence limit of the correlation coefficient varies between 0.51 and 0.62, while the upper 95-percent confidence limit varies between 0.68 and 0.85, across the five roadway types. Obviously, the roadway groups with the smallest sample sizes (3T and 4D) produce the widest confidence interval for the correlation coefficient. Similar patterns are shown for fatal-and-injury and property-damage-only accidents.

The correlation coefficients are, in a sense, expected to be relatively high since both observed and predicted accident frequencies are 3-year averages. If one were to compare the three yearly observed accident frequencies (these are simply three observations from a NB distribution) to the three predicted accident frequencies (these show very little variability, if any, due to minor year-to-year variations in ADT only), then the correlation between observed and predicted would be expected to be lower. An extreme case would be to correlate an almost constant predicted accident frequency of one or many sites with the same ADT but many years of accident data to the observed accident frequencies—in that case, the correlation coefficient should be close zero for all practical purposes.

The values of correlation coefficients, in the overall range from 0.51 to 0.85, are consistent with a methodology based on SPFs that are statistically significant, but vary in their goodness of fit (see Chapter 5).

Percent of Overpredicted and Underpredicted Values

The percentage of values that are either overpredicted or underpredicted, calculated by summing counts of positive or negative errors across sites and dividing by the total validation sample size, provides another measure of the predictive capability of the methodology. Since these values are complements of each other, the larger of the two percentages indicates systematic over- or underprediction of the methodology. A 50/50 split of these percentages would be a desirable result. The percentage of overpredicted or underpredicted values is simply equivalent to counting how many points are below or above the line of equality in Figures 24 through 28. As such, these percentages do not provide a measure of the error in prediction as does the MAE, but rather some measure of overall tendency for over- or underprediction.

In all but one case (property-damage-only accidents on 4D roadways with 52 percent underprediction), the methodology slightly overpredicts average accident frequencies, with 52 to 66 percent of sites having predicted accident frequencies to exceed their observed accident frequencies. This is to be expected based on the theoretical behavior of the NB distribution of accident frequencies. These percentages are consistent with the median ratios of observed to predicted accident frequencies discussed above.

Likelihood of Extreme Observed Accident Frequencies

Consideration of the likelihood of extreme observed accident frequencies, either high or low, is a method that directly takes into account the parameters of the NB distribution (i.e., mean and over dispersion) of accident frequencies at a particular type of roadway segment. The observed accident frequency for a given collision type (i.e., multiple-vehicle nondriveway, single-vehicle, or driveway accident) on a roadway segment is assumed to come from a distribution that has been modeled by means of an SPF (i.e., an NB regression model). This applies to total accidents, fatal and injury accidents, and PDO accidents.

Equations (45), (46), and (47) from Chapter 7 used in the methodology to predict N_{rs}, the average number of roadway segment accidents per year, can be rewritten as:

$$N_{rs} = (N_{brmv} + N_{brsv} + N_{brdwy})\, AMF_{1r} AMF_{2r} AMF_{3r}\, (1 + f_{pedr} + f_{biker})\, C_r \qquad \textbf{(80)}$$

For discussion purposes, this can be further simplified to read:

$$N_{rs} = (N_{brmv} + N_{brsv} + N_{brdwy})\, A \qquad \textbf{(81)}$$

where $$A = AMF_{1r}AMF_{2r}AMF_{3r}\,(1 + f_{pedr} + f_{biker})\,C_r \qquad \textbf{(82)}$$

Let the total number of *observed* accidents, O_i, of a given type (total, fatal and injury, or PDO) on a roadway segment, i, during the 3-year period, be represented by:

$$O_i = O_{mv,i} + O_{sv,i} + O_{dwy,i} + O_{ped,i} + O_{bike,i} \qquad \textbf{(83)}$$

where $O_{mv,i}$, $O_{sv,i}$, $O_{dwy,i}$, $O_{ped,i}$, and $O_{bike,i}$ are the observed 3-year multiple-vehicle nondriveway, single-vehicle, driveway, pedestrian, and bicycle accident frequencies, respectively, at site i.

The likelihood of observing O_i accidents is then computed under the assumption that $O_{mv,i}$, is an observation from a NB distribution with mean μ_1 and over dispersion parameter k_1; $O_{sv,i}$, is an observation from a NB distribution with mean μ_2 and over dispersion parameter k_2; and $O_{dwy,i}$ is an observation from a NB distribution with mean μ_3 and over dispersion parameter k_3. The means μ_1, μ_2, and μ_3 are simply the 3-year sums of the respective yearly SPF values calculated for the site-specific ADT and segment length. These SPFs along with their respective over dispersion parameters are presented in Tables 30, 35, and 38 for multiple-vehicle nondriveway, single-vehicle, and driveway-related collisions, respectively.

Based on Equation (83), the likelihood of observing O_i or fewer accidents at site i can then be written as:

$$p_i = \text{prob}\{\text{Total number of accidents } \le O_i\} \qquad \textbf{(84)}$$

or

$$p_i = \text{prob}\{(MV_i + SV_i + DWY_i)\,A \le O_i\} \qquad \textbf{(85)}$$

or

$$p_i = \text{prob}\{(MV_i + SV_i + DWY_i) \le B\} \qquad \textbf{(86)}$$

where $$B = \text{floor}(O_i/A)\text{, the largest integer} \le (O_i/A) \qquad \textbf{(87)}$$

Note that, in Equation (85), the coefficient A is based on the average yearly calibration factor, C_r, as shown earlier in this chapter. Expanding Equation (85) to yearly terms, each with a yearly calibration factor, would add an unnecessary level of complexity to the calculations of the probabilities, p_i.

Assuming that MV, SV, and DWY accident frequencies are independent random variables, Equation (86) can be written as:

$$p_i = \sum_{x=0}^{B} \sum_{y=0}^{B-x} \sum_{z=0}^{B-x-y} f_{MVi}(x)\, f_{SVi}(y)\, f_{DWYi}(z) \qquad \textbf{(88)}$$

where $f_{MVi}(x)$ is the probability distribution function for a NB with mean μ_1 and over dispersion parameter k_1; $f_{SVi}(y)$ and $f_{DWYi}(z)$ are defined similarly; and values of x, y, and z = 0, 1, 2, …, B. If $p_i \ge 0.5$, then p_i was calculated as $1-p_i$. This approach is equivalent to calculating the area under the cumulative distribution curve at either low or high tail of the distribution.

Two final single-value criteria to assess how well the observed accident frequencies can be estimated by the proposed methodology are then proposed:

- Mean probability, the average across all segments of a given roadway type for a particular accident type (i.e., total, FI, and PDO). On average, the mean would be close to 0.25 if the methodology is to fit the observed data reasonably well.
- The percent of sites with observed accident frequencies outside the upper and lower 2.5-percentile of the theoretical distribution. This value is calculated as:

$$Pct_{Unlikely} = 100\ [\text{Number of sites where } p_i \leq 0.025)/\text{Total number of sites}] \qquad \textbf{(89)}$$

On average, the value of $Pct_{Unlikely}$ would be close to 5 percent if the methodology is to fit the observed data reasonably well, since one would expect, on average, that 5 percent of the observed values be outside the middle 95 percent of the distribution.

Figures 33 through 35 illustrate the distribution of the individual p_i values, separately for each accident type, in the form of side-by-side box plots for each roadway type. The dot in each box indicates the mean probability for a given roadway type, while the horizontal line in each box indicates the median. The shaded box represents the middle 50 percent of the distribution; the width of each box is proportional to the number of roadway segments of a specific type. The whiskers simply connect a box to the highest and lowest p_i values. Three horizontal reference lines are drawn: one at 0.25, the expected median probability; the others at 0.125 and 0.375 to mark the 25th- and 75th-percentiles of the theoretical distribution.

The average probabilities for all roadway types for total accident frequencies are all very close to the theoretically expected value of 0.25, ranging between 0.23 and 0.28, as shown in Table 89. Furthermore, one would theoretically expect the middle 50 percent of the estimated probabilities, p_i, to be between 0.125 and 0.375 (i.e., 0.25±0.25/2). This is consistent with the information shown in the box plots in Figure 33 where the shaded box falls, for all practical purposes, between the dashed lines drawn at 0.125 and 0.375 for each roadway type. The most extreme observations, i.e., those with less than a 5-percent chance of being observed, given the prediction model used, occur less than 5.9 percent of the time as shown in the last column of Table 89.

Figure 34 and Table 89 show that the average probabilities for all roadway types except 2U, for fatal-and-injury accident frequencies are slightly above the theoretically expected value of 0.25, ranging between 0.25 and 0.31. In this case, the middle 50 percent of the estimated probabilities are slightly elevated and outside the 0.125 and 0.375 band, except for 2U roadways. Therefore, the most extreme observations occur less than 4.2 percent of the time as shown in the last column of Table 89.

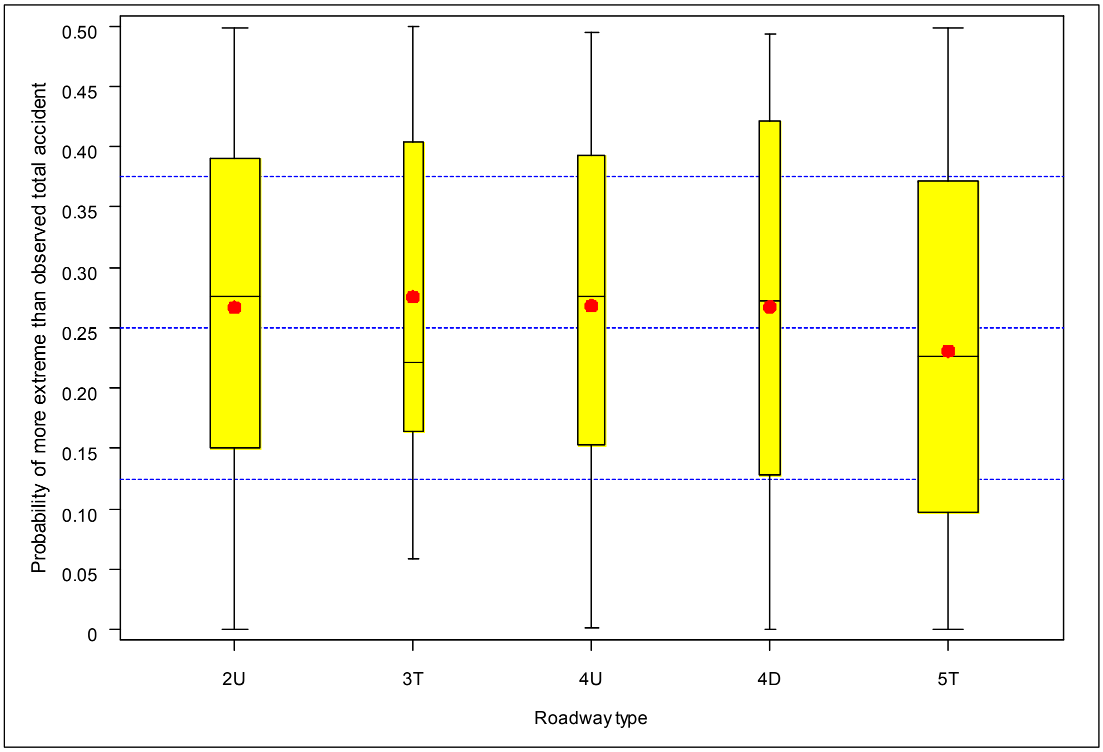

Figure 33. Distribution of probabilities of observing a more extreme total accident count than the observed count on roadway segments in Washington

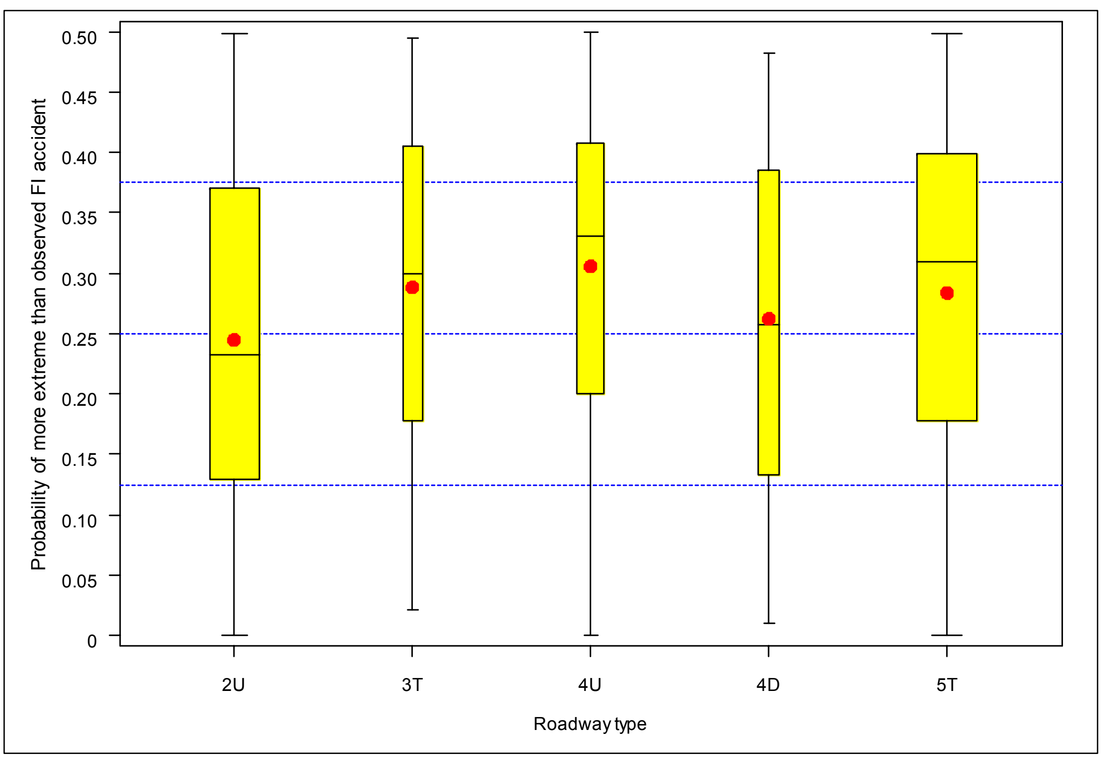

Figure 34. Distribution of probabilities of observing a more extreme FI accident count than the observed count on roadway segments in Washington

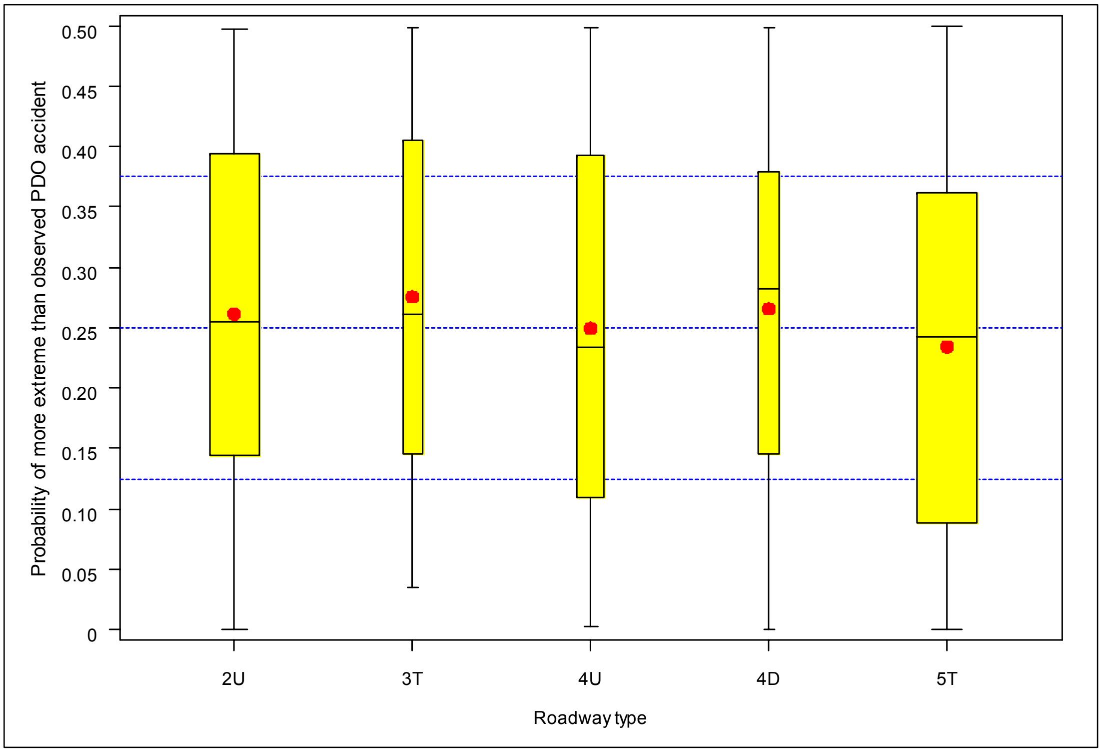

Figure 35. Distribution of probabilities of observing a more extreme PDO accident count than the observed count on roadway segments in Washington

Figure 35 and Table 89 show that the average probabilities for all roadway types for property-damage-only accident frequencies are all close to the theoretically expected value of 0.25, ranging between 0.23 and 0.28. In this case, the middle 50 percent of the estimated probabilities fall, for all practical purposes, inside the 0.125 and 0.375 band. The most extreme observations occur less than 9.3 percent of the time as shown in the last column of Table 89. These percentages are slightly elevated as compared to 5 percent for 2U, 4D, and 5T roadways, indicating that a few sites in these roadway categories experienced either unusually low or unusually high accident frequencies given the prediction model used.

For all three accident types, these results support the conclusion that the methodology has acceptable predictive capability across all roadway types considered. Specifically, the average probability of predicting a more extreme accident frequency is consistently close to the expected value of 0.25. The percent of extreme observations is generally below or only slightly above the expected value of 5 percent. The results indicate that the NB distribution was an appropriate choice for the modeling effort and that the SPF parameters estimated from the Minnesota and Michigan data combined with the HSM methodology as a whole are applicable to the validation dataset.

Summary of Roadway Segment Validation

The HSM methodology developed from the Minnesota and Michigan data shows good overall predictive performance when calibrated and applied to a new dataset from Washington State. This is evidenced by the relatively good fit of the empirical distribution of the Washington State data to the theoretical distribution derived from the Minnesota and Michigan data. On average, the prediction errors of the methodology are larger than we would like, but the relative magnitude of the prediction error appears to be strongly influenced by the magnitude of the predicted accident frequency.

For sites for which the methodology predicts relatively high accident frequencies, the predictions are quite accurate in relative terms (see the left end of the plots in Figures 31 and 32). For sites for which the methodology predicts low accident frequencies, the predictions are less accurate in relative terms. In other words, some sites with low predicted accident frequencies also have low observed accident frequencies, but other sites with low predicted accident frequencies have high observed accident frequencies.

It should be recognized that the purpose of the HSM methodology is to predict the effect of engineering factors on safety. The methodology does not incorporate the effect of driver behavioral factors, such as alcohol or safety belt usage, because the prediction of behavioral effects was not within the scope of the research and because no site-specific data on behavioral factors were available for use in the research even if the evaluation of behavioral factors had been desired.

The accuracy of the procedure for sites with high predicted accident frequencies is a valuable result, because it is precisely at sites with high accident frequencies that the accuracy of the methodology is most important. (If the opposite were true and the methodology were most accurate for sites with low accident frequencies, it would be more difficult to justify the use of the methodology.) While there is no formal way to separate the influences of engineering and behavioral factors on observed accident frequencies, the relative accuracy of the procedure for sites with high predicted accident frequencies suggests that the methodology does reasonably well in assessing the safety effects of engineering factors and that, where low accident frequencies are predicted, but high accident frequencies are observed, this may be explained by behavioral factors.

INTERSECTION VALIDATION STUDY

The HSM methodology for intersections was validated through application to a set of signalized intersections on arterials under the jurisdiction of the Florida Department of Transportation. The intersection validation dataset included 454 four-leg signalized intersections. The intersection characteristics and crash database for these sites was assembled by Abdel-Aty et al. (*136*) for the Florida Department of Transportation.

Tables 90 and 91 summarize the characteristics of the Florida intersection validation set. Florida accidents are reported in two different levels of detail. A long-form accident report is used for all fatal and injury accidents and for property-damage-only crashes that meet several established criteria. A short form is used for all other property-damage-only accidents. Since reporting levels for short-form accidents are likely to vary widely, only accidents reported on the long form were considered in the validation study. The 454 intersections experienced 3,702 accidents reported on the long form in a 2-year period including 2000 and 2001. The Florida intersections are located in Brevard, Hillsborough, Miami-Dade, Orange, and Seminole Counties.

TABLE 90. Summary of intersection accident and exposure data for Florida validation dataset

Average major-road ADT (veh/day)	Average minor-road ADT (veh/day)	Total exposure (10^6 veh-mi)[a]	Total number of accidents[a]	Average accident rate (per 10^6 veh-mi)
21,828	11,038	10,892.6	3,702	0.34

NOTE: Based on 454 four-leg signalized intersections.
[a] In 2-year period.

TABLE 91. Descriptive statistics for intersection traffic volumes in Florida validation dataset

ADT statistic (veh/day)	Major-road ADT	Minor-road ADT
Mean	21,828	11,038
Median	18,498	8,277
Standard deviation	606	413
Minimum	2,214	251
Maximum	67,521	64,392

NOTE: Based on 454 four-leg signalized intersections.

The Florida intersection data set is not as suitable for validation purposes as the Washington roadway segment dataset because only two years of crash data are available and because the crash counts for those two years are combined and cannot be separated. Thus, for the Florida intersection validation study, calibration was performed for both years of crash data combined rather than separately for each year.

Validation Procedure

The first step in applying the HSM methodology to the Florida intersection sites was to calibrate the methodology to Florida conditions. The HSM methodology was applied to each intersection and a calibration factor was computed as the ratio of the total observed accident frequency to the total predicted accident frequency for all 454 four-leg signalized intersections. The calibration factor (C_i) determined from this process for Florida intersections was 0.56. This calibration factor indicates that the average accident frequency for the Florida intersections is about half that of the Minnesota and North Carolina intersections from which the HSM methodology was derived.

The HSM methodology was then reapplied to each of the Florida intersections incorporating the calibration factor, C_i. The predicted accident frequency for a 2-year period at each intersection was then compared to the observed accident frequency for the same 2-year period. The validation results are presented below.

Validation Results

The approach used to validate the intersection prediction methodology is identical to that used for roadway segments. Figure 36 shows a plot of the predicted and observed total accident frequencies for each Florida intersection (acc/int/yr). The line of equality which would represent perfect agreement between the predicted and observed accident frequencies is also shown.

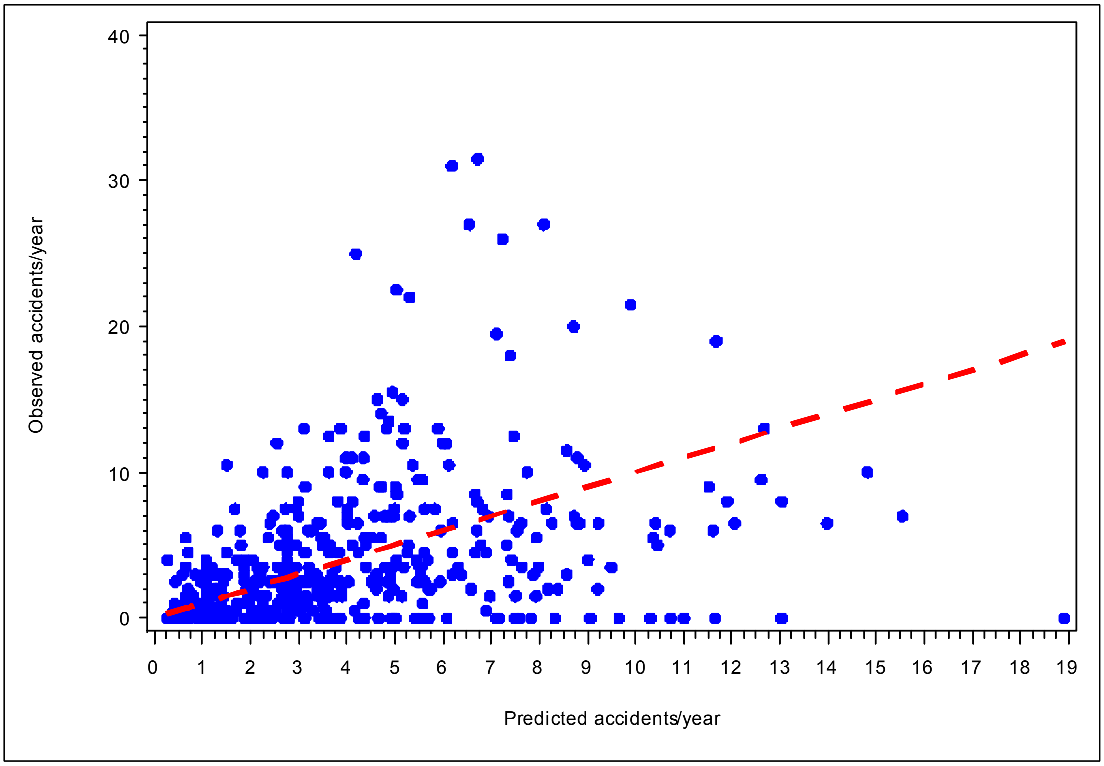

Figure 36. Comparison of total observed vs. predicted accident frequencies at four-leg signalized intersections in Florida

As in the roadway segment validation, a number of measures of the level of agreement between the predicted and observed accident frequencies were derived. Table 92 presents the values of these measures for the intersection validation study.

For total and fatal-and-injury accidents, the mean absolute error is relatively high compared to the mean accident frequency if one compares the square of MAE with the mean, as was the case for the Washington roadway segments. The methodology on average, overpredicts accident frequencies, as is expected given the nature of the data to which the NB distribution is

TABLE 92. Validation measures for Florida intersections

Accident severity level	Mean accident frequency (acc/yr)[a]	MAE (acc/yr)[b]	Ratio of observed/predicted accident frequency			Pearson correlation coefficient		Percent over-predicted	Percent under-predicted	Average probability of more extreme accident frequency	Percent of extreme observations at 5% significance level
			Minimum	Median	Maximum	Lower 95% confidence limit	Upper 95% confidence limit				
Total	4.08	3.08	0	0.73	14.40	0.27	0.43	58	42	0.18	22
Fatal and injury	2.44	2.13	0	0.78	10.71	0.13	0.30	56	44	0.18	19
Property damage only	1.63	1.32	0	0.67	20.80	0.35	0.50	63	37	0.18	25

NOTE: Based on 454 four-leg signalized intersections.
[a] Mean observed and predicted accident frequencies are equal as a result of calibration.
[b] Mean absolute error.

applied. This is supported by the percentage of overpredicted values, ranging from 56 to 63 percent across the three accident severity levels; These percentages of overpredicted values are similar to those for the Washington roadway segments.

Figure 37 presents the variation of ratio of observed to predicted accident frequency for the Florida intersections in comparison to the predicted accident frequency. As in the case of the Washington roadway segments, there is much less scatter at higher predicted accident frequencies (the right side of the plot) than at lower predicted accident frequencies (the left side of the plot). However, the points on the right side of the plot appear to be more consistently below the line representing a ratio of 1.0 (i.e., generally overpredicted) than was the case for the Washington data.

Although significantly different from zero, the correlation coefficient between predicted and observed accident frequencies are lower than for the Washington roadway segment dataset, with upper confidence limits of the coefficient ranging from 0.3 to 0.5.

The last two columns in Table 92 indicate the relatively poor predictive capability of the methodology based on this intersection dataset. This fact is further supported by the box plots of the distribution of the probability, p_i, of observing more extreme accident frequencies than those observed shown in Figure 38. It is clear that the middle 50 percent of the distribution is not contained within the 0.125 to 0.375 probability band for either accident type. The mean probability is 0.18 for all three accident type, markedly below the expected value of 0.25. Furthermore, as shown in the last column of Table 92, the most extreme observations occur between 19 and 25 percent of the time, which is substantially higher than the expected value of 5 percent.

Summary of Intersection Validation

The results of the validation study for Florida intersections are less promising than the results presented above for Washington roadway segments. While the roadway segment validation suggests that the HSM methodology provides results for Washington roadway segments that are consistent with the Minnesota and Michigan roadway segments from which the methodology was developed, the same cannot be said for the Florida intersections. The authors believe that this may arise more from the nature of the Florida dataset than from an inherent weakness of the HSM intersection methodology, but this cannot be proven from the available results. It is recommended that an alternative dataset be sought for intersection validation.

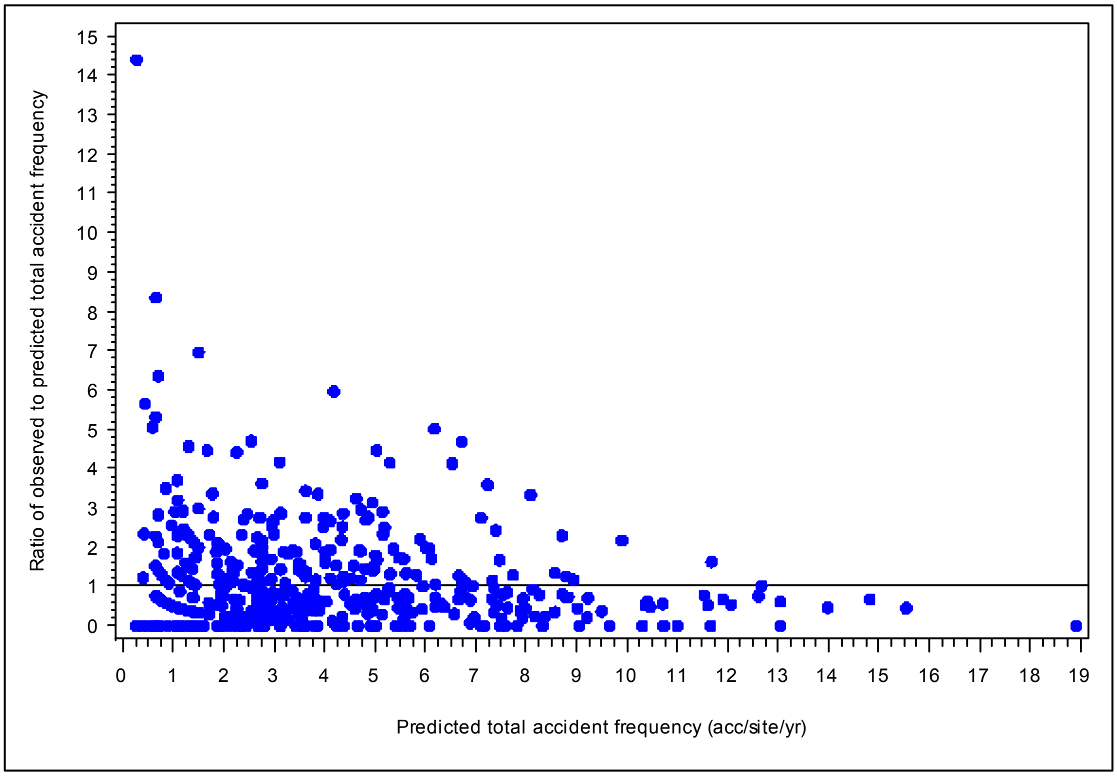

Figure 37. Variation of ratio of observed to predicted accident frequency for 4SG intersections in Florida

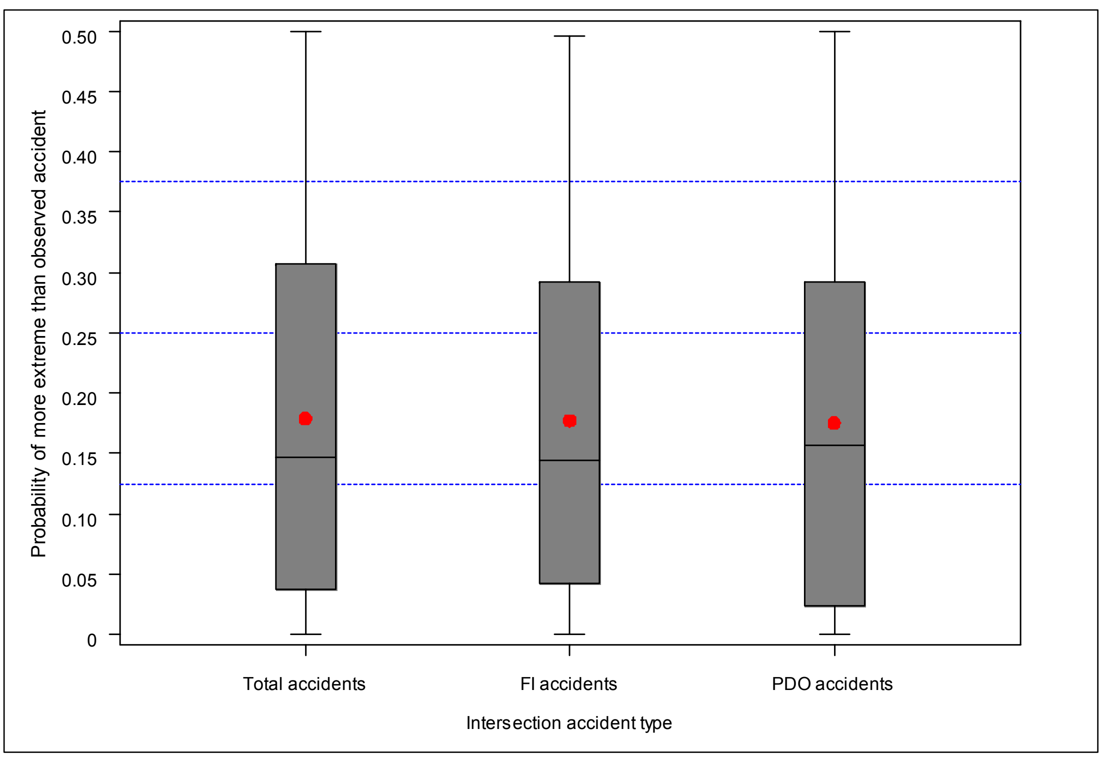

Figure 38. Distribution of probabilities of observing a more extreme accident count than the observed count at four-leg signalized intersections in Florida

CHAPTER 9.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations have been developed in this research:

1. The forthcoming *Highway Safety Manual* (HSM) can serve as a valuable tool to assist highway agencies in estimating the safety performance of specific roads. The safety effectiveness of planned projects for specific roads can be estimated by applying the HSM methodologies to the conditions before and after the project.
2. The research presented in this report has developed and demonstrated a safety prediction methodology for urban and suburban arterials suitable for incorporation in the HSM.
3. The safety prediction methodology for urban and suburban arterials uses a "building block" approach. Separate predictions of safety performance are made for the individual roadway segments and intersections that constitute a roadway or project of interest. Those separate predictions are summed over all of the segments and intersections that make up the roadway or project. For each individual roadway segment or intersection, predictions of safety performance are made for multiple-vehicle nondriveway collisions, single-vehicle collisions, driveway-related collisions, vehicle-pedestrian collisions, and vehicle-bicycle collisions and are then combined to estimate the safety performance of the roadway segment or intersection.
4. The safety prediction methodology incorporates base models that account for the effects of traffic volume levels on safety and accident modification factors (AMFs) that account for the safety effects of on-street parking, roadside fixed objects, shoulder width, and lighting for roadway segments, and left-turn lanes, left-turn signal phasing, right-turn lanes, right turn on red, red light cameras, and lighting for intersections.
5. The safety prediction methodology has been developed with data for roadway segments in Minnesota and Michigan and intersections in Minnesota and North Carolina. The methodology includes a calibration procedure to adjust the methodology to local conditions in each geographical area to which it is applied. A calibration procedure has been recommended, but further efforts are underway in NCHRP Project 17-36 to develop a common calibration procedure for all of the HSM prediction methodologies.
6. The safety prediction methodology for vehicle-pedestrian collisions presented in this report is preliminary. A more complete pedestrian safety prediction methodology is currently under development in Phase III of NCHRP Project 17-26 and will be incorporated in the HSM methodology when development and testing is complete.

7. The safety prediction methodology for roadway segments has been validated using data for arterial roadway segments in Washington. The validation results indicate that the HSM methodology is most accurate for sites where the predicted accident frequency is high. The HSM methodology provides less accurate predictions for sites with low predicted accident frequencies where factors not incorporated in the methodology, such as driver behavioral factors, may have a stronger relative influence on observed accident frequencies.

8. The safety prediction model for intersections has been validated using data for arterial intersections in Florida. The results of the validation study indicate that the intersection methodology is less accurate than the roadway segment methodology. However, this result may reflect limitations of the validation dataset rather than weakness of the safety prediction methodology. Additional validation with a different dataset would be desirable.

9. The safety prediction methodology presented in this report, with the planned improvements to the pedestrian safety prediction methodology, appears suitable for incorporation in the first edition of the HSM. Further validation of the intersection methodology may be desirable. The roadway segment and intersection methodologies in the draft of HSM Chapter 10 should be reviewed for consistency with the other methodologies planned for incorporation in the first HSM edition. The HSM will be maintained and updated over time.

10. The safety prediction models presented in this report should be incorporated into the first edition of the HSM as part of the HSM production work in NCHRP Project 17-36, with further modifications as needed.

11. The HSM needs to explain the limitations of the safety prediction methodology, including the methodology's focus on engineering factors. It should be clearly noted that the HSM prediction methodologies have not been developed to address site-by-site variations in driver behavioral factors.

12. Future research needs to improve the HSM methodology in future editions of the HSM include refining the current base models and AMFs and incorporating models or factors for the effect on safety of traffic speed on roadway segments and intersection approaches, multiple turn lanes at intersections, channelized right turns at intersections, sight distance at intersections, roundabouts, and school zones.

13. Further development of the factors for driveway safety would be desirable including the incorporation of factors to distinguish between the safety performance of driveways on divided arterials with and without median openings.

14. Future research is needed on the effect of bicycle facilities on vehicle-bicycle collisions, similar to the research that is currently underway on the effect of pedestrian facilities on vehicle-pedestrian collisions.

REFERENCES

1. Transportation Research Board, *Highway Capacity Manual*, 2000.
2. Hughes, W., K. Eccles, D. W. Harwood, I. B. Potts, and E. Hauer, *Development of a Highway Safety Manual*, NCHRP Web Document 62, Transportation Research Board, March 2004.
3. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Washington, DC, 2001.
4. Hauer, E, *Lane Width and Safety*, March 2000. Available on http://www.roadsafetyresearch.com/
5. Transportation Research Board, *Highway Capacity Manual*, 1985.
6. Transportation Research Board, *Highway Capacity Manual*, 1994.
7. Harwood, D. W., F. M. Council, E. Hauer, W. E. Hughes, and A. Vogt, *Prediction of the Expected Safety Performance of Rural Two-Lane Highways,* Federal Highway Administration, Report No. FHWA-RD-99-207, December 2000.
8. Griffin, L. I., and K. K. Mak, *The Benefits to Be Achieved From Widening Rural, Two-Lane Farm-to-Market Roads in Texas, Report No. IAC* (86-87) - 1039, Texas Transportation Institute, College Station, TX, April 1987.
9. Zegeer, C. V., D. W. Reinfurt, J. Hummer, L. Herf, and W. Hunter, "Safety Effects of Cross-Section Design for Two-Lane Roads," *Transportation Research Record* 1195, Transportation Research Board, 1988.
10. Harwood, D. W., E. R. Kohlman Rabbani, K. R. Richard, H. W. McGee, and G. L. Gittings, *Systemwide Impact of Safety and Traffic Operations Design Decisions for Resurfacing, Restoration, or Rehabilitation (RRR) Projects*, NCHRP Report 486, Transportation Research Board, 2003.
11. Hauer, E., "Safety Models for Urban Four-Lane Undivided Road Segments," presented at the 83rd annual meeting of the Transportation Research Board, Washington DC, January 2004.
12. Strathman, J. G., K. J. Dueker, J. Zhang, and T. Williams, *Analysis of Design Attributes and Crashes on the Oregon Highway System,* Oregon Department of Transportation, Center for Urban Studies, Portland State University, June 2001.
13. Hadi, M. A., J. Aruldhas, L. F. Chow, and J. A. Wattleworth, "Estimating Safety Effects of Cross-Section Design for Various Highway Types Using Negative Binomial Regression," *Transportation Research Record 1500*, Transportation Research Board, 1995.
14. Harwood, D. W., *Effective Utilization of Street Width on Urban Arterials*, NCHRP Report 330, Transportation Research Board, 1990.

15. Zegeer, C. V., H.F. Huang, J. C. Stutts, E. Rodgman, and J. E. Hummer, "Commercial Bus Accident Characteristics and Roadway Treatments," Transportation Research Record 1467, Transportation Research Board, 1994.

16. Potts, I. B., D. W. Harwood, D. J. Torbic, K. M. Bauer, D. K. Gilmore, D. K. Lay, J. F. Ringert, J. D. Zegeer, D. L. Harkey, and J. M. Barlow, *Lane Widths, Channelized Right Turns, and Right-Turn Deceleration Lanes in Urban and Suburban Areas,* Final Report of NCHRP Project 3-72, Midwest Research Institute, 2006.

17. Chu, X., and M. R. Baltes, *Pedestrian Mid-block Crossing Difficulty*, Report NCTR-392-09, National Center for Transit Research, University of South Florida, December 2001.

18. Zegeer, C. V., C. Tan Esse, J. R. Stewart, H. H. Huang, P. Lagerwey, "Safety Analysis of Marked Versus Unmarked Crosswalks in 30 Cities," *ITE Journal*, January 2004.

19. Landis, B. W., V. R. Vattikuti, R. M. Ottenberg, D. S. McLeod, and M. Guttenplan, "Modeling the Roadside Walking Environment – Pedestrian Level of Service," *Transportation Research Record 1773*, Transportation Research Board, 2001.

20. Dixon, L. B., "Bicycle and Pedestrian Level-of-Service Performance Measures and Standards for Congestion Management Systems," *Transportation Research Record 1538*, Transportation Research Board, 1996.

21. Harkey, D. L., J. R. Stewart, and E. A. Rodgman, *Evaluation of Shared-Use Facilities for Bicycles and Motor Vehicles*, University of North Carolina, Highway Safety Research Center, March 1996.

22. McHenry, S. R., and M. J. Wallace, *Evaluation of Wide Curb Lanes as Shared Lane Bicycle Facilities*, Report No. FHWA/MD-85/06, Maryland Department of Transportation, August 1985.

23. Jilla, R. J., *Effects of Bicycle Lanes on Traffic Flow*, Purdue University, School of Engineering, West Lafayette, IN, June 1974.

24. Kroll, B., and M. R. Ramey, "Effects of Bike Lanes on Driver and Bicyclist Behavior," *Transportation Engineering Journal*, American Society of Civil Engineers, March 1977.

25. Hunter, W. W., J. R. Stewart, J. C. Stutts, H. H. Huang, and W. E. Pein, *A Comparative Analysis of Bicycle Lanes Versus Wide Curb Lanes*, Report No. FHWA-RD-98-034, Federal Highway Administration, 1999.

26. Davis, J, *Bicycle Safety Evaluation*, Auburn University, City of Chattanooga and Chattanooga-Hamilton County Regional Planning Commission, Chattanooga, TN, 1987.

27. Landis, B. W., V. R. Vattikuti, and M. T. Brannick, "Real-Time Human Perceptions, Toward a Bicycle Level of Service," *Transportation Research Record 1578*, Transportation Research Board, 1997.

28. Harkey, D. L., D. W. Reinfurt, M. Knuiman, J. R. Stewart, and A. Sorton, *Development of the Bicycle Compatibility Index: A Level of Service Concept, Final Report*, Report FHWA-RD-98-072, Federal Highway Administration, December 1998.

29. Hauer, E., *Shoulder Width, Shoulder Paving, and Safety*, March 2000. Available on http://www.roadsafetyresearch.com/

30. Hauer, E., and J. Lovell, *Safety Measures Aimed at Reducing Accidents Occasioned by Vehicles Stopped on Freeway Shoulders*, Department of Civil Engineering, University of Toronto, 1984.

31. Zegeer, C. V., R. C. Deen, and J. G. Mayes, "Effect of Lane and Shoulder Width on Accident Reduction on Rural, Two-Lane Roads," *Transportation Research Record 806*, Transportation Research Board, 1981.

32. Harwood, D. W., *Multilane Design Alternatives for Improving Suburban Arterials*, NCHRP Report 282, Transportation Research Board, 1986.

33. Reinfurt, D. W., C. V. Zegeer, B. J. Shelton, and T. R. Neuman, "Analysis of Vehicle Operations on Horizontal Curves," *Transportation Research Record 1318*, Transportation Research Board, 1991.

34. Taylor, J. I., H. W. McGee, E. L. Sequin, and R. S. Hostetter, *Roadway Delineation Systems*, NCHRP Report 130, Highway Research Board, 1972.

35. Stimpson, W. A., H. W. McGee, W. K. Kittelson, and R. H. Ruddy, Field *Evaluation of Selected Delineation Treatments for Two-Lane Rural Highways*, FHWA-RD-77-118, Federal Highway Administration, 1977.

36. Datta, T. K., D. D. Perkins, J. I. Taylor, and H. T. Thompson, *Accident Surrogates for Use in Analyzing Highway Safety Hazards, Volume II: Technical Report*, Report No. FHWA-RD-82/103-105, Federal Highway Administration, 1983.

37. Terhune, K. W. and M. R. Parker, *An Evaluation of Accident Surrogates for Safety Analysis of Rural Highways*, Report No. FHWA-RD-86-127, Federal Highway Administration, 1986.

38. Zegeer, C., R. Stewart, D. Reinfurt, F. Council, T. Neuman, E. Hamilton, T. Miller, and W. Hunter, *Cost Effective Geometric Improvements for Safety Upgrading of Horizontal Curves*, Report No. FHWA-RD-90-021, Federal Highway Administration, 1990.

39. Anderson, I. B. and R. A. Krammes, "Speed Reduction as a Surrogate for Accident Experience at Horizontal Curves on Rural Two-Lane Highways," *Transportation Research Record 1701*, Transportation Research Board, 2000.

40. Fitzpatrick, K., L. Elefteriadou, D. W. Harwood, J. M. Collins, J. McFadden, I. B. Anderson, R. A. Krammes, N. Irizarry, K. D. Parma, K. M. Bauer, K. Pasetti, *Speed Prediction for Two-Lane Rural Highways*, Report No. FHWA-RE-99-171, Federal Highway Administration, 2000.

41. *Transportation Research Board Special Report 214: Designing Safety Roads: Practices for Resurfacing, Restoration, and Rehabilitation*, 1987.

42. Leisch, J. E. and J. P. Leisch, "New Concepts in Design Speed Application," *Transportation Research Record 631*, Transportation Research Board, 1977.

43. Choueiri, E. M. and R. Lamm, *Rural Roads Speed Inconsistencies Design Methods*, Part 1. NTIS Report PB88-160460, 1987.

44. Glennon, J. C., T. R. Neuman, and J. E. Leisch, *Safety and Operational Considerations for Design of Rural Highway Curves*, Report No. FHWA-RD-86-035, Federal Highway Administration, 1985.

45. Potts, I. B., D. W. Harwood, and K. M. Bauer, "Effect of Preceding Tangent Length on Safety for Horizontal Curves," conference proceedings from the Second International Symposium on Highway Geometric Design, Mainz, Germany, 2000.

46. Brindle, R. "Relationship Between Accidents and Access Conditions," *ARRB Research Report No. 320*, AARB Transport Research, Australia, 1998.

47. Colorado Department of Highways, *Final Report of the Colorado Access Control Demonstration Project*, June 1985.

48. Flora, J. W., and K. M. Keitt, *Access Management for Streets and Highways*, Report No. FHWA-IP-82-3, Federal Highway Administration, 1982.

49. Gattis, J. L., and D. Hutchison, *Comparison of Delay and Accidents on Three Roadway Access Designs in Springfield*, proceedings of the Fourth National Access Management Conference, Portland, Transportation Research Board, August 2000.

50. Levinson, H. S. and J. S. Gluck, *Access Spacing and Safety: Recent Research Results*, proceedings of the Fourth National Access Management Conference, Portland, Transportation Research Board, August 2000.

51. McMonagle, S. C., "Effect of Roadside Features in Traffic Accidents," *Traffic Quarterly*, Eno Foundation for Traffic Control, April 1952.

52. Staffield, P. R., "Accident Rates Related to Access Points and Advertising Signs—Minnesota Highway Study," *Traffic Quarterly*, Eno Foundation for Traffic Control, January 1953.

53. Schoppert, D. W., "Predicting Traffic Accidents from Roadway Elements of Rural-Two-Lane Highways with Gravel Shoulders," *Highway Research Record 158*, Highway Research Board, 1957.

54. Head, S. A., "Predicting Traffic Accidents from Roadway Elements on Urban Extensions of State Highways," *Highway Research Board Bulletin 208*, Highway Research Board, Washington, DC, 1959.

55. Cribbins, P. D., J. M. Aray, and J. K. Donaldson, "Effects of Selected Roadway and Operational Characteristics on Accidents on Multi-lane Highways," *Highway Research Record 188*, Highway Research Board, Washington, DC, 1967.

56. Mullinazzi, T. C., and H. L. Michael, "Correlation of Design Characteristics and Operational Controls with Accident Rates on Urban Arterials," Engineering Bulletin of Purdue University; proceedings, 53rd Annual Road School, March 1967.

57. Dart, O. K., and L. Mann, "Relationship of Rural Highway Geometry to Accident Rates in Louisiana," *Highway Research Record 312*, Highway Research Board, 1970.

58. Cirillo, J. A. et al., *Interstate System – Volume 1*, Bureau of Public Roads, Washington, DC, 1970.

59. McGuirk, W. M., "Evaluation of Factors Influencing Driveway Accidents," Joint Highway Research Project C-36-598, Purdue University, West Lafayette, Indiana, 1973.

60. Uckotter, D. A., "Analysis of Motor Vehicle Accidents at Commercial Driveway Locations," Purdue University, West Lafayette, Indiana, 1974.

61. Glennon, J. D., J. J. Valenta, B. B. Thorson, J. Azzeh, and C. J. Wilton, "Evaluation of Techniques for the Control of Direct Access to Arterial Highways," Report No. FHWA-RD-76-85, Federal Highway Administration, 1975.

62. Glennon, J. D., J. J. Valenta, B. B. Thorson, J. Azzeh, and C. J. Wilton, "Technical Guidelines for the Control of Direct Access to Arterial Highways, Volume I: General Framework for Implementing Access Control Techniques," Report No. FHWA-RD-76-86, Federal Highway Administration, 1975.

63. Glennon, J. D., J. J. Valenta, B. B. Thorson, J. Azzeh, and C. J. Wilton, "Volume II: Detailed Description of Access Control Techniques," Report No. FHWA-RD-76-87, Federal Highway Administration, 1975.

64. *Waushara County Access Control Plan*, East Central Wisconsin Regional Planning Commission, 1986.

65. Sokolow, G., "Access Management and Its Role in Congestion Management," Florida Department of Transportation, RA/International Conference Centre Amsterdam, The Netherlands, April 1992.

66. Long, G., C. Gan, and B. S. Morrison, *Safety Impacts of Selected Median and Design Features—Final Report*, Transportation Research Center, University of Florida, prepared in cooperation with Florida Department of Transportation, 1993.

67. Li, J., *Study of Access and Accident Relationships*, prepared for Highway Safety Branch, Ministry of Transportation and Highways, British Columbia, 1993.

68. Millard, W., "Accident Analysis Relating Crashes to Major Access Management Features—US 41," Lee County, Florida, 1993.

69. Urbitran Associates, "Improvements Options," prepared for Michigan Department of Transportation, 1994.

70. Fitzpatrick, K., and K. Balke, "An Evaluation of Flush Medians and Two-Way Left Turns on Rural Four-Lane Highways," presented at the 74th annual meeting of the Transportation Research Board, Washington, DC, 1995

71. Lall, B. K., et al., "Analysis of Traffic Accidents within the Functional Area of Intersections and Driveways," Technical Report TRANS I-95, Portland State University, 1995.

72. Urbitran Associates with I. K. Chann Associates and H. S. Levinson, *Route 7 Driveway and Access Management Plan—Final Report*, prepared for Southwest Regional Planning Agency, Norwalk Connecticut, 1996.

73. Garber, N. J. and T. E. White, *Guidelines for Commercial Driveway Spacing on Urban and Suburban Arterial Roads*, University of Virginia, 1995.

74. McLean, J., "Practical Relationship for the Assessment of Road Feature Treatments—Summary Reports," *ARRB Transport Research Report 315*, ARRB Transport Research, Australia, 1997.

75. Preston, H., R. Newton, D. Keltner, and C. Albrecht, *Statistical Relationship Between Vehicular Crashes and Highway Access*, Report No. MN/RC-1998-27, August 1998.

76. Gluck J., H. S. Levinson, and V. Stover, *Impacts of Access Management Techniques*, NCHRP Report 420, Transportation Research Board, 1999.

77. Hauer, E., *Access and Safety*, March 2000. Available on http:/www.roadsafetyresearch.com/.

78. Williams, K., *Ten Ways to Manage Roadway Access in Your Community*, proceedings of the Third National Access Management Conference, Ft. Lauderdale, Transportation Research Board, October 1998.

79. Box, P. C., *Effect of Intersections on Driveway Accidents*, proceedings of the Third National Access Management Conference, Ft. Lauderdale, Transportation Research Board, October 1998.

80. *Guidelines for Driveway Location and Design: A Recommended Practice of the ITE*, Institute of Transportation Engineers, 1987.

81. Harwood, D. W., K. M. Bauer, I. B. Potts, D. J. Torbic, K. R. Richard, E. R. Kohlman Rabbani, E. Hauer, and L. Elefteriadou. *Safety Effectiveness of Intersection Left- and Right-Turn Lanes*, Federal Highway Administration, Report No. FHWA-RD-02-089, July 2002.

82. McCoy, P. T., and M. S. Malone, "Safety Effects of Left-Turn Lanes on Urban Four-Lane Roadways," *Transportation Research Record 1239*, Transportation Research Board, 1989.

83. McFarland, W. F., et al., *Assessment of Techniques for Cost-Effectiveness of Highway Accident Countermeasures*, Report No. FHWA-RD-79-53, Federal Highway Administration, 1979.

84. Hauer, E., *The Safety of Older Persons at Intersections, Transportation in an Aging Society: Improving Mobility and Safety of Older Persons*, Vol. 2, Special Report 218, Transportation Research Board, 1988.

85. Lacy, J. D., "Traffic Operations Program to Increase Capacity and Safety," *Traffic Quarterly*, Vol. 26, No. 3, July 1972.

86. Bauer, K. M., and D. W. Harwood, *Statistical Models of At Grade Intersection Accidents*, Report No. FHWA-RD-96-125, Federal Highway Administration, 1996.

87. Bauer, K. M., and D. W. Harwood, *Statistical Models of At-Grade Intersection Accidents - Addendum.* Report No. FHWA-RD-99-094, Federal Highway Administration, 2000.

88. Poch, M., and F. L. Mannering, "Negative Binomial Analysis of Intersection Accident Frequencies," presented at the 75th Annual Meeting of the Transportation Research Board, January 1996.

89. David, N. A., and J. R. Norman, *Motor Vehicle Accidents in Relation to Geometric and Traffic Features of Highway Intersections: Vol II – Research Report*, Report No. FHWA-RD-76-129, Federal Highway Administration, 1976.

90. New Jersey Department of Transportation, *Route 130 Accident Study*, 1993.

91. Greiwe, R. R., "Intersection Management Techniques for the Left-Turning Vehicle," *ITE Journal*, June 1986.

92. Agent, K. R., "Warrants for Left-Turn Lanes," *Transportation Quarterly*, Vol. 37, No. 1, Eno Foundation for Transportation, Inc., 1983.

93. Ben-Yakov, Y., and J. Craus, *Warrants for Left-Turn Lanes on Two-Lane Roads*, Publication No. 80-28, Transportation Research Institute, Technion-Israel Institute of Technology, 1980.

94. Craus, J., and D. Mahalel, "Analysis of Operations and Safety Characteristics of Left-Turn Lanes," *ITE Journal*, July 1986.

95. Tamburri, T. N., and C. G. Hammer, *Evaluation of Minor Improvements: (Part 5) Left-Turn Channelization*, State of California, Division of Highways, Traffic Department, 1968.

96. Wilson, J. C., et al., "Simple Types of Intersection Improvements," in *Improved Street Utilization Through Traffic Engineering,* Special Report 93, Highway Research Board, 1967.

97. Maze, T. H., J. L. Henderson, and S. Sankar, *Impacts of Safety on Left-Turn Treatment at High-Speed Signalized Intersections,* Iowa Highway Research Board, Project HR-347, Iowa State University, January 1994.

98. Foody, T. J., and W. C. Richardson, *Evaluation of Left Turn Lanes as a Traffic Control Device*, Ohio Department of Transportation, 1973.

99. Harwood, D.W., M. T. Pietrucha, M.D. Wooldridge, R. E. Brydia, and K. Fitzpatrick, *Median Intersection Design*, NCHRP Report 375, Transportation Research Board, 1995.

100. McCoy, P. T., U. R. Navarro, and W. E. Witt, "Guidelines for Offsetting Opposing Left-Turn Lanes on Four-Lane Divided Roadways," *Transportation Research Record 1356*, Transportation Research Board, 1992.

101. Joshua, S. C., and A. A. Saka, "Mitigation of Sight Distance Problem for Unprotected Left-Turning Traffic at Intersections," *Transportation Research Record* 1356, Transportation Research Board, 1992.

102. Vogt, A., *Crash Models for Rural Intersections: Four-Lane by Two-Lane Stop-Controlled and Two-Lane by Two-Lane Signalized*, Report No. FHWA-RD-99-128, Federal Highway Administration, October 1999.

103. Dale, C. W, *Cost-Effectiveness of Safety Improvement Projects*, Federal Highway Administration, May 1973.

104. Dixon, K. K., J. L. Hibbard, and H. Nyman, *Right-Turn Treatment for Signalized Intersections*, Urban Street Symposium, Dallas, TX, Transportation Research Circular E-C019, Transportation Research Board, 1999.

105. Vogt, A., and J. G. Bared, *Accident Models for Two-Lane Rural Roads: Segments and Intersections*, Report No. FHWA-RD-98-133, Federal Highway Administration, 1998.

106. McCoy, P. T., R. R Bishu, J. A. Bonneson, J. W. Fitts, M. D. Fowler, S. L. Gaber, M. E. Lutjeharms, B. A. Moen, and D. L. Sicking, *Guidelines for Free Right-Turn Lanes at Unsignalized Intersections on Rural Two-Lane Highways—Final Report*, Report No. TRP-02-32-95, University of Nebraska-Lincoln, Nebraska Department of Roads, March 1995.

107. Staplin, L., D. L. Harkey, K. H. Lococo, and M. S. Tarawneh, *Intersection Geometric Design and Operational Guidelines for Older Drivers and Pedestrians, Volume I: Final Report*, Report No. FHWA-RD-96-132, Federal Highway Administration, June 1997.

108. Exnicios, H. F., "Accident Reduction Through Channelization of Complex Intersections," *Improved Street Utilization Through Traffic Engineering*, Special Report 93, Highway Research Board, 1967.

109. Rowan, N. J., and T. G. Williams, *Channelization*, Report 19-4, Texas Transportation Institute, March 1966.

110. Washington, S. P., et al., *Evaluation of High-Speed Isolated Intersections in California*, Report No. FHWA/CA/TO/91-2, Federal Highway Administration, June 1991.

111. Forrestel, R. R. *Design and Safety of Pedestrian Facilities*, Prepared by ITE Technical Council Committee 5A-5, December 1994.

112. Templer, J. A., *Provisions for Elderly and Handicapped Pedestrians, Volume 3: The Development and Evaluation of Countermeasures*, Report No. FHWA-RD-79/3, Federal Highway Administration, 1980.

113. McCoy, P. T., E. J. Tripi, and J. A. Bonneson, *Guidelines for Realignment of Skewed Intersections*, Nebraska Department of Roads Research Project Number RES1 (0099) P471, 1994.

114. Hanna, J. T., T. E. Flynn, and L. T. Webb, "Characteristics of Intersection Accidents in Rural Municipalities," *Transportation Research Record 601*, Transportation Research Board, 1976.

115. Fambro, D. B., K. Fitzpatrick, and R. Koppa, *Determination of Stopping Sight Distances*, NCHRP Report 400, Transportation Research Board, 1997.

116. Priest, R. V., "Statistical Relationships between Traffic Volume, Median Width, and Accident Frequency on Divided Highway Grade Intersections," *Highway Research News*, No. 13, Highway Research Board, June 1964.

117. Van Maren, P. A., *Correlation of Design and Control Characteristics with Accidents at Rural Multi-Lane Highway Intersections*, Purdue University and Indiana State Highway Commission, July 1980.

118. Florida Department of Transportation, *Florida Pedestrian Planning and Design Hardbook*, April 1999.

119. Pietrucha, M. T., and Opiela, K. S., "Safe Accommodation of Pedestrians at Intersections," *Transportation Research Record 1385*, Transportation Research Board, 1993.

120. Hunter, W. W., "Evaluation of Innovative Bike-Box Application in Eugene, Oregon," *Transportation Research Record 1705*, Transportation Research Board, 2000.

121. Hunter, W. W., J. R. Stewart, J. C. Stutts, H. H. Huang, and W. E. Pein, *A Comparative Analysis of Bicycle Lanes Versus Wide Curb Lanes*. Report No. FHWA-RD-98-034, Federal Highway Administration, 1999.

122. SAS Institute, Inc., *SAS/STAT 9.1 User's Guide*, Cary, NC, 2004.

123. Magee, L., "Measures Based on Wald and Likelihood Ratio Joint Significance Tests," *The American Statistician*, Vol. 44, No. 3, August 1990.

124. SAS Institute, Inc., *The GLIMMIX Procedure for SAS 9.13 Release*, Cary, NC, June 2006.

125. Hauer, E., *Observational Before-After Studies in Road Safety*, Pergamon/Elsevier Science, Inc., Tarrytown, New York, 1997.

126. Hauer, E., "Overdispersion in Modeling Accidents on Road Sections and in Empirical Bayes Estimation," *Accident Analysis and Prevention*, Vol. 33, No. 6, November 2001, pp. 799-808, 2001.

127. Miaou, S-P., and D. Lord, "Modeling Traffic Crash-Flow Relationships for Intersections: Dispersion Parameter, Functional Form, and Bayes Versus

Empirical Bayes Methods," *Transportation Research Record 1840*, Transportation Research Board, 2003.

128. Knuiman, M. W., F. M. Council, and D. W. Reinfurt, "Association of Median Width and Highway Accident Rates," *Transportation Research Record 1401*, Transportation Research Board, 1993.

129. Bonneson, J. A., K. Zimmerman, And K. Fitzpatrick, *Roadway Safety Design Synthesis*, Report No. FHWA/TX-05/0-4703--1, Texas Department of Transportation, November 2005.

130. Zegeer, C. V., and M. J. Cynecki, "Determination of Cost-Effective Roadway Treatments for Utility Pole Accidents," *Transportation Research Record 970*, Transportation Research Board, 1984.

131. Elvik, R. and T. Vaa, *The Handbook of Road Safety Measures,* Elsevier Science, 2004.

132. Hauer, E., *Left-Turn Protection, Safety, Delay and Guidelines: A Literature Review*, prepared for Federal Highway Administration, October 2004, www.roadsafetyresearch.com.

133. Lyon, C., A. Haq, B. Persaud, and S. T. Kodama, "Development of Safety Performance Functions for Signalized Intersections in a Large Urban Area and Application to Evaluation of Left-Turn Priority Treatment," presented at the *2005 Annual Meeting of the Transportation Research Board*, Washington, D.C., January 2005.

134. Clark, J. E., S. Maghsoodloo, and D. B. Brown, "Public Good Relative to Right-Turn-on-Red in South Carolina and Alabama," *Transportation Research Record 926*, 1983.

135. Bahar, G., et al., *Development of Parts I and II of the Highway Safety Manual*, Interim Report of NCHRP Project 17-27, 2005.

136. Abdel-Aty, M., C. Lee, X. Wang, P. Nawathe, J. Keller, S. Kowdla, and H. Prasad, *Identification of Intersections' Crash Profiles Patterns*, Final Report of Work Order #10, Contract No. BC-355, University of Central Florida, February 2006.

Appendix A

Survey Results

SURVEY RESULTS

This appendix presents the results of the survey of potential HSM users that was conducted as part of the research. The purpose of this survey was to obtain an assessment of potential user needs for the safety prediction methodology for urban and suburban arterials.

SURVEY QUESTIONNAIRE

Figure A-1 presents a copy of the questionnaire that was used to conduct the survey. The survey was sent to recipients by both mail and e-mail. Responses were returned by mail, e-mail, and fax.

SURVEY RECIPIENTS

The mailing list for the survey included:

- 50 state highway agencies
- 100 local highway agencies
- 100 metropolitan planning organizations (MPOs)
- 28 TRB task force members

Thus, a total of 278 survey questionnaires were mailed.

The state highway agency surveys were sent to the state safety or traffic engineer; for state highway agencies represented on the AASHTO Standing Committee on Highway Traffic Safety, the survey was sent to the committee member.

The local agency surveys were sent to traffic engineers in city or urban/suburban county agencies throughout the U.S. that were members of the ITE Urban Traffic Engineers Council; the local agencies were selected to include one to four agencies in each state depending on the size of the state.

The MPOs to whom the survey was sent were selected from the web site of the Association of Metropolitan Planning Organizations. From one to four MPOs were selected in each state, depending upon the size of the state.

The survey was also sent to the members of the TRB Task Force on Developments of the Highway Safety Manual. The task force members represent a broad range of researchers and practitioners and their views are very relevant because the task force has the responsibility to approve all HSM materials prior to publication.

SURVEY ON PREDICTION OF THE SAFETY PERFORMANCE FOR URBAN AND SUBURBAN ARTERIALS
NCHRP Project 17-26

The Transportation Research Board (TRB) and the National Cooperative Highway Research Program (NCHRP), which is managed by TRB and cosponsored by the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO), are working to create a *Highway Safety Manual* (HSM). The HSM will organize knowledge about highway safety for application by highway agencies and will include procedures to predict the quantitative safety performance of highways and streets, much as the *Highway Capacity Manual* (HCM) predicts the quantitative traffic operational performance of highways and streets.

In NCHRP Project 17-26, Midwest Research Institute (MRI) is working to develop a methodology to predict the safety performance of urban and suburban arterials that can be used in the HSM chapter on urban and suburban arterials. The purpose of this survey is to obtain your views on the scope and content of the methodology and to identify sources of data that may be useful in developing the methodology. Your response to the following questions would be appreciated.

1. What type of organization do you represent?
   ___State highway agency
   ___County highway agency
   ___City/municipal highway agency
   ___Metropolitan planning organization
   ___Other government agency
   ___Consulting firm
   ___University/research agency
   ___Other private sector organization

HIGHWAY SAFETY MANUAL **APPLICATIONS**

2. Please rate the potential usefulness to your agency for each of the following potential applications of a quantitative safety prediction procedure for urban and suburban arterials (i.e., which applications would be most useful to your agency)? [Rating 5=high priority (very useful); 1=low priority (not needed)] <u>Please do not assign the same priority to every application; your thoughtful assessment of which applications would be most useful to your agency would be very helpful.</u>

   ___ estimating the current safety performance for an existing arterial for which accident history data are not available or are not considered reliable

Figure A-1. Questionnaire Used for Survey

___ estimating the safety performance for an existing arterial combining model predictions and actual, reliable accident history data

___ forecasting the future change in safety performance that may occur on an existing facility as traffic volumes grow

___ forecasting the safety effectiveness for a proposed improvement project on an existing arterial

___ forecasting the safety performance of a new arterial that has not yet been constructed

___ forecasting the effect on safety of new driveways or new development that may be proposed along an existing arterial (i.e., for development impact studies or driveway permit requests)

CURRENT SAFETY PREDICTION METHODS

3. Does your agency use, or are you developing, any statistical models or other methods to predict or estimate the safety performance of particular urban and suburban arterials?

   ___ YES ___ NO

   If YES, please describe or attach a copy of your agency's models or methodology:

INPUT VARIABLES TO THE PREDICTION METHODOLOGY

In developing the prediction methodology, candidate input variables will be selected based on a literature review of the relationships of specific variables to safety, statistical relationships developed as part of the research, and the priorities of potential HSM users. Therefore, we would appreciate your assessments to, if you had your choice, which variables should be included in the safety prediction methodology for urban and suburban arterials in the first edition of the HSM. Please rate the following variables in terms of their potential value to your agency for inclusion in the prediction methodology for urban and suburban arterials [5=high priority; 1=low priority]. Please use the full range of ratings from 1 to 5 so that your ratings are useful in setting overall priorities; if your assessment is that a particular input variable is not needed in the first edition HSM, please use a rating of 1. In establishing the ratings, please do *not* consider the availability of data in your agency's databases.

Figure A-1. Questionnaire Used for Survey (Continued)

Candidate Input Variables for Roadway Segments Between Intersections

______ bicycle facilities
____ bicycle volumes
____ delineation
____ design or posted speed
____ grades
____ horizontal curves
____ lane widths
____ lighting
____ median type
____ median width
____ number and type of median openings
____ number and type of driveways
____ number of through lanes
____ older drivers/driver population characteristics
____ one-way vs. two-way operation
____ pavement friction
____ pedestrian facilities
____ pedestrian volumes
____ presence of curb parking
____ presence of frontage roads
____ presence of reversible lanes
____ roadside design/clear zones/roadside objects
____ segment length
____ shoulder width/curb type
____ spacing between driveways
____ spacing between signals
____ speed variance
____ traffic volume (AADT) (veh/day)
____ traffic volume in peak period (veh/hr)
____ traffic volumes for individual driveways
____ transit facilities
____ vehicle mix (e.g., percent trucks)
____ vehicle speed in peak periods
____ vehicle speed in off-peak periods
____ vertical curves

Candidate Input Variables for At-Grade Intersections

____ approach speed in peak periods
____ approach speed in off-peak periods
____ bicycle facilities
____ bicycle volumes
____ curb parking on intersection approaches

Figure A-1. Questionnaire Used for Survey (Continued)

- ____ grade of intersection approaches
- ____ horizontal alignment of intersection approaches
- ____ intersection sight distance
- ____ intersection skew angle
- ____ lanes widths on intersection approaches
- ____ lighting
- ____ level of service (LOS)
- ____ median type/presence of median
- ____ number of intersection legs
- ____ number of through lanes on intersection approaches
- ____ number and length of added through lanes at intersections
- ____ older drivers/driver population
- ____ pedestrian facilities
- ____ pedestrian volumes
- ____ presence/number of left-turn lanes
- ____ presence of median refuge area for pedestrians
- ____ presence of right-turn lanes
- ____ shoulder/curb type on intersection approaches
- ____ shoulder/curb width on intersection approaches
- ____ signal phasing (e.g., left-turn phasing)
- ____ signal timing
- ____ signal visibility
- ____ spacing between intersection and nearby driveways
- ____ type of traffic control
- ____ traffic volumes (AADTs) for major- and minor-road legs (veh/day)
- ____ traffic volumes in peak period for major- and minor-road legs (veh/hr)
- ____ transit facilities
- ____ type of left-turn channelization (painted vs. raised curb)
- ____ vehicle mix (e.g., percent trucks)

4. Are there other potential input variables, not listed above, that you think should have a high priority for inclusion in the safety prediction methodology for urban and suburban arterials?

SAFETY MEASURES OF EFFECTIVENESS TO BE PREDICTED

5. The safety measures of effectiveness to be estimated by the safety prediction methodologies in the first edition of the *Highway Safety Manual* are:

- annual accident or crash frequency for a roadway segment or intersection

Figure A-1. Questionnaire Used for Survey (Continued)

- crash severity distribution (percentage of crashes by severity level)
- crash type distribution (percentage of crashes by collision type)

Are there any additional measures of safety effectiveness that you currently use or that you recommend for estimation by prediction models in the HSM?

DATA AVAILABILITY

The data availability questions are directed specifically to representatives of public agencies that operate and maintain roadways. Respondents from other types of organizations may skip Questions 7 through 9.

6. For the urban and suburban arterials under your agency's jurisdiction, does your agency have computerized files of:

   Accident data (i.e., records for each individual crash) ____YES ____NO
   Roadway segment inventory data (i.e., geometrics and traffic control for roadway segments between intersections) ____YES ____NO
   Intersection inventory (i.e., geometrics and and traffic control for each individual intersection) ____YES ____NO

7. In your agency's computerized accident data, can driveway-related crashes be distinguished from other non-intersection crashes? ____YES ____NO

8. Do your agency's computerized data files use a common location reference system to allow direct linking of :

   Individual accident records to the inventory data for the roadway segment on which the accident occurred ____YES ____NO
   Individual accident records to the inventory data for the intersection at which the accident or the intersection to which the accident is related ____YES ____NO

Figure A-1. Questionnaire Used for Survey (Continued)

CONTACT PERSON

9. May we have the name of an individual in your agency that we can contact for further information, if necessary?

Name ________________________________
Title ________________________________
Agency ______________________________
Address _____________________________
____
____
Phone _____________ Fax _____________ e-mail _______________

THANK YOU FOR YOUR ASSISTANCE. Please send the completed questionnaire by September 30, 2003, to:

Mr. Douglas W. Harwood
Principal Traffic Engineer
Midwest Research Institute
425 Volker Boulevard
Kansas City, MO 64110
Phone: (816) 753-7600, x 1571 Fax: (816) 561-6557
e-mail: 0Hdharwood@mriresearch.org

Figure A-1. Questionnaire Used for Survey (Continued)

RESPONSE RATE

Tables A-1 and A-2 summarize the number of survey responses received from various agencies. Of the 278 surveys that were mailed out, 109 responses have been received, for an overall response rate of 39 percent. The highest response rate was received from state highway agencies (74 percent), followed by TRB task force members (68 percent), MPOs (33 percent), and local agencies (20 percent). Overall, the responses received include representatives from 41 state highway agencies, 23 local highway agencies, 33 MPOs, three other government agencies, five consulting firms, and four university/research agencies.

HIGHWAY SAFETY MANUAL APPLICATIONS

In Question 2, were asked to rate the usefulness of six potential applications of a quantitative safety prediction procedure for urban and suburban arterials. Table A-3 summarizes the responses received to this question. Each group of respondents indicated that forecasting the safety effectiveness for a proposed improvement project on an existing arterial was the most important application. The other two applications of the HSM methodology that were most frequently cited by users were forecasting the future change in safety performance that may occur on an existing facility as traffic volumes grow and forecasting the effect on safety of new driveways or new development that may be proposed along an existing arterial (i.e., for development impact studies or driveway permit requests).

CURRENT SAFETY PREDICTION METHODS

In Question 3, agencies were asked whether they were currently using, or developing, any methods to predict or estimate the safety performance of urban and suburban arterials. Table A-4 summarizes the responses to this question. Only 4 percent of responding agencies indicated that they currently employ any sort of safety prediction methodology. Follow-up contacts are being made with those agencies to determine the specific types of prediction methodologies being used. However, the general lack of current safety prediction methodologies indicates definite need for the HSM.

INPUT VARIABLES TO THE SAFETY PREDICTION METHODOLOGY

In Question 4, agencies were given the opportunity to rank two sets of input variables according to perceived importance of those variables in the safety prediction methodology for urban and suburban arterials. The ranking included a variable set for roadway segments and a variable set for intersections.

Table A-1. Response Rate for the Survey

Agency/organization type	Number of questionnaires mailed	Number of responses received	Response rate (%)
State highway agencies	50	37	74.0
Local highway agencies	100	20	20.0
MPOs	100	33	33.0
TRB Task Force	28	19	67.9
TOTAL	278	109	39.2

Table A-2. Types of Organizations Responding for the Survey

Agency/organization type	Number (percentage) of responses
State highway agency	41 (37.6)
Local highway agency	23 (21.1)
MPO	33 (30.3)
Other Government Agency	3 (2.7)
Consulting firm	5 (4.6)
University/research agency	4 (3.7)
TOTAL	109

NOTE: Number of responses for some agency/organization types exceeds the value shown in Table A-1 because some TRB task force members also represent that agency/organization type.

Table A-3. Assessment of Priority Ratings for Specific HSM Applications to Urban and Suburban Arterials

Application type	State highway agencies			Local highway agencies			MPOs			TRB Task Force		
	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max
Estimating the current safety performance for an existing arterial for which accident history data are not available or are not considered reliable	2.8	1	5	2.4	1	5	3.6	1	5	2.6	1	5
Estimating the safety performance for an existing arterial combining model predictions and actual, reliable accident history data	3.7	1	5	3.6	1	5	3.6	2	5	2.9	1	5
Forecasting the future change in safety performance that may occur on an existing facility as traffic volumes grow	3.7	1	5	3.6	1	5	4.0	1	5	4.0	2	5
Forecasting the safety effectiveness for a proposed improvement project on an existing arterial	4.5	3	5	4.5	3	5	4.1	1	5	4.6	3	5
Forecasting the safety performance of a new arterial that has not yet been constructed	3.1	1	5	2.8	1	5	2.9	1	5	3.4	1	5
Forecasting the effect on safety of new driveways or new development that may be proposed along an existing arterial (i.e., for development impact studies or driveway permit requests)	3.8	1	5	4.2	1	5	3.3	1	5	4.1	2	5

NOTE: Priority ratings are on a scale from 1 to 5 (1 = lowest priority; 5 = highest priority).

Table A-4. Use of Existing Safety Prediction Methods by Highway Agencies and MPOs

Agency/organization type	Number (percentage) of agencies with safety prediction methodologies used or under development	
	Yes	No
State highway agencies	2 (5.4)	35 (94.6)
Local highway agencies	1 (5.0)	19 (95.0)
MPOs	1 (3.0)	32 (97.0)
TOTAL	4 (4.4)	86 (95.6)

Table A-5 presents the priority ranking of these input variable sets by state highway agencies. The potential input variables are arranged in descending order of priority ranking by the respondents; thus, the highest ranked variables are presented first. The highest ranked input variables by state highway agencies for roadway segments include:

- Traffic volume (AADT) (veh/day)
- Design or posted speed
- Lane widths
- Number of through lanes4
- Number and type of driveways

The highest ranked input variables by state highway agencies for intersections include:

- Intersection sight distance
- Type of traffic control
- Traffic volumes (AADT) for major- and minor-road legs (veh/day)
- Intersection skew angle
- Number of intersection legs

Tables A-6, A-7, and A-8 present comparable rankings of potential input variables for local highway agencies, MPOs, and TRB task force members, respectively.

Table A-9 presents a summary of the ranking of the potential input variables by all four types of respondents.

In Question 5, agencies were also asked to identify additional variables, not listed in the survey, that they felt should be included into the safety prediction methodology for urban and suburban arterials. Table A-10 summarizes the additional candidate input variables identified by surveyed agencies. The table shows the number of respondents that mentioned each variable and the percentage of total survey respondents that they represent. It should be noted that even those additional input variables mentioned most frequently were cited by less than 5 percent of the surveyed respondents. Some of the additional variables identified in Table A-10 were, in fact, included in the survey questionnaire, but in a different form.

SAFETY MEASURES OF EFFECTIVENESS TO BE PREDICTED

The safety measures of effectiveness to be estimated by the safety prediction methodologies in the first edition of the HSM are:

- annual accident or crash frequency for a roadway segment or intersection
- crash severity distribution (percentage of crashes by severity level)
- crash type distribution (percentage of crashes by collision type)

Table A-5. Candidate Input Variables in Descending Priority Order as Ranked by State Highway Agencies

Candidate input variable	Priority rating			Rank order
	Avg	Min	Max	
ROADWAY SEGMENTS				
Traffic volume (AADT) (veh/day)	4.5	2	5	1
Design or posted speed	4.3	3	5	2
Lane widths	4.2	2	5	3
Number of through lanes	4.2	1	5	3
Number and type of driveways	4.1	2	5	5
Traffic volume in peak period (veh/h)	4.1	2	5	5
Horizontal curves	3.9	1	5	7
Number and type of median openings	3.9	2	5	7
Roadside design/clear zones/roadside objects	3.9	1	5	7
Median width	3.8	2	5	10
Shoulder width/curb type	3.8	2	5	10
Spacing between signals	3.8	2	5	10
Vertical curves	3.7	2	5	13
Grades	3.6	1	5	14
Segment length	3.6	1	5	14
Lighting	3.5	1	5	16
Median type	3.5	1	5	16
Spacing between driveways	3.5	2	5	16
Speed variance	3.4	1	5	19
Vehicle mix (e.g., percent trucks)	3.4	1	5	19
Vehicle speed in off-peak periods	3.4	2	5	19
Vehicle speed in peak periods	3.4	2	5	19
One-way vs. two-way operation	3.3	1	5	23
Presence of curb parking	3.3	1	5	23
Delineation	2.9	1	5	25
Presence of frontage roads	2.9	1	5	25
Pedestrian volumes	2.8	1	5	27
Pedestrian facilities	2.7	1	5	28
Presence of reversible lanes	2.7	1	5	28
Traffic volumes for individual driveways	2.7	1	5	28
Pavement friction	2.6	1	5	31
Older drivers/driver population characteristics	2.4	1	4	32
Bicycle volumes	2.2	1	4	33
Bicycle facilities	2.1	1	4	34
Transit facilities	2.1	1	5	34

Table A-5. Candidate Input Variables in Descending Priority Order as Ranked by State Highway Agencies (Continued)

Candidate input variable	Priority rating			Rank order
	Avg	Min	Max	
INTERSECTIONS				
Intersection sight distance	4.5	2	5	1
Type of traffic control	4.5	3	5	1
Traffic volumes (AADT) for major- and minor-road legs (veh/day)	4.4	2	5	3
Intersection skew angle	4.2	2	5	4
Number of intersection legs	4.2	3	5	4
Presence/number of left-turn lanes	4.2	1	5	4
Signal phasing (e.g., left-turn phasing)	4.2	1	5	4
Traffic volumes in peak period (veh/h)	4.2	2	5	4
Lane widths on approaches	4.1	2	5	9
Number of through lanes on approaches	4.1	2	5	9
Horizontal alignment of approaches	4.0	2	5	11
Presence of right-turn lanes	4.0	2	5	11
Number and length of added through lanes	3.9	3	5	13
Signal visibility	3.9	1	5	13
Signal timing	3.8	1	5	15
Spacing between intersections and nearby driveways	3.8	2	5	15
Level of Service (LOS)	3.7	1	5	17
Approach speed in peak periods	3.6	2	5	18
Lighting	3.6	2	5	18
Median type/presence of median	3.6	2	5	18
Approach speed in off-peak periods	3.5	2	5	21
Curb parking on approaches	3.5	1	5	21
Grade of approaches	3.5	1	5	21
Presence of median refuge area for pedestrians	3.4	1	5	24
Vehicle mix (e.g., percent trucks)	3.2	1	5	25
Shoulder/curb width on approaches	3.1	1	5	26
Pedestrian volumes	3.0	1	5	27
Pedestrian facilities	2.8	1	5	28
Shoulder/curb type on approaches	2.8	1	4	28
Type of left-turn channelization (painted vs. curb)	2.8	1	5	28
Bicycle facilities	2.4	1	4	31
Bicycle volumes	2.4	1	4	31
Older drivers/driver population characteristics	2.4	1	4	31
Transit facilities	2.1	1	4	34

Table A-6. Candidate Input Variables in Descending Priority Order as Ranked by Local Highway Agencies

Candidate input variable	Priority rating			Rank order
	Avg	Min	Max	
ROADWAY SEGMENTS				
Traffic volume in peak period (veh/h)	4.5	3	5	1
Traffic volume (AADT) (veh/day)	4.2	3	5	2
Horizontal curves	3.9	1	5	3
Number and type of driveways	3.9	2	5	3
Design or posted speed	3.8	2	5	5
Lane widths	3.7	1	5	6
Number and type of median openings	3.7	1	5	6
Vertical curves	3.7	1	5	6
Spacing between signals	3.6	1	5	9
Spacing between driveways	3.5	2	5	10
Grades	3.4	1	5	11
Number of through lanes	3.4	1	5	11
One-way vs. two-way operation	3.4	1	5	11
Delineation	3.3	1	5	14
Median type	3.3	1	5	14
Vehicle speed in peak periods	3.3	2	5	14
Lighting	3.2	1	4	17
Presence of curb parking	3.2	1	5	17
Vehicle mix (e.g., percent trucks)	3.1	1	4	19
Vehicle speed in off-peak periods	3.0	1	5	20
Pedestrian volumes	2.9	1	5	21
Roadside design/clear zones/roadside objects	2.9	1	4	21
Median width	2.8	1	5	23
Pedestrian facilities	2.8	1	5	23
Speed variance	2.8	1	5	23
Traffic volumes for individual driveways	2.8	1	5	23
Bicycle facilities	2.7	1	5	27
Shoulder width/curb type	2.7	1	5	27
Presence of frontage roads	2.6	1	5	29
Presence of reversible lanes	2.6	1	5	29
Transit facilities	2.6	1	4	29
Segment length	2.4	1	5	32
Bicycle volumes	2.3	1	5	33
Pavement friction	2.3	1	4	33
Older drivers/driver population characteristics	1.9	1	3	35

Table A-6. Candidate Input Variables in Descending Priority Order as Ranked by Local Highway Agencies (Continued)

Candidate input variable	Priority rating			Rank order
	Avg	Min	Max	
INTERSECTIONS				
Signal phasing (e.g., left-turn phasing)	4.5	2	5	1
Traffic volumes for peak period (veh/h)	4.4	3	5	2
Intersection sight distance	4.3	2	5	3
Presence/number of left-turn lanes	4.2	3	5	4
Signal timing	4.2	2	5	4
Signal visibility	4.2	2	5	4
Spacing between intersections and nearby driveways	4.2	3	5	4
Type of traffic control	4.2	3	5	4
Traffic volumes (AADT) for major- and minor-road legs (veh/day)	3.9	2	5	9
Horizontal alignment of approaches	3.8	2	5	10
Number of intersection legs	3.8	2	5	10
Intersection of skew angle	3.7	2	5	12
Median type/presence of median	3.7	1	5	12
Level of Service (LOS)	3.6	1	5	14
Number and length of added through lanes	3.6	2	5	14
Number of through lanes on approaches	3.6	2	5	14
Presence of right-turn lanes	3.6	2	5	14
Lane widths on approaches	3.5	1	5	18
Approach speed in peak periods	3.4	2	5	19
Curb parking on approaches	3.4	1	5	19
Grade of approaches	3.3	1	5	21
Lighting	3.3	2	5	21
Approach speed in off-peak periods	3.1	2	4	23
Type of left-turn channelization (painted vs. curb)	3.1	1	5	23
Presence of median refuge area for pedestrians	2.9	1	5	25
Pedestrian facilities	2.8	1	5	26
Pedestrian volumes	2.8	1	4	26
Shoulder/curb width on approaches	2.7	1	5	28
Shoulder/curb type on approaches	2.6	1	5	29
Transit facilities	2.6	1	4	29
Vehicle mix (e.g., percent trucks)	2.6	1	4	29
Bicycle facilities	2.5	1	4	32
Bicycle volumes	2.3	1	4	33
Older drivers/driver population characteristics	1.7	1	3	34

Table A-7. Candidate Input Variables in Descending Priority Order as Ranked by Metropolitan Planning Organizations

Candidate input variable	Priority rating			Rank order
	Avg	Min	Max	
ROADWAY SEGMENTS				
Traffic volume in peak period (veh/day)	4.4	2	5	1
Design or posted speed	4.2	1	5	2
Lane widths	4.2	2	5	2
Traffic volume (AADT) (veh/day)	4.2	2	5	2
Number of through lanes	3.8	1	5	5
Spacing between signals	3.7	1	5	6
Vehicle speed in peak periods	3.7	1	5	6
Number and type of driveways	3.6	2	5	8
Shoulder width/curb type	3.6	1	5	8
Spacing between driveways	3.6	1	5	8
Speed variance	3.6	1	5	8
One-way vs. two-way operation	3.5	1	5	12
Vehicle speed in off-peak periods	3.5	1	5	12
Horizontal curves	3.4	1	5	14
Lighting	3.4	1	5	14
Median type	3.4	1	5	14
Vehicle mix (e.g., percent trucks)	3.4	1	5	14
Pedestrian facilities	3.3	1	5	18
Presence of curb parking	3.3	1	5	18
Vehicle curves	3.2	1	5	20
Grades	3.1	1	5	21
Number and type of median openings	3.1	1	5	21
Transit facilities	3.1	1	5	21
Delineation	3.0	1	5	24
Pedestrian volumes	3.0	1	5	24
Bicycle facilities	2.8	1	5	26
Older drivers/driver population characteristics	2.8	1	5	26
Roadside design/clear zones/roadside objects	2.8	1	5	26
Segment length	2.8	1	5	26
Traffic volumes for individual driveways	2.8	1	5	26
Median width	2.7	1	5	31
Bicycle volumes	2.6	1	5	32
Presence of frontage roads	2.4	1	5	33
Presence of reversible lanes	2.3	1	5	34
Pavement friction	2.1	1	4	35

Table A-7. Candidate Input Variables in Descending Priority Order as Ranked by Metropolitan Planning Organizations (Continued)

Candidate input variable	Priority rating			Rank order
	Avg	Min	Max	
INTERSECTIONS				
Traffic volumes in peak period (veh/h)	4.3	2	5	1
Level of Service (LOS)	4.2	1	5	2
Presence/number of left-turn lanes	4.2	1	5	2
Type of traffic control	4.2	2	5	2
Traffic volumes (AADT) for major- and minor-road legs (veh/day)	4.1	2	5	5
Intersection sight distance	4.0	1	5	6
Number of through lanes on approaches	3.9	1	5	7
Signal phasing (e.g., left-turn phasing)	3.9	1	5	7
Signal timing	3.9	1	5	7
Lane widths on intersection approaches	3.8	1	5	10
Presence of right-turn lanes	3.8	1	5	10
Approach speed in peak periods	3.7	1	5	12
Number of intersection legs	3.7	1	5	12
Signal visibility	3.7	1	5	12
Spacing between intersections and nearby driveways	3.7	1	5	12
Number and length of added through lanes	3.6	1	5	16
Presence of median refuge area for pedestrians	3.6	1	5	16
Approach speed in off-peak periods	3.4	1	5	18
Curb parking on intersection approaches	3.4	1	5	18
Pedestrian facilities	3.4	1	5	18
Horizontal alignment of intersection approaches	3.3	1	5	21
Intersection skew angle	3.3	1	5	21
Lighting	3.3	1	5	21
Grade of intersection approaches	3.2	1	5	24
Median type/presence of median	3.2	1	5	24
Vehicle mix (e.g., percent trucks)	3.2	1	5	24
Pedestrian volumes	3.1	1	5	27
Transit facilities	3.1	1	5	27
Bicycle facilities	3.0	1	5	29
Type of left-turn channelization (painted vs. curb)	2.9	1	5	30
Shoulder/curb width on approaches	2.8	1	5	31
Shoulder/curb type on approaches	2.7	1	5	32
Older drivers/driver population characteristics	2.7	1	5	32
Bicycle volumes	2.6	1	5	34

Table A-8. Candidate Input Variables in Descending Priority Order as Ranked by TRB Task Force Members

Candidate input variable	Priority rating			Rank order
	Avg	Min	Max	
ROADWAY SEGMENTS				
Traffic volume (AADT) (veh/day)	4.4	1	5	1
Number of through lanes	4.3	2	5	2
Number and type of driveways	4.2	1	5	3
Design or posted speed	4.1	1	5	4
Lane widths	4.1	1	5	4
Traffic volume in peak period (veh/day)	4.1	2	5	4
Number and type of median openings	3.9	2	5	7
Spacing between driveways	3.9	1	5	7
Horizontal curves	3.8	2	5	9
Shoulder width/curb type	3.8	1	5	9
Lighting	3.7	1	5	11
Median type	3.7	1	5	11
Median width	3.7	1	5	11
Presence of curb parking	3.7	1	5	11
One-way vs. two-way operation	3.6	1	5	15
Segment length	3.6	1	5	15
Spacing between signals	3.6	1	5	15
Roadside design/clear zones/roadside objects	3.5	1	5	18
Grades	3.4	2	5	19
Speed variance	3.3	1	5	20
Vertical curves	3.3	1	5	20
Vehicle mix (e.g., percent trucks)	3.2	1	5	22
Vehicle speed in peak periods	3.2	1	4	22
Pedestrian volumes	3.1	1	5	24
Vehicle speed in off-peak periods	3.1	1	5	24
Pedestrian facilities	2.9	1	5	26
Presence of frontage roads	2.7	1	5	27
Transit facilities	2.7	1	5	27
Delineation	2.6	1	5	29
Traffic volumes for individual driveways	2.6	1	5	29
Pavement friction	2.5	1	4	31
Presence of reversible lanes	2.4	1	5	32
Older drivers/driver population characteristics	2.3	1	5	33
Bicycle volumes	2.2	1	5	34

Table A-8. Candidate Input Variables in Descending Priority Order as Ranked by TRB Task Force Members (Continued)

Candidate input variable	Priority rating Avg	Priority rating Min	Priority rating Max	Rank order
Bicycle facilities	2.1	1	4	35
INTERSECTIONS				
Presence/number of left-turn lanes	4.7	2	5	1
Intersection sight distance	4.4	1	5	2
Number of intersection legs	4.4	3	5	2
Type of traffic control	4.4	1	5	2
Traffic volumes (AADT) for major- and minor-road legs (veh/day)	4.4	2	5	2
Presence of right-turn lanes	4.3	2	5	6
Signal phasing (e.g., left-turn phasing)	4.3	1	5	6
Horizontal alignment of intersection approaches	4.2	2	5	8
Intersection skew angle	4.2	2	5	8
Number of through lanes on approaches	4.2	2	5	8
Number and length of added through lanes	4.1	2	5	11
Traffic volumes in peak period (veh/h)	4.1	2	5	11
Signal timing	3.9	2	5	13
Spacing between intersections and nearby driveways	3.9	2	5	13
Lane widths on intersection approaches	3.8	2	5	15
Lighting	3.8	1	5	15
Signal visibility	3.7	1	5	17
Level of Service (LOS)	3.6	1	5	18
Median type/presence of median	3.6	2	5	18
Curb parking on intersection approaches	3.5	1	5	20
Grade of intersection approaches	3.5	2	5	20
Vehicle mix (e.g., percent trucks)	3.4	1	5	22
Approach speed in peak periods	3.3	1	5	23
Pedestrian facilities	3.3	1	5	23
Pedestrian volumes	3.3	1	5	23
Presence of median refuge area for pedestrians	3.3	1	5	23
Type of left-turn channelization (painted vs. curb)	3.2	1	5	27
Approach speed in off-peak periods	3.1	1	5	28
Shoulder/curb type on approaches	3.1	1	5	28
Shoulder/curb width on approaches	3.1	1	5	28

Table A-8. Candidate Input Variables in Descending Priority Order as Ranked by TRB Task Force Members (Continued)

Candidate input variable	Priority rating			Rank order
	Avg	Min	Max	
Transit facilities	2.6	1	4	31
Older drivers/driver population characteristics	2.5	1	5	32
Bicycle facilities	2.4	1	5	33
Bicycle volumes	2.3	1	4	34

Table A-9. Summary of Priority Ratings of Candidate Input Variables

Candidate input variable	Average priority rating				Rank order			
	State	Local	MPO	Task force	State	Local	MPO	Task force
ROADWAY SEGMENTS								
Bicycle facilities	2.1	2.7	2.8	2.1	34	27	26	35
Bicycle volumes	2.2	2.3	2.6	2.2	33	33	32	34
Delineation	2.9	3.3	3.0	2.6	25	14	24	29
Design or posted speed	4.3	3.8	4.2	4.1	2	5	2	4
Grades	3.6	3.4	3.1	3.4	14	11	21	19
Horizontal curves	3.9	3.9	3.4	3.8	7	3	14	9
Lane widths	4.2	3.7	4.2	4.1	3	6	2	4
Lighting	3.5	3.2	3.4	3.7	16	17	14	11
Median type	3.5	3.3	3.4	3.7	16	14	14	11
Median width	3.8	2.8	2.7	3.7	10	23	31	11
Number and type of median openings	3.9	3.7	3.1	3.9	7	6	21	7
Number and type of driveways	4.1	3.9	3.6	4.2	5	3	8	3
Number of through lanes	4.2	3.4	3.8	4.3	3	11	5	2
Older drivers/driver population characteristics	2.4	1.9	2.8	2.3	32	35	26	33
One-way vs. two-way operation	3.3	3.4	3.5	3.6	23	11	12	15
Pavement friction	2.6	2.3	2.1	2.5	31	33	35	31
Pedestrian facilities	2.7	2.8	3.3	2.9	28	23	18	26
Pedestrian volumes	2.8	2.9	3.0	3.1	27	21	24	24
Presence of curb parking	3.3	3.2	3.3	3.7	23	17	18	11
Presence of frontage roads	2.9	2.6	2.4	2.7	25	29	33	27
Presence of reversible lanes	2.7	2.6	2.3	2.4	28	29	34	32
Roadside design/clear zones/ roadside objects	3.9	2.9	2.8	3.5	7	21	26	18
Segment length	3.6	2.4	2.8	3.6	14	32	26	15
Shoulder width/curb type	3.8	2.7	3.6	3.8	10	27	8	9
Spacing between driveways	3.5	3.5	3.6	3.9	16	10	8	7

Table A-9. Summary of Priority Ratings of Candidate Input Variables (Continued)

Candidate input variable	Average priority rating				Rank order			
	State	Local	MPO	Task force	State	Local	MPO	Task force
ROADWAY SEGMENTS (Continued)								
Spacing between signals	3.8	3.6	3.7	3.6	10	9	6	15
Speed variance	3.4	2.8	3.6	3.3	19	23	8	20
Traffic volume (AADT) (veh/day)	4.5	4.2	4.2	4.4	1	2	2	1
Traffic volume in peak period (veh/h)	4.1	4.5	4.4	4.1	5	1	1	4
Traffic volumes for individual driveways	2.7	2.8	2.8	2.6	28	23	26	29
Transit facilities	2.1	2.6	3.1	2.7	34	29	21	27
Vehicle mix (e.g., percent trucks)	3.4	3.1	3.4	3.2	19	19	14	22
Vehicle speed in peak period	3.4	3.3	3.7	3.2	19	14	6	22
Vehicle speed in off-peak periods	3.4	3.0	3.5	3.1	19	20	12	24
Vertical curves	3.7	3.7	3.2	3.3	13	6	20	20
INTERSECTIONS								
Approach speed in peak periods	3.6	3.4	3.7	3.3	18	19	12	23
Approach speed in off-peak periods	3.5	3.1	3.4	3.1	21	23	18	28
Bicycle facilities	2.4	2.5	3.0	2.4	31	32	29	33
Bicycle volumes	2.4	2.3	2.6	2.3	31	33	34	34
Curb parking on approaches	3.5	3.4	3.4	3.5	21	19	18	20
Grade of approaches	3.5	3.3	3.2	3.5	21	21	24	20
Horizontal alignment of approaches	4.0	3.8	3.3	4.2	11	10	21	8
Intersection sight distance	4.5	4.3	4.0	4.4	1	3	6	2
Intersection skew angle	4.2	3.7	3.3	4.2	4	12	21	8
Lane widths on approaches	4.1	3.5	3.8	3.8	9	18	10	15
Lighting	3.6	3.3	3.3	3.8	18	21	21	15
Level of Service (LOS)	3.7	3.6	4.2	3.6	17	14	2	18
Median type/presence of median	3.6	3.7	3.2	3.6	18	12	24	18
Number of intersection legs	4.2	3.8	3.7	4.4	4	10	12	2
Number of through lanes on approaches	4.1	3.6	3.9	4.2	9	14	7	8

Table A-9. Summary of Priority Ratings of Candidate Input Variables (Continued)

Candidate input variable	Average priority rating				Rank order			
	State	Local	MPO	Task force	State	Local	MPO	Task force
INTERSECTIONS (Continued)								
Number and length of added through lanes	3.9	3.6	3.6	4.1	13	14	16	11
Older drivers/driver population characteristics	2.4	1.7	2.7	2.5	31	34	32	32
Pedestrian facilities	2.8	2.8	3.4	3.3	28	26	18	23
Pedestrian volumes	3.0	2.8	3.1	3.3	27	26	27	23
Presence/number of left-turn curves	4.2	4.2	4.2	4.7	4	4	2	1
Presence of median refuge area for pedestrians	3.4	2.9	3.6	3.3	24	25	16	23
Presence of right-turn lanes	4.0	3.6	3.8	4.3	11	14	10	6
Shoulder/curb type on approaches	2.8	2.6	2.7	3.1	28	29	32	28
Shoulder/curb width on approaches	3.1	2.7	2.8	3.1	26	28	31	28
Signal phasing (e.g., left-turn phasing)	4.2	4.5	3.9	4.3	4	1	7	6
Signal timing	3.8	4.2	3.9	3.9	15	4	7	13
Signal visibility	3.9	4.2	3.7	3.7	13	4	12	17
Spacing between intersections and nearby driveways	3.8	4.2	3.7	3.9	15	4	12	13
Type of traffic control	4.5	4.2	4.2	4.4	1	4	2	2
Traffic volumes (AADTs) for major- and minor road legs (veh/day)	4.4	3.9	4.1	4.4	3	9	5	2
Traffic volumes in peak period (veh/h)	4.2	4.4	4.3	4.1	4	2	1	11
Transit facilities	2.1	2.6	3.1	2.6	34	29	27	31
Type of left-turn channelization (painted or curb)	2.8	3.1	2.9	3.2	28	23	30	27
Vehicle mix (e.g., percent trucks)	3.2	2.6	3.2	3.4	25	29	24	22

Table A-10. Additional Candidate Input Variables in Descending Priority Order as Recommended by Those Surveyed

Additional input variable	Number of respondents citing the additional input variable (by respondent type)				Total	Percent of all respondents
	State	Local	MPO	Task Force		
Adjacent land use			2	3	5	4.6
Weather conditions	1	1	1	2	5	4.6
Accident history	4				4	3.7
Roundabouts	3			1	4	3.7
School zones	1		2	1	4	3.7
Multiple turn lanes			1	2	3	2.8
Presence of channelized right turn lanes	1			2	3	2.8
Road hierarchy/classification			1	2	3	2.8
Advanced warning	1			1	2	1.8
Bus turnouts/priority			2		2	1.8
Driveways per mile		1		1	2	1.8
Enforcement presence/activity levels			1	1	2	1.8
Left-turn volume				2	2	1.8
Location of signal heads (mast arm vs. pedestal)	1	1			2	1.8
Percent of vehicles turning at an intersection	1			1	2	1.8
Queue and storage lengths	1		1		2	1.8
Rumble strips	2				2	1.8
Signal phasing sequence		1		1	2	1.8
Accident rates for similar facilities	1				1	0.9
Cameras (red light and speed)	1				1	0.9
Detection of platoons	1				1	0.9
Frequency of unsignalized intersections				1	1	0.9
Input from other software packages	1				1	0.9
Inter-green interval				1	1	0.9
Intersection conflicts (including ped/bike)				1	1	0.9
Intersection leg offset				1	1	0.9
Median openings per mile		1			1	0.9
Median widths for intersections				1	1	0.9
Number of overhead signal heads				1	1	0.9

Table A-10. Additional Candidate Input Variables in Descending Priority Order as Recommended by Those Surveyed (Continued)

Additional input variable	Number of respondents citing the additional input variable (by respondent type)				Total	Percent of all respondents
	State	Local	MPO	Task Force		
Pavement markings through the intersection	1				1	0.9
Pedestrian signals	1				1	0.9
Right turn on red prohibited	1				1	0.9
Sidewalks			1		1	0.9
Signal coordination	1				1	0.9
Signals per mile		1			1	0.9
Spatial variables			1		1	0.9
Toll booths			1		1	0.9
Turn restrictions				1	1	0.9
TWTL or special purpose lanes				1	1	0.9
Underpass or tunnel approaches and facilities		1			1	0.9
TOTAL	25	7	14	28	74	

In Question 6, agencies were asked to provide any additional measures of safety effectiveness that they currently utilize or recommend utilizing in the safety prediction methodologies other than the three listed above. The suggestions made are summarized in Table A-11. The table shows the number of respondents that mentioned each variable and the percentage of total survey respondents that they represent. It can clearly be seen that the measure accident/crash rate is mentioned most often (13 percent of respondents), but even this is not a significant demand. In addition, some of the proposed measures closely resemble those measures of safety effectiveness currently used in the safety prediction methodology in the first edition of the HSM.

DATA AVAILABILITY

The survey questions related to data availability were directed only to representatives of public agencies that operate and maintain roadways (i.e., state and local highway agencies).

In Question 7, agencies were asked whether they had electronic accident data, roadway segment inventory data, and intersection inventory data files available within their jurisdiction. Table A-12 summarizes the number of affirmative responses received from those surveyed. This table also provides a column presenting the number of agencies that have all three types of data available. Contacts have been made to those agencies that indicated that they possess computerized data files for each data type to determine if such data would be useful for model development in this research.

Question 8 was a follow-up to the question found above dealing with accident data. Agencies were asked if driveway-related crashes could be distinguished from other nonintersection crashes in their computerized files. The responses to this question are summarized in Table A-13. Fifty percent of agencies responding indicated that driveway-related crashes were distinguishable within their computerized data files.

In Question 9, agencies were again referred to previous questions to find out if they had the ability to link computerized accident data to inventory data for the roadway segment on which the accident occurred, or the intersection to which the accident was related. Table A-14 summarizes the responses to this question. The capability to link accidents to the location at which they occurred is of critical importance for modeling to create a safety in creating a prediction methodology. However, only 18 percent of respondents indicated that both roadway segment and intersection data can be linked to accident data.

Table A-11. Additional Measures of Safety Effectiveness in Descending Priority Order as Recommended by Those Surveyed

Additional Measures of Safety Effectiveness	Number of respondents citing the additional input variable (by respondent type)				Total	Percent of all respondents
	State	Local	MPO	Task Force		
Accident/crash rate	6	6	1	1	14	12.8
Combination of crash severity and type distribution	3				3	2.8
Crash distribution by TOD	1	1	1		3	2.8
Crash reduction factors based on improvement	1	1	1		3	2.8
Actual crash frequency vs. average crash frequency	2				2	1.8
Crash distribution by pavement surface condition		1	1		2	1.8
Crash distribution by weather	1		1		2	1.8
Crash distribution by vehicle type			2		2	1.8
Economic loss/ cost-benefit analysis	1	1			2	1.8
Annual injury and fatality accident/crash frequency				1	1	0.9
Capacity/LOS vs. safety	1				1	0.9
Crash cause		1			1	0.9
Crash distribution by light conditions			1		1	0.9
Crash rate by measure of exposure	1				1	0.9
Crash type distribution by mode (i.e., auto, bike, ped)	1				1	0.9
Index for defining a problem			1		1	0.9
Index for type distribution for functional class			1		1	0.9
Index for nominalizing crashes to rank intersections	1				1	0.9
Work zone crash rates	1				1	0.9
TOTAL	20	11	10	2	43	

Table A-12. Availability of Highway Agency Computerized Data Files for Urban and Suburban Arterials

Agency/organization type	Number (percentage) of agencies with computerized files available			
	Accident data	Roadway segment inventory data	Intersection inventory data	All three data types
State highway agencies	36 (97.3)	24 (64.9)	13 (35.1)	12 (32.4)
Local highway agencies	13 (65.0)	8 (40.0)	9 (45.0)	5 (25.0)
TOTAL	49 (86.0)	32 (56.1)	22 (38.6)	17 (29.8)

Table A-13. Ability of Highway Agency Accident Data to Distinguish Driveway-Related Non-intersection Accidents

Agency/organization type	Number (percentage) of agencies with accident data that identify driveway-related accidents	
	Yes	No
State highway agencies	20 (54.1)	17 (45.9)
Local highway agencies	8 (42.1)	11 (57.9)
TOTAL	28 (50.0)	28 (50.0)

Table A-14. Ability of Highway Agencies to Link Roadway Segment and Intersection Inventory Data Files to Accident Data

Agency/ organization type	Number (percentage) of agencies with inventory data files that can be linked to accident data					
	Roadway segments		Intersections		Both roadway segments and intersections	
	Yes	No	Yes	No	Yes	No
State highway agencies	21 (56.8)	16 (43.2)	10 (27.0)	27 (73.0)	9 (24.1)	28 (75.9)
Local highway agencies	1 (5.3)	18 (94.7)	4 (21.1)	15 (78.9)	1 (5.3)	18 (94.7)
TOTAL	22 (39.3)	34 (60.7)	14 (25.0)	42 (75.0)	10 (17.9)	46 (82.1)

Appendix B

Draft Version of HSM Chapter 10 can be provided upon request to NCHRP.