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16. Abstract

Highway safety is an ongoing concern to the Texas Department of Transportation (TxDOT). As part of its proactive commitment to improving highway safety, TxDOT is moving toward including quantitative safety analyses earlier in the project development process. The objectives of this research project are: (1) the development of safety design guidelines and evaluation tools to be used by TxDOT designers, and (2) the production of a plan for the incorporation of these guidelines and tools in the planning and design stages of the project development process.

This document describes the effect of key design components on street and highway safety. The information presented herein represents the findings from a critical review of the literature and an evaluation of the reported safety trends and relationships. The purpose of this document is to promote the explicit and objective consideration of safety in the design process. It is envisioned to be a reference document that will be useful to engineers and researchers who desire detailed safety information on various highway geometric design elements. The information in this document was used to develop the guidelines presented in the *Interim Roadway Safety Design Workbook*.

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ROADWAY SAFETY DESIGN SYNTHESIS

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data published herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) and/or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation. It is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was James Bonneson, P.E. #67178.

NOTICE

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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TABLE OF CONTENTS

Page

TABLE OF CONTENTS (Continued)

Page

LIST OF FIGURES

Figure **Page**

LIST OF FIGURES (Continued)

Figure **Page**

LIST OF TABLES

Table **Page**

LIST OF TABLES (Continued)

Table **Page**

LIST OF TABLES (Continued)

Table **Page**

Chapter 1
Introduction

Page

OVERVIEW

The traditional approach to highway geometric design incorporates a nominal level of safety through adherence to minimum design criteria for key design elements. The level of safety provided is referred to as "nominal" because the correlation between actual crash frequency and key design elements is unknown by the designer. Ideally, guidance relating design choices to crash frequency (and severity) would be available to help guide the engineer during the design process such that:

- Each design component provides an acceptable level of safety.
- The design features that comprise the design are consistent in the degree of safety provided.
- The combination of design components for each design feature is cost-effective (i.e., neither over- nor under-designed).
- The benefits derived from the resulting design can be shown to outweigh its costs and represent the best use of limited program funds.

The objective of this document is to synthesize information in the literature that quantitatively describes the relationship between various geometric design components and safety. This information is intended to provide a basis for the development of a procedure for estimating the safety benefit of alternative designs. This procedure is documented in the *Roadway Safety Design Workbook* (*1*).

HIGHWAY SAFETY AND GEOMETRIC DESIGN

This part of the chapter describes the nature of the highway safety problem and the role of geometric design in providing a safe highway system. Initially, crash data are examined to quantify current highway crash trends in Texas and the United States. Then, the need for a more explicit consideration of safety in the design process is outlined. Finally, a process is described that can be used by designers to evaluate the impact of alternative design decisions on safety.

Defining the Safety Problem

During the past 40 years, the safe and efficient operation of the nation's highways has been a growing public concern. High-speed freeways, changes in vehicle size and power, and an increasingly mobile society led to serious safety problems and an increasing death toll on roads in the United States. Fatal crash data cited in *Traffic Safety Facts* (*2*) indicate that 50,900 people died in motor vehicle related crashes in 1966. The fatality rate that year was 5.5 fatalities per 100 million vehicle-miles traveled (f/hmvm). By 1995, additional safety technology in vehicles and roadside objects, and improved design processes resulted in this rate being reduced to 1.73 f/hmvm (a 69 percent reduction).

In 1995, the American Association of State Highway and Transportation Officials (AASHTO) undertook the development of a strategic highway safety plan. The plan was adopted in December 1997 (*3*). The goal of the plan was to reduce fatalities by 5000 to 7000 per year by year 2004 (i.e., to between 37,000 and 39,000 fatalities in 2004). The data in Table 1-1 provide an indication of progress toward this goal. As indicated by these data, the annual number of fatalities has not changed; however, the amount of travel has increased by 13 percent. As a result, the fatality rate has decreased from 1.73 to 1.52 f/hmvm (a 12 percent reduction). Nevertheless, further safety improvements will be needed nationally to reduce the annual number of fatalities and achieve the AASHTO goal.

Table 1-1. National Motor Vehicle Crash Statistics for 1995 and 2000.

Category	Statistics	Year		Change, %
		1995	2000	
Crashes	Total	6,699,000	6,394,000	-5
	Fatal	37,200	37,400	1
Fatalities	Total	41,800	41,800	0
	Collisions with pedestrians	5600	4700	-16
	Collisions with bicycles	800	700	-13
	Other	35,400	36,400	3
Exposure	Vehicles registered	197,065,000	217,028,000	10
	Motor-vehicle-miles (millions)	2,423,000	2,750,000	13
Rates	Fatal crashes per 100 million vehicle-miles	1.54	1.36	-11
	Fatalities per 100 million vehicle-miles	**1.73**	**1.52**	**-12**

Crash data from Texas were compared to similar data for the United States to gain insight on the nature of the highway safety problem within Texas. Summary statistics that facilitate this comparison are listed in Table 1-2. They are based on crash records for 2000. They indicate that 3800 fatalities occurred on Texas highways that year, representing about 9 percent of the nation's 41,800 fatalities. When normalized by vehicle-miles of travel, Texas had a fatality rate of 1.73 f/hmvm, which is about 14 percent higher than the national average. It was also noted that pedestrian fatality rates in Texas are about 20 percent higher than the national average.

Conscious Consideration of Safety

In 1974, AASHTO defined a safe roadway as follows:

> "*A safe roadway is one in which none of the driver-vehicle-roadway interactions approaches the critical level at any point along its length.*" (*4*)

Put another way, neither the physics of keeping a vehicle on the roadway nor the driver's mental workload should approach their limits on a safe roadway. Roadway designers should keep this ideal condition in mind and provide for it wherever possible. Intuitively, a straight, flat roadway with few at-grade access points is safer than a roadway with steep grades, sharp curves, and frequent access points. However, few roadways can be constructed without some consideration for curvature, grade, and access. In practice, the roadway designer must consult guidelines, conduct analyses, and rely on judgment to determine the most acceptable curve radii and allowable grades.

Table 1-2. Comparison of Texas with National Motor Vehicle Crash Statistics.

Category	Statistics	Year 2000		Difference, %
		United States	Texas	
Crashes	Total	6,394,000	300,000	
	Fatal	37,400	3200	
Deaths	Total	41,800	3800	
	Collisions with pedestrians	4700	412	
	Collisions with bicycles	700	37	
	Other	36,400	3351	
Exposure	Vehicles registered	217,028,000	14,300	
	Motor-vehicle-miles (millions)	2,750,000	220,000	
	Population (thousands)	274,600	20,100	
Rates	Fatal crashes per 100 million vehicle-miles	1.36	1.45	7
	Fatalities per 100 million vehicle-miles	**1.52**	**1.73**	**14**
	Pedestrian fatalities per 100,000 persons	1.71	2.05	20
	Bicycle fatalities per 100,000 persons	0.25	0.18	-28

Design guidelines often identify minimum acceptable design criteria that, if adhered to, are intended to provide a safe highway design. These criteria appear in various places, such as AASHTO's *A Policy on Geometric Design of Highways and Streets* (*5*) and TxDOT's *Roadway Design Manual* (*6*). Oftentimes, considerations of cost, safety, and efficiency have resulted in numerous elements of a roadway just satisfying the criteria. Subsequent experience with some of these roadways has revealed that more generous (and more forgiving) levels of design are associated with fewer crashes. Closer examination of the elements associated with minimum design often revealed that their combination at specific locations on the roadway may accumulate to push drivers and vehicles to their performance limits. In recognition of this realization, AASHTO commented in 1974:

> *"...While minimum standards may have been adequate for the existing or even the assumed conditions, they may be inadequate if speeds and traffic volumes increase more rapidly than anticipated...Frequently, a more liberal design would have cost little more over the life of the project and would increase its safety and usefulness substantially...Often the safety deficiencies generated by minimum design are impossible to correct by any known device or appurtenance. A warning sign is a poor substitute for adequate geometric design...Highways built with high design standards put the traveler in an environment which is fundamentally safer because it is likely to compensate for the driving errors he will eventually make."* (*4*)

AASHTO additionally recommended consistency in design and the use of above-minimum design element dimensions, especially in the areas of sight distance, right-of-way width, and sideslopes. This recommendation is intended to result in the use of more "forgiving" design dimensions and thereby, produce a safer highway system. In more recent years, this idea has evolved into a process called Safety-Conscious Design.

Overview of Safety-Conscious Design

In recent years, the safety provided by the nation's highway system has come under additional scrutiny, not just for safety but increased security. The Transportation Equity Act for the 21st Century (TEA-21) has required states to provide for consideration of projects and strategies that will increase the safety and security of the transportation system for motorized and non-motorized users. To achieve further large reductions in crash frequency, research indicates that it will be necessary to change the focus of future safety initiatives from driver behavior to design policies and technologies that reduce the likelihood of a crash, just as AASHTO recommended in 1974. The conscious consideration of safety in the design processes is one way to accomplish this goal.

Safety-conscious design represents the explicit evaluation of the safety consequences associated with design alternatives. It is incorporated at key points in the design process where changes necessary to accommodate safety considerations can be easily incorporated. This process can be contrasted with the traditional design process where minimum design criteria are used to constrain design element combinations and sizes with the implicit assumption that the resulting design will provide an acceptable level of safety.

The concept of "safety-conscious design" was first described in *Special Report 214* (*7*). More recently, the Transportation Association of Canada (TAC) incorporated safety-conscious design in its design guide for new location and reconstruction projects (i.e., the *Geometric Design Guide for Canadian Roads*) (*8*). The justification offered for changing its design philosophy was the observation that: (1) the traditional approach to design has become less dependent on experience and judgment and more dependent on adherence to minimum criteria; (2) there is a belief among designers that safety is an automatic by-product of the design process; and (3) the difficulties associated with quantifying safety have relegated safety considerations to being only a secondary objective of the design process.

Safety-conscious design can be implemented by using safety evaluation tools to quantify the effect of alternative design choices on safety. These tools, combined with economic principles, are used to evaluate the benefits and costs of design alternatives. In recognition of the time required to use these tools, the evaluations tend to be reserved for more complex design conditions or those that involve high construction costs.

ROLE OF SAFETY-CONSCIOUS DESIGN IN THE TxDOT DESIGN PROCESS

This part of the chapter provides an overview of TxDOT's design process and illustrates where safety-conscious design concepts are applied (or can be introduced). This process is part of a larger project development process that takes the project from concept to letting. This process consists of six stages: planning and programming; preliminary design; environmental; right-of-way and utilities; plans, specifications, and estimates (PS&E) development; and letting. The planning and programming, preliminary design, and PS&E development stages are stages where safety can be explicitly considered in the design process. The sequence of these stages in the development process is shown in Figure 1-1.

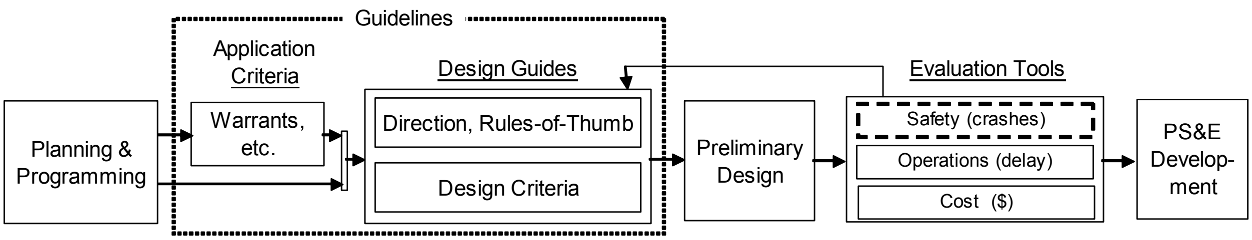

Figure 1-1. Components of the Project Development Process.

As indicated by Figure 1-1, evaluation tools are used by the designer to verify the performance potential of alternative designs. The evaluation quantifies the design's performance in terms of safety, operations, construction cost, etc. The objective of this evaluation is to ensure that the design offers a reasonable balance between cost and effectiveness. Tools are readily available to conduct level-of-service analyses and to estimate construction costs. Tools for quantifying an alternative's impact on safety are not as readily available at this time.

There are two stages of the project development process that embody the design process. These stages are: (1) preliminary design and (2) PS&E development. During the preliminary design stage, the location of a facility (if it is new or being relocated) and its major design features are identified. Then, alternative locations and features are considered and the more promising ones are evaluated in greater detail.

The final design of the proposed facility is undertaken during the PS&E development stage. The environmental and right-of-way issues have generally been resolved by the start of this stage and a "preferred" alignment has been identified. The main product of this stage is a completed plan set with appropriate specifications for construction. Another product of this stage is a list of estimated material quantities needed for the construction bidding process.

Table 1-3 identifies safety tasks that can be undertaken in TxDOT's project development process *(9)*. Also identified is the step in the corresponding development process stage within which they would be conducted.

As indicated in Table 1-3, "key" design elements are identified in Step 4 of the preliminary design stage and then used to direct the safety evaluation tasks. Key design elements are those elements that: (1) are associated with the "controlling criteria" that dictate the need for a design exception or have a known effect on safety, and (2) are used in situations where atypical conditions exist, the design is complex, or construction costs are high. The controlling criteria vary by project type *(6)*; those applicable to Rehabilitation Projects (3R) include:

- Design Speed
- Shoulder Width
- Structural Capacity
- Vertical Alignment
- Stopping Sight Distance
- Lane Width
- Bridge Width
- Horizontal Alignment
- Grade

Table 1-3. Potential Safety Tasks in the Project Development Process.

Stage	Step	Potential Safety-Related Task
Planning and programming	1. Needs identification	• Screen facilities for locations with safety needs.
Preliminary design	1. Preliminary design conference	• Document safety needs. • Identify atypical conditions, complex elements, and high-cost components.
	2. Data collection/preliminary design preparation	• Diagnose safety data to identify crash patterns. • Refine project scope if necessary.
	4. Preliminary schematic	• Perform preliminary level of safety analysis for "key" design elements.[1]
	5. Geometric schematic	• Perform detailed level of safety analysis for "key" design elements.[1]
	6. Value engineering	• Compare cost of specific elements and overall roadway with safety and operational benefits.
	7. Geometric schematic approval	• Document safety of design choices (use results for design exception request, if necessary).
PS&E development	3. Final alignments/profiles	• Re-evaluate alignment, cross section, and roadside design to ensure acceptable level of safety.
	9. Traffic control plan	• Evaluate safety of long-term detour roadway design.

Note:
1 - Key design elements are those elements that: (1) are associated with the controlling criteria specified for the project or have a known effect on safety, and (2) are used in situations where atypical conditions exist, the design is complex, or construction costs are high.

The controlling criteria for New Location and Reconstruction Projects (4R) include all of the above criteria plus: cross slope, superelevation, and vertical clearance. Additional important design elements that may also be considered as "key" because of their known effect on safety include: turn bays at intersections, median treatment, and clear zone (i.e., horizontal clearance). For non-key design elements, the traditional design process (i.e., compliance with design criteria and warrants) will likely provide an acceptable level of safety.

The implementation of these tasks will add time to the design process. However, by limiting the evaluation of safety to primarily "key" design elements, it is hoped that the additional time required will be kept to a minimum and incurred only where it is likely to provide some return in terms of improved safety, lower construction cost, or both. This added time represents an immediate and direct cost to the design process. However, it also represents a more cost-effective approach to design because additional benefit will be derived through fewer crashes (by provision of effective features) and lower construction costs (by not over-designing some design elements).

PURPOSE AND ORGANIZATION OF THE SYNTHESIS

Purpose

The purpose of the *Roadway Safety Design Synthesis* is to present a compilation of the available, quantitative safety information for roadway design. Qualitative (i.e., non-numeric) safety information is available from other sources, notably the AASHTO *Highway Safety Design and Operations Guide* (*10*). The safety information presented in this synthesis allows the roadway designer to have a deeper understanding of the sources, ranges, and limitations of the quantitative information contained in the *Roadway Safety Design Workbook* (*1*).

Organization

The *Roadway Safety Design Synthesis* is divided into seven chapters. The first chapter introduces the Synthesis. The subsequent six chapters synthesize the available quantitative information for various roadway facilities. These "quantitative" chapters are titled:

- Freeways,
- Rural Highways,
- Urban Streets,
- Interchange Ramps,
- Rural Intersections, and
- Urban Intersections.

Each chapter contains two main parts. The first part describes safety prediction models that predict the expected number of crashes that will occur on a particular roadway segment, interchange ramp, or intersection. These models are compared and discussed, providing a roadway designer some insight into the model types and design-related factors that correlated with crash frequency. The safety performance models in each chapter were used to generate the crash rates shown in the corresponding chapter of the *Roadway Safety Design Workbook* (*1*).

The second part of each chapter contains accident modification factors (AMFs) for various design-related factors that have been found to be correlated with crash frequency. AMFs represent the relative change that occurs in crash frequency when a particular geometric component is added, removed, or changed in size. As such, it is multiplied by the expected crash frequency before the change to estimate the expected crash frequency after the change. An AMF in excess of 1.0 is an indication that the corresponding change will increase crash frequency. An AMF less than 1.0 indicates that the change will decrease crash frequency.

Differences in crash reporting threshold among agencies can introduce uncertainty in crash data analysis and regional comparison of crash trends. A majority of the crashes that often go unreported (or, if reported, not filed by the agency) are those identified as "property-damage-only." In contrast, severe crashes (i.e., those crashes with an injury or fatality) tend to be more consistently reported across jurisdictions. Thus, safety relationships tend to be more transferrable among jurisdictions when they are developed using only severe crash data. In recognition of this benefit, this document is focused on models and AMFs applicable to severe crashes. Unless explicitly stated otherwise, all references to "crash frequency" refer to *severe* crash frequency.

There is frequent reference in this document to the traffic "lane" or "lanes" provided in the roadway or at an intersection. All such references pertain to the *through* traffic lane or lanes, unless explicitly stated otherwise. Lanes for turning traffic are referenced in terms of the turn maneuver they serve (e.g., two-way left-turn lane, right-turn lane, etc.).

The AMFs and crash rates in this document are derived from research conducted throughout the United States, including Texas. All of the research findings were screened for applicability to Texas conditions. Several AMFs require the distribution of crashes (by crash type or median type) as an input. The distributions tabulated herein for these AMFs were obtained from the crash database maintained by the State of Texas, Department of Public Safety (DPS).

The AMFs described in this document were neither compared to DPS crash data to confirm the stated trends, nor calibrated to Texas conditions. As a general rule, AMFs are not the subject of such examinations on a local level due to the high cost of the effort and the limitations of local data. Moreover, experience indicates that AMFs derived from large, multi-state databases tend to provide the accuracy needed for the comparative assessment of design alternatives for any given location.

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Chapter 2
Freeways

Page

INTRODUCTION

A freeway is intended to provide mobility to travelers, with minimal interference from entering or exiting traffic. It has a divided, multilane cross section that reflects a "generous" design condition (i.e., design element sizes consistently exceed minium design criteria). The freeway is grade-separated whenever it intersects another route such that freeway movements flow freely, without interruption by traffic control devices. Freeways require considerable amounts of right-of-way. The cost of this right-of-way, especially in urban areas, presents challenges to the development of freeway designs that are not only safe and efficient but also cost-effective.

The development of a safe, efficient, and economical freeway design typically reflects the consideration of a variety of design alternatives, especially when access to the freeway is included. A variety of techniques exist for estimating the operational benefits of alternatives; many are automated through software tools. Techniques for estimating construction and right-of-way costs are also available to the designer. Unfortunately, techniques for estimating the safety benefits of alternative designs are not as readily available. This chapter summarizes information in the literature that can be used to estimate the crash frequency associated with various freeway design alternatives.

Objective

The objective of this chapter is to synthesize information in the literature that quantitatively describes the relationship between various freeway design components and safety. This information is intended to provide a basis for the development of a procedure for estimating the safety benefit of alternative designs. This procedure is documented in Chapter 2 of the *Roadway Safety Design Workbook* (*1*).

The presentation consists of an examination of safety prediction models and accident modification factors (AMFs). Safety prediction models provide an estimate of the expected crash frequency for a typical freeway segment. They include variables for traffic volume and segment length. They also include variables for other factors considered to be correlated with crash frequency (e.g., median type, number of lanes, etc.). One or more AMFs can be multiplied by the expected crash frequency obtained from the prediction model to produce an estimate of the expected crash frequency for a specific freeway segment.

Scope

This chapter addresses the safety of the main lanes of a freeway segment. It does not address the safety of interchange ramps, ramp gores, and frontage roads. For this reason, the crashes addressed herein are referred to as "mid-junction" crashes. Crashes that occur at, or are related to, ramps and frontage roads are the subject of Chapter 5.

When available, safety relationships that estimate the frequency of *severe* (i.e., injury or fatal) crashes are given preference for inclusion in this document. This preference is due to a wide variation in reporting threshold among cities and states. This variation complicates the extrapolation of crash trends found in one location to another location. Moreover, it can confound the development of safety prediction models using data from multiple agencies. Reporting threshold is strongly correlated with the number of property-damage-only (PDO) crashes found in a crash database. Agencies with a high reporting threshold include relatively few PDO crashes in their database and vice versa. As a consequence, the total crash frequency for a given roadway will be low if it is located in an area with a high reporting threshold. In contrast, this same roadway will have a high crash frequency if it is located in an area with a low reporting threshold. This problem is minimized when crash data analyses, comparisons, and models are based on data pertaining only to severe crashes.

Overview

This chapter documents a review of the literature related to freeway safety. The focus is on quantitative information that relates severe crash frequency to various geometric design components of the freeway main lanes. The review is not intended to be comprehensive in the context of referencing all works that discuss freeway safety. Rather, the information presented herein is judged to be the most current information that is relevant to freeway design in Texas. It is also judged to be the most reliable based on a review of the statistical analysis techniques used and the explanation of trends.

Where appropriate, the safety relationships reported in the literature are compared herein, with some interpretation offered to explain any differences noted. The relationships are typically presented as reported in the literature; however, the names or the units of some variables have been changed to facilitate their uniform presentation in this chapter.

This chapter is envisioned to be useful to design engineers who desire a more complete understanding of the relationship between various freeway design components and severe crash frequency. As previously noted, it is also intended to serve as the basis for the development of the safety evaluation procedure described in Chapter 2 of the *Roadway Safety Design Workbook* (*1*).

This chapter consists of two main parts. In the first part to follow, several safety prediction models reported in the literature are described. In the second part, accident modification factors are described. Within this part, the various factors examined are organized into the following categories: roadway geometric design, roadside design, and "other" factors.

SAFETY PREDICTION MODELS

Described in this part of the chapter are several models that were developed to estimate the expected frequency of crashes on freeway segments. Four of the five models were specifically developed for freeway segments. The fifth model was developed for rural divided highways, which are similar to rural freeways except for the provision of access from intersecting roadways. After the models are summarized, they are then compared graphically for a range of common input conditions.

Hadi Models

The models described in this section are derived from the equations reported by Hadi et al. (*2*). They used negative binomial regression analysis to calibrate a set of safety prediction models using data from Florida roadways. The models are categorized by crash severity, area type (i.e., urban, rural), and number of through lanes. The three freeway models developed by Hadi et al. address the following facility types:

- rural freeways with four or six lanes,
- urban four-lane freeways, and
- urban six-lane freeways.

These models are listed below as Equations 2-1, 2-3, and 2-5, respectively.

$$C_{RF} = 0.25\ ADT^{0.9599}\ (1000\,L)^{0.9107}\ e^{B_{RF}} \tag{2-1}$$

with,

$$B_{RF} = -14.032 - 0.0407\ W_{is} + 0.2127\ N_x \tag{2-2}$$

where:
C_{RF} = frequency of mid-junction injury crashes on rural freeways, crashes/yr;
ADT = average daily traffic, veh/d;
L = freeway segment length, mi;
W_{is} = inside shoulder width, ft; and
N_x = number of interchanges on freeway segment.

$$C_{UF,4} = 0.25\ ADT^{1.1832}\ (1000\,L)^{0.7733}\ e^{B_{UF,4}} \tag{2-3}$$

with,

$$B_{UF,4} = -10.61 - 0.307\,W_l - 0.0232\,W_s - 0.0154\,V + 0.24\,N_x - 0.06\,(W_m)^{0.5} \tag{2-4}$$

where:
$C_{UF,4}$ = frequency of mid-junction injury crashes on urban four-lane freeways, crashes/yr;
W_l = lane width, ft;
W_s = outside shoulder width, ft;
V = speed limit, mph; and
W_m = median width, ft.

$$C_{UF,6} = 0.25 ADT^{1.405} L^{0.93} e^{B_{UF,6}} \tag{2-5}$$

with,

$$B_{UF,6} = -14.04 - 0.339\, W_l - 0.0594\, W_s - 0.031\, (W_m)^{0.5} \tag{2-6}$$

where:

$C_{UF,6}$ = frequency of mid-junction injury crashes on urban six-lane freeways, crashes/yr.

The models developed by Hadi et al. (*2*) estimate injury crash frequency only; they do not include PDO or fatal crash frequency. Hadi et al. prepared separate models for estimating total crash frequency and fatal crash frequency. Analysis of these models indicates that the estimates obtained from Equations 2-1, 2-3, and 2-5 should be inflated by 5 percent (i.e., multiplied by 1.05) to obtain an estimate of severe crash frequency.

The sign of the constant associated with any single variable in the model's linear terms (i.e., those variables in Equations 2-2, 2-4, and 2-6) indicates the correlation between a change in the variable value and injury crash frequency. For example, the negative sign of the constant associated with speed limit V in Equation 2-4 indicates that injury crash frequency is lower on roads with a higher speed limit. This trend is likely a reflection of the fact that higher speed roads are typically built to have more generous design element sizes and more forgiving roadside safety features. It may also be possible that drivers are more cautious when driving at higher speeds.

The characteristics of the data used by Hadi et al. (*2*) are summarized in Table 2-1. They used four years of crash data to develop their safety prediction models. They did not report the number of road segments reflected in the database. However, the road segments used were on the Florida state highway system.

Table 2-1. Database Characteristics for Various Freeway Safety Prediction Models.

Model Developers	Database Characteristics				
	Area Type	Years of Crash Data	Number of Road Segments	Total Section Length, mi	Number of Through Lanes
Hadi et al. (*2*)	Rural	4	not available	not available	4, 6
	Urban	4	not available	not available	4
	Urban	4	not available	not available	6
Persaud & Dzbik (*3*)	Urban	2	not available	991	4
	Urban	2	not available	247	More than 4
Wang et al. (*4*)	Rural	6	622	432	3, 4

Persaud and Dzbik Models

Persaud and Dzbik (*3*) developed two prediction models using data for urban freeways in Ontario, Canada. Separate models were developed for total crashes and severe (fatal plus injury) crashes. The models for severe crashes are presented in Equations 2-7 and 2-8.

$$C_{UF,4} = 0.0000354 \ ADT^{1.082} \ L \tag{2-7}$$

$$C_{UF,6} = 0.0000099 \ ADT^{1.206} \ L \tag{2-8}$$

Unlike the models by Hadi et al. (*2*), the models by Persaud and Dzbik do not include variables for specific roadway elements (e.g., lane width, posted speed limit, median width, etc.). Moreover, the freeway segments do not distinguish between those with and those without ramp speed-change lanes. Because speed-change lanes tend to be associated with more crashes than basic freeway segments, the model predictions may overestimate crash frequency for mid-junction segments.

The characteristics of the data used by Persaud and Dzbik (*3*) are summarized in Table 2-1. They used two years of crash data to develop their safety prediction models. They reported that they used approximately 500 freeway "sections" in their database. However, they did not provide a breakdown of the number of segments that had four lanes and the number that had more than four lanes.

Wang Model

Wang et al. (*4*) developed a safety prediction model for rural multilane divided highways using crash and geometry data from Minnesota. The database does not include data for freeways. However, the behavior of a rural divided highway with few or no access points is similar to that of a rural freeway. Equations 2-9 and 2-10 describe this model.

$$C_{RH} = 0.000233 \ ADT^{1.073} \ L^{1.073} \ e^{B_{RH}} \ (1 - 0.01 \ PDO) \tag{2-9}$$

with,

$$\begin{aligned} B_{RH} &= 0.131 \ R_{haz} - 0.151 \ I_{ac} + 0.034 \ D_d + 0.163 \ D_{it} + 0.052 \ D_{wo} \\ &\quad - 0.572 \ I_{fc} - 0.094 \ W_s - 0.003 \ W_m + 0.429 \ I_{at} \\ &= 0.131 \ (1) - 0.151 \ (1) + 0.034 \ (0) + 0.163 \ (0) + 0.052 \ (0) \\ &\quad - 0.572 \ (1) - 0.094 \ W_s - 0.003 \ W_m + 0.429 \ (0) \\ &= -0.592 - 0.094 \ W_s - 0.003 \ W_m \end{aligned} \tag{2-10}$$

where:

C_{RH} = frequency of severe mid-junction crashes on rural multilane highways, crashes/yr;
R_{haz} = roadside hazard rating (1, 2, ..., 7; where 1 represents best and 7 is worst from a safety standpoint);
I_{ac} = access control (1 for partial control, 0 for no control);
D_d = driveway density, driveways/mi;
D_{it} = density of unsignalized intersections with turn lanes, intersections/mi;
D_{wo} = density of unsignalized intersections without turn lanes, intersections/mi;
I_{fc} = functional class (1 for rural principal arterial, 0 otherwise);
W_s = outside shoulder width, ft;
I_{at} = area location type (1 for highways in a rural municipality, 0 otherwise); and
PDO = percent property-damage-only crashes on rural multilane divided highways (= 62.5).

As indicated in Equation 2-10, the model contains some variables that are not relevant to freeway applications, such as driveway density and intersection density. Values for these variables that are appropriate for freeways (e.g., driveway density of 0 driveways/mi) were substituted in the original model to obtain a reduced model form suitable for freeway-like highway segments. The roadside hazard rating describes the relative safety of the roadside. A rating of "1" is assigned to roadsides with horizontal clearances of 30 ft or more and side slopes of 1V:4H or flatter.

Equation 2-9 was originally derived to predict total crashes (i.e., PDO plus injury and fatal). An adjustment multiplier of "$1 - 0.01\ PDO$" was added to this equation to convert the prediction into severe crash frequency. This term requires an estimate of the percentage of PDO crashes. This percentage is estimated at 62.5 percent for rural multilane divided highways in Minnesota (*5*).

The characteristics of the data used by Wang et al. (*4*) are summarized in Table 2-1. They used five years of crash data for 622 multilane highway segments to develop their safety prediction models.

Comparison of Crash Models

The crash prediction models described in the previous sections are compared in this section. The objective of this comparison is to determine which model or models are reasonable in their prediction of severe crash frequency. To facilitate this comparison, the models are examined over a range of traffic volume levels. The values of other model variables were set at typical values for freeways. These values are listed in Table 2-2.

Table 2-2. Typical Values Used for Freeway Model Comparison.

Model Variable	Typical Value	Safety Prediction Model Developer				
		Hadi et al. (2)			Persaud & Dzbik (3)	Wang et al. (4)
		Rural	Urban Four Lane	Urban Six Lane	Urban Four & Six	Rural Four Lane
Inside shoulder width, ft	4	✔	--	--	--	--
Outside paved shoulder width, ft	10	--	✔	✔	--	✔
Lane width, ft	12	--	✔	✔	--	--
Median width, ft	30	--	✔	✔	--	✔
Speed limit, mph	55	--	✔	--	--	--
Interchanges per mile	0	✔	✔	--	--	--

The expected severe crash frequencies obtained from the various models are shown in Figures 2-1 and 2-2. The model developer and facility type for a given trend line is indicated in each figure. Several trends can be seen in these figures. First, the trends indicate that severe crash frequency increases in a nearly linear manner with traffic volume. However, crash frequency on urban six-lane freeways increases more rapidly than the crash frequency on urban four-lane freeways. Second, comparing across the two figures, it appears that rural four-lane freeways have fewer crashes than their urban counterparts.

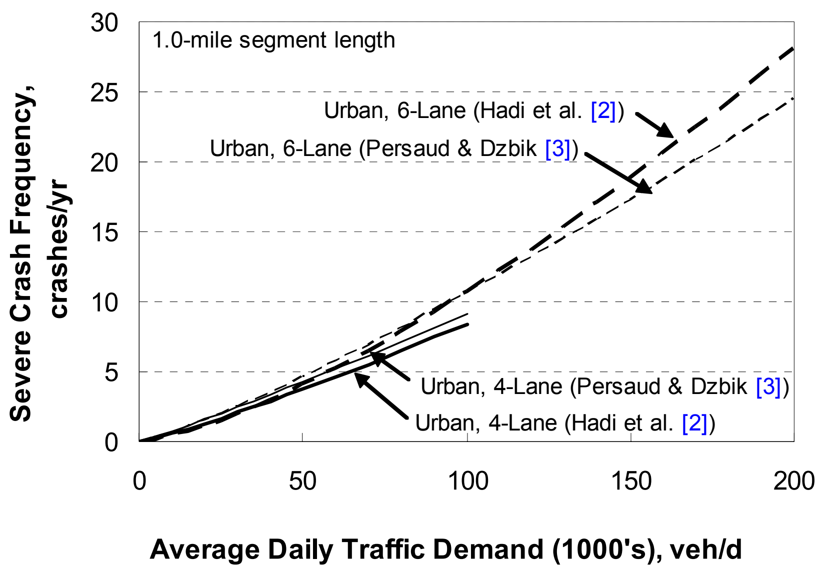

Figure 2-1. Severe Crash Frequency for Urban Freeways.

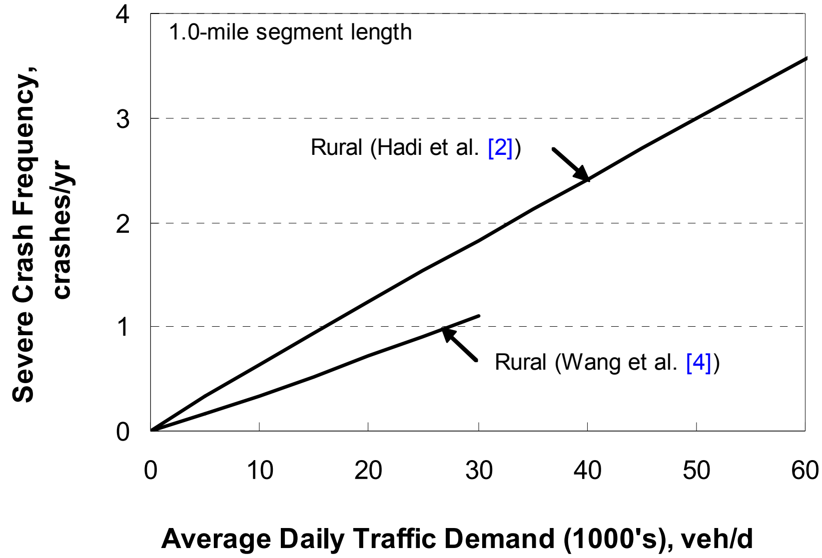

Figure 2-2. Severe Crash Frequency for Rural Freeways.

ACCIDENT MODIFICATION FACTORS

This part of the chapter describes various accident modification factors that are related to the design of an urban or rural freeway. The various factors examined are organized into the following categories: roadway geometric design, roadside design, and "other" factors.

Geometric Design

This section describes AMFs related to the geometric design of a freeway. Topics specifically addressed are listed in Table 2-3. Many geometric design components or elements are not listed in Table 2-3 (e.g., weaving section length) that are also likely to have some effect on severe crash frequency. However, a review of the literature did not reveal useful quantitative information describing these effects. The list of available AMFs for freeway geometric design is likely to increase as new research in this area is undertaken.

Table 2-3. AMFs Related to Geometric Design of Freeways.

Section	Accident Modification Factor		
Horizontal alignment	Horizontal curve radius		
Vertical alignment	Grade		
Cross section	Lane width Median width	Outside shoulder width Shoulder rumble strips	Inside shoulder width

In some instances, an AMF is derived from a safety prediction model as the ratio of "segment crash frequency with a changed condition" to "segment crash frequency without the change." In other instances, the AMF is obtained from a before-after study. Occasionally, crash data reported in the literature were used to derive an AMF.

All of the AMFs described in this section are developed to yield a value of 1.0 when the associated design component or element represents a "typical" condition. Deviation from this base condition to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Horizontal Alignment

This subsection describes AMFs related to horizontal alignment. However, no reliable AMFs specific to freeway alignments were found during the literature review. The only horizontal alignment AMF found in the literature is for horizontal curve radius. This section summarizes this review of the literature.

Zegeer et al. (*6*) developed a model for determining the safety of horizontal curves on two-lane rural highways. This model was subsequently modified by Harwood et al. (*7*) to produce a horizontal curve AMF. The equation for this AMF is:

$$AMF_{cr} = \frac{1.55\,L_c + \dfrac{80.2}{R} - 0.012\,I_s}{1.55\,L_c} \qquad \textbf{(2-11)}$$

where:

AMF_{cr} = horizontal curve accident modification factor;
L_c = length of horizontal curve (= $I_c \times R$ / 5280 / 57.3), mi;
I_c = curve deflection angle, degrees;
R = curve radius, ft; and
I_s = presence of a spiral transition curve (1 if a spiral transition is present, 0 otherwise).

While not developed for freeway segments, this AMF is widely recognized as a reliable reflection of the relationship between curvature and crash frequency on two-lane highways.

Raff (*8*) examined curve and tangent crash rates for four-lane, controlled access highways. A regression analysis of these rates (described in Chapter 3) revealed the following relationship between curve radius and crash rate *CR*: $CR = 1.20 + (6130/R)^2 \times 1/I_c$ (R^2 = 0.96). The AMF derived from this relationship is:

$$AMF_{cr} = 1 + \frac{1}{I_c}\left(\frac{5590}{R}\right)^2 \qquad \textbf{(2-12)}$$

Figure 2-3 compares the two AMFs for a range of curve radii and deflection angles. The trend lines indicate that the AMF converges to 1.0 as the curve radius increases for both two-lane undivided and four-lane divided highways. For a given radius, larger deflection angles correspond to lower AMF values. This trend suggests that most curve crashes are associated with the curve entry maneuver such that curves with a larger deflection angle (for a given radius) are easier for the driver to detect in advance and safely negotiate the entry maneuver. In spite of these basic similarities, the trend lines shown suggest that divided, controlled-access highways have greater sensitivity (in terms of increased crash risk) to curvature than two-lane highways. Given the age of the Raff data and the fact that the controlled-access highways were not explicitly stated to be freeways, an accurate AMF for freeway horizontal curvature cannot be derived from these data.

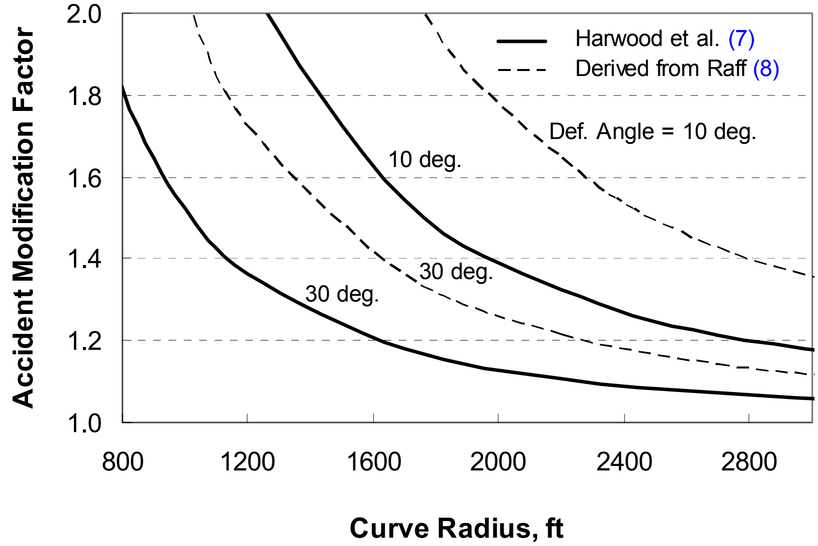

Figure 2-3. Horizontal Curve AMF for Freeways.

Vertical Alignment

This subsection describes AMFs related to features of the freeway's vertical alignment. At this time, the only AMF addressed in this subsection is grade.

The relationship between grade and crash frequency was derived from the safety prediction model developed by Milton and Mannering (*9*). This AMF is based on a mix of urban and rural multilane highways in Washington State. The highways were classified as principal arterials. They have up to six lanes and ADTs of up to 18,000 veh/d per lane. This AMF is shown in Figure 2-4.

Also shown in Figure 2-4 is an AMF developed by Harwood et al. (*7*). It is based on rural two-lane highway crash data. They presented it in tabular form (i.e., AMF values are offered for specific grades); however, a best-fit trend line is shown in the figure. Although not derived from freeway crash data, the Harwood AMF provides some validation of the Milton and Mannering AMF.

The Harwood AMF and the Milton and Mannering AMF can be described using Equation 2-13. The base condition for grade is 0 percent (i.e., flat). That is, flat freeway segments have a grade AMF equal to 1.0. The grade variable is an absolute value implying that the AMF has the same value, regardless of whether the grade is uphill or downhill.

$$AMF_g = (e^{b P_g} - 1.0) P_s + 1.0 \qquad \textbf{(2-13)}$$

where:

AMF_g = grade accident modification factor;
b = regression coefficient;
P_g = percent grade (absolute value), %; and
P_s = proportion of crashes to which the AMF applies.

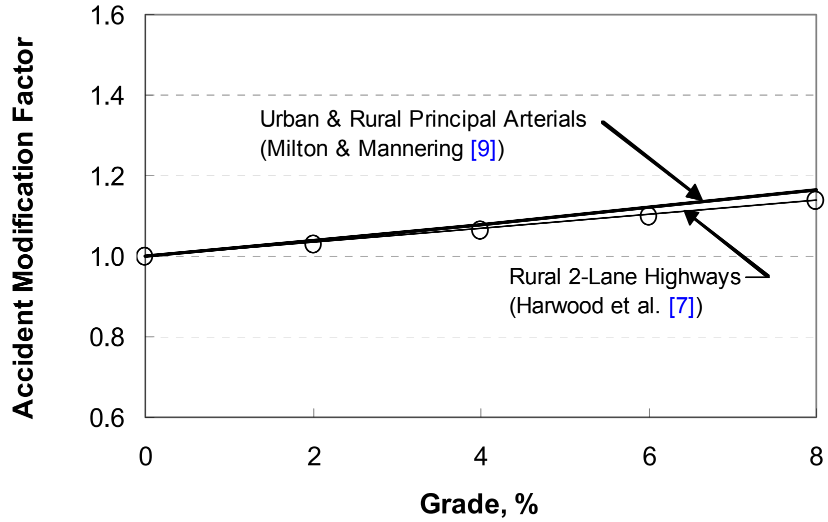

Figure 2-4. Grade AMF for Freeways.

The values of variables b and P_s in Equation 2-13 are provided in Table 2-4. For example, substitution of the values of 0.019 and 1.0 for b and P_s, respectively, results in the trend line shown in Figure 2-4 that is attributed to Milton and Mannering *(9)*.

Table 2-4. Coefficient Values for Grade on Freeways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s	Coefficient b
Milton & Mannering *(9)*	Urban & rural principal arterials	All	All	1.0	0.019
Harwood et al. *(7)*	Rural, 2-lane, undivided	All	All	1.0	0.016

The trend lines shown in Figure 2-4 indicate that the AMF for grade ranges from 1.0 for 0 percent to 1.14 for 8 percent. Based on this analysis, the Milton and Mannering AMF is reasoned to be applicable to urban and rural freeways.

Cross Section

Lane Width. A relationship between lane width and crash frequency was derived from the safety prediction models developed by several researchers. This derivation is described in the

Geometric Design section of Chapter 3. When applied to freeways, the following equation was developed for computing the lane width AMF for the base combination of lanes and median type (i.e., rural, four-lane freeway). The base condition lane width for this AMF is 12 ft.

$$AMF_{lw,b} = e^{-0.047\,(W_l\,-\,12)} \quad \textbf{(2-14)}$$

where:

$AMF_{lw,\,b}$ = lane width accident modification factor for the base combination of lanes and median type (i.e., rural, four-lane freeway).

Equation 2-14 can be combined with the following equation to obtain the lane width AMF for other combinations of lanes and median type.

$$AMF_i = (AMF_b - 1.0)\,\frac{P_i}{P_b} + 1.0 \quad \textbf{(2-15)}$$

where:

AMF_i = adjusted accident modification factor for lane and median combination i;
AMF_b = accident modification factor for base combination of lanes and median type;
P_i = proportion of "related" crashes that occur on roadways with lane and median combination i; and
P_b = proportion of "related" crashes that occur on roadways with the base combination of lanes and median type.

The values of P_i and P_b correspond to the crashes influenced by the AMF, and are both represented as a proportion of all crashes. The crash distribution used for P_i is that reflecting the crash history of the roadway type corresponding to b_i. The distribution used for P_b is that reflecting a specified base combination of lanes and median type. For this analysis, a rural, four-lane freeway was chosen to represent the base combination. The proportion of influential crashes for the base combination P_b is 0.37.

The AMF that results from the combination of Equations 2-14 and 2-15 is:

$$AMF_{lw,i} = (e^{-0.047\,(W_l\,-\,12)} - 1.0)\,\frac{P_i}{0.37} + 1.0 \quad \textbf{(2-16)}$$

It can be tailored to a desired number of lanes and median type by using the variable P_i. Values for this variable are listed in Table 2-5. The proportions in this table (and subsequent similar tables) were obtained from the crash database maintained by the Texas Department of Public Safety.

The AMF obtained from Equation 2-16 is illustrated in Figure 2-5. It is labeled "derived" and has a bold line weight. It is compared to the AMFs developed by two other researchers (see Table 3-7, Chapter 3). The trend is one of increasing AMF value with a reduction in lane width. The AMFs attributed to Hadi et al. (*2*) suggest that a 1 ft reduction in lane width results in a 35 to 40 percent increase in crashes. This increase is unrealistically large and is probably a result of colinearity among variables in the safety prediction model developed by Hadi et al.

In Figure 2-5, the AMF attributed to Harwood et al. (*10*) was developed for divided highways (instead of freeways) by an expert panel convened by Harwood. It is shown in the figure because it is in general agreement with the AMF computed using Equation 2-16 and offers evidence of bias in the two AMFs attributed to Hadi et al (*2*). It is rationalized that the design and operational differences between a freeway and a divided highway (as related solely to the correlation between lane width and crash frequency) are not so distinct as to invalidate this comparison.

Table 2-5. Crash Distribution for Freeway Lane Width AMF.

Facility Type	Area Type	Crash Type Subset	Through Lanes	Subset Proportion
Freeway	Rural	Single-vehicle run-off-road, same direction sideswipe	4	0.37
			6	0.49
	Urban	Single-vehicle run-off-road, same direction sideswipe	4	0.40
			6	0.40
			8	0.40
			10	0.43

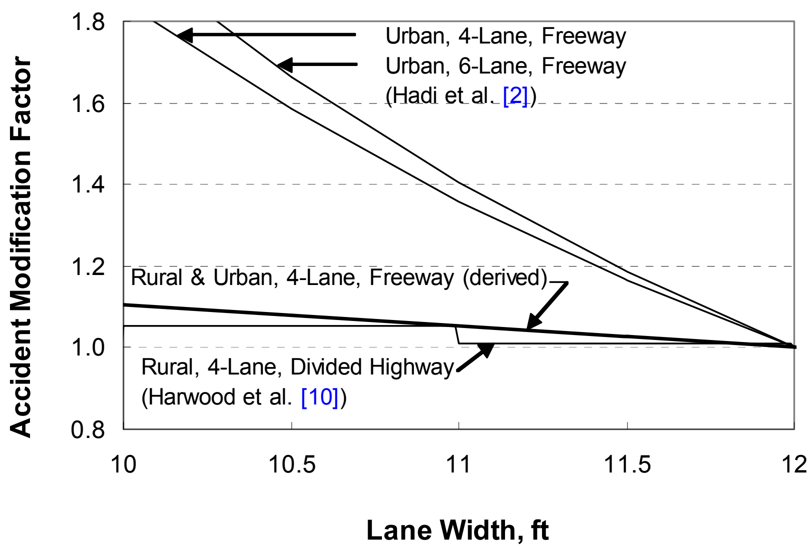

Figure 2-5. Lane Width AMF for Freeways.

Outside Shoulder Width. The following equation represents the shoulder width AMF for the base combination of lanes and median type (i.e., rural, four-lane freeway). It was derived in a similar manner as the lane width AMF. The base condition shoulder width is 10 ft.

$$AMF_{swo,b} = e^{-0.021\,(W_s - 10)} \quad \textbf{(2-17)}$$

where:

$AMF_{swo,\,b}$ = outside shoulder width accident modification factor for the base combination of lanes and median type (i.e., rural, four-lane freeway); and

W_s = outside shoulder width, ft.

Equation 2-17 can be combined with Equation 2-15 to obtain the outside shoulder width AMF for other combinations of lanes and median type. The proportion of influential crashes for the base combination P_b is 0.15. The resulting AMF is:

$$AMF_{swo,i} = \left(e^{-0.021\,(W_s - 10)} - 1.0 \right) \frac{P_i}{0.15} + 1.0 \tag{2-18}$$

It can be tailored to the desired number of lanes and median type by using the variable P_i. The value of this variable is selected from Table 2-6.

Table 2-6. Crash Distribution for Freeway Outside Shoulder Width AMF.

Area Type	Crash Type Subset	Through Lanes	Subset Proportion
Rural	Single-vehicle run-off-road, right side	4	0.15
		6	0.19
Urban	Single-vehicle run-off-road, right side	4	0.16
		6	0.14
		8	0.12
		10	0.13

The AMF obtained from Equation 2-18 is illustrated in Figure 2-6. It is labeled "derived" and has a bold line weight. It is compared to the AMFs developed by other researchers. The trend is one of increasing AMF value with a reduction in shoulder width. The AMFs attributed to Hadi et al. (*2*) and Knuiman et al. (*11*) are believed to be overly sensitive to shoulder width due to colinearity among the variables in their respective regression models.

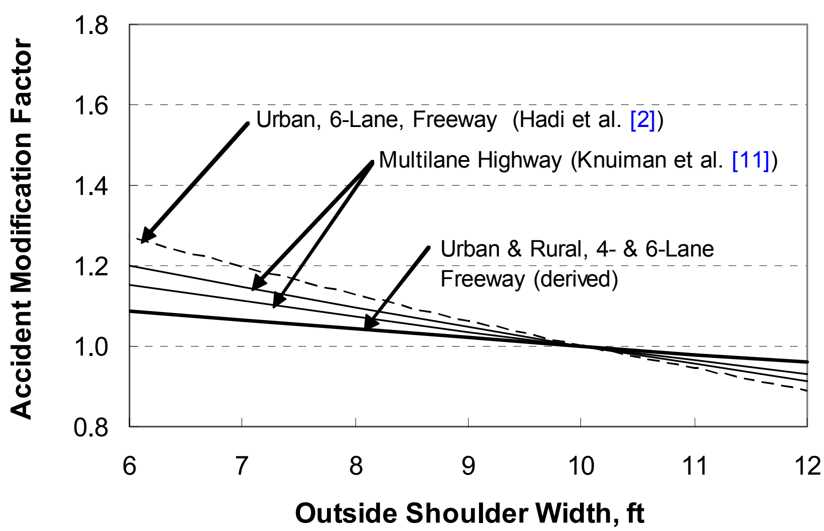

Figure 2-6. Outside Shoulder Width AMF for Freeways.

A shoulder width AMF was derived from the model developed by Wang et al. (*4*). However, it was very sensitive to shoulder width. In fact, it has a value of 2.5 when the shoulder width is 0.0 ft, which is much higher than the values obtained from other AMFs. An examination of the data evaluated by Wang et al. indicated that only 4 percent of the segments had shoulder widths less than 4 ft, yet these same segments accounted for 20 percent of the crashes. Hence, it is likely that there are unspecified factors that underlie this over-representation of crashes when categorized by shoulder width. For these reasons, the AMF derived from the Wang model was not considered further.

Inside Shoulder Width. A review of the literature revealed only one safety prediction model that included a variable for inside shoulder of a freeway. It was developed by Hadi et al. (*2*). However, when the regression coefficient associated with the inside-shoulder-width variable in this model was compared with coefficients for outside shoulder width (see Chapter 3), no significant difference was found between coefficients. Hence, Equation 2-17 is rationalized to be equally applicable to inside shoulder width. It was combined with Equation 2-15 to obtain the following inside shoulder width AMF:

$$AMF_{swi,i} = (e^{-0.021\,(W_{is} - W_{sb})} - 1.0)\frac{P_i}{0.15} + 1.0 \qquad \textbf{(2-19)}$$

where:

AMF_{swi} = inside shoulder width accident modification factor;
W_{is} = inside shoulder width, ft; and
W_{sb} = base inside shoulder width (= 4.0 ft for four lanes and 10.0 ft for six or more lanes).

The base condition for this AMF is a 4 ft inside shoulder width for four-lane freeways and a 10 ft width for freeways with six or more lanes. The proportion of influential crashes for the base combination P_b is 0.15. Equation 2-19 can be tailored to the desired number of lanes and median type by using the variable P_i. The value of this variable is selected from Table 2-7.

Table 2-7. Crash Distribution for Freeway Inside Shoulder Width AMF.

Area Type	Crash Type Subset	Through Lanes	Subset Proportion
Rural	Single-vehicle run-off-road, left side	4	0.15
		6	0.20
Urban	Single-vehicle run-off-road, left side	4	0.15
		6	0.13
		8	0.12
		10	0.11

The AMF obtained from Equation 2-19 for a four-lane rural freeway is illustrated in Figure 2-7. It is labeled "derived" and has a bold line weight. It is compared to the AMF developed by Hadi et al. (*2*). The trend is one of increasing AMF value with a reduction in shoulder width. The Hadi AMF is believed to be overly sensitive to shoulder width due to colinearity among the variables in their respective regression models. A weighted regression analysis was used to negate some of this bias. Details of this analysis are described in the Geometric Design section of Chapter 3.

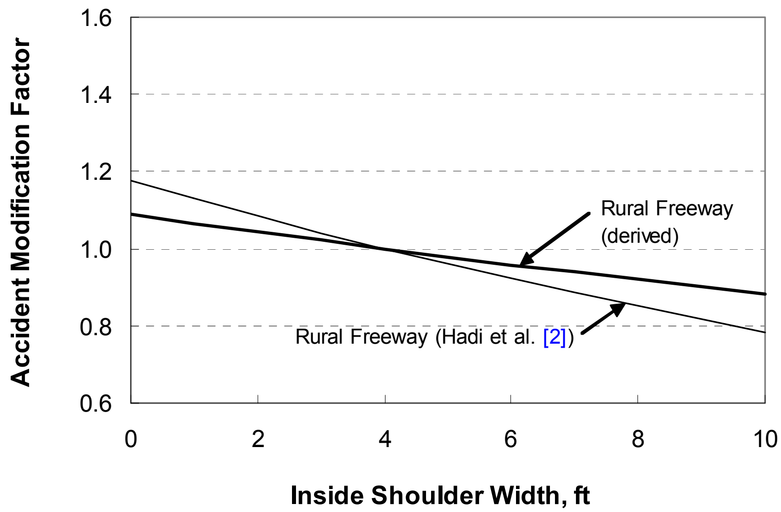

Figure 2-7. Inside Shoulder Width AMF for Freeways.

Median Width. An AMF for median width was developed from safety prediction models developed by Hadi et al. (*2*) and from data reported by Knuiman et al. (*11*). The Hadi AMF is based on models developed for four-lane and six-lane urban freeways. Knuiman et al. examined crash data for rural freeways, urban freeways, and major highways. The crash rate adjustment factors (analogous to AMFs) they computed are shown in Figure 2-8, as are the "best-fit" trend lines.

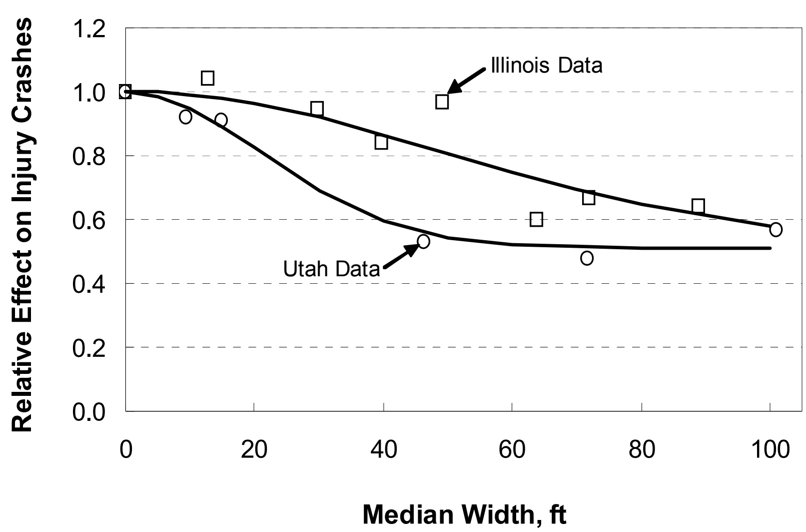

Figure 2-8. Relationship between Median Width and Severe Crash Frequency.

Equation 2-20 was developed to represent the AMFs derived from the literature. The base condition for this AMF is a median width of 76 ft for depressed medians and 24 ft for surfaced (i.e., flush-paved) medians. The coefficients used in this equation are listed in columns 4 through 6 of Table 2-8.

$$AMF_{mw} = \frac{b_0\,(e^{b_1 W_m^{b_2}} - 1.0) + 1.0}{b_0\,(e^{b_1 W_{mb}^{b_2}} - 1.0) + 1.0} \qquad \textbf{(2-20)}$$

where:

AMF_{mw} = median width accident modification factor;
W_{mb} = base median width (24 ft for surfaced median; 76 ft for depressed median), ft; and
W_m = median width, ft.

Table 2-8. Coefficient Values for Median Width on Freeways.

Model Source	Roadway Type	Crash Severity	Base Coefficients			Equivalent b_1 Coeff.	
			b_0	b_1	b_2	W_m< 40ft	W_m> 40ft
Hadi et al. (*2*)	Urban, 4-lane, freeway	Injury	1.0	-0.060	0.5	-0.060	-0.060
	Urban, 6-lane, freeway	Injury	1.0	-0.037	0.5	-0.037	-0.037
Knuiman et al. (*11*)	Urban & rural, 4-lane, freeway (Utah)	Severe	0.488	-0.00111	2.0	-0.153	-0.055
	Urban & rural, 4-lane, freeway (Illinois)	Severe	0.483	-0.00020	2.0	-0.042	-0.111
					Weighted Average:	**-0.046**	**-0.050**

Figure 2-9 illustrates the AMFs listed in Table 2-8 for four-lane freeways. They have a base condition median width of 24 ft. Similar trends are obtained for a base width of 76 ft. Two of the AMFs are in good agreement. The AMF from Knuiman et al. using Utah data is less in agreement and indicates a greater sensitivity to median width for widths of 40 ft or less.

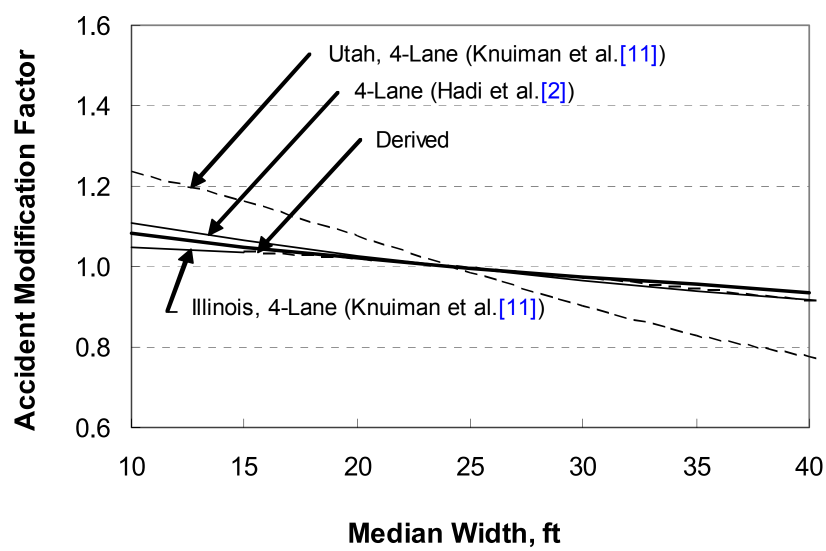

Figure 2-9. Median Width AMF for Freeways.

The derived AMF relationship is shown in Figure 2-9. It is based on Equation 2-20 with b_0 = 1.0, b_2 = 0.5, and the weighted average of the equivalent b_1 coefficients listed in columns 7 and 8 of Table 2-8 (where the weight w for each coefficient is equal to its reciprocal squared [i.e., $w = 1/b^2$]). The equivalent b_1 coefficient for each model source was derived from a regression analysis using Equation 2-20 with $b_0 = 1.0$ and $b_2 = 0.5$ for median widths ranging from 10 to 40 ft. The analysis was repeated for median widths ranging from 40 to 80 ft. The AMFs obtained from Equation 2-20 (using the base coefficients) served as the dependent variable.

Shoulder Rumble Strips. Griffith (*12*) investigated the correlation between the presence of continuous, rolled-in rumble strips and crash frequency on urban and rural freeways in California and Illinois. The focus of his examination was single-vehicle run-off-road crashes. The reported crash data were used to calculate a rumble strip AMF. These calculations are shown in Table 2-9.

Table 2-9. AMFs for Shoulder Rumble Strips on Freeways.

Data Source	State	Crash Severity	Treated Site Crashes [1]		Comparison Site Crashes		Base AMF (f_{rs})[2]	Standard Deviation[3]
			After	Before	After	Before		
Griffith (*12*)	Illinois	All	1895	2801	1833	2288	0.84	0.036
	California	All	469	579	364	417	0.93	0.088
	Combined	All	2364	3380	2197	2705	0.86	0.034
	Illinois	Injury	877	1135	765	874	**0.88**	0.059

Notes:
1 - Analysis applies to single-vehicle run-off-road crashes.
2 - $f_{rs} = After_{treated} \times Before_{comp} / (Before_{treated} \times After_{comp})$.
3 - Standard deviation = $f_{rs} \times (1/After_{treated} + 1/Before_{comp} + 1/Before_{treated} + 1/After_{comp})^{0.5}$.

The shoulder rumble strip AMF can be computed using Equation 2-21. It is based on the overall base AMF for severe crashes in Table 2-9. For Texas applications, it requires the appropriate crash distribution proportion for the roadway type of interest. These proportions are listed in Table 2-10.

$$AMF_{rs} = (0.88 - 1.0)\, P_i + 1.0 \qquad \textbf{(2-21)}$$

where:
AMF_{rs} = shoulder rumble strip accident modification factor; and
P_i = proportion of influential crashes that occur on roadway type i (from Table 2-10).

Table 2-10. Crash Distribution for Freeway Shoulder Rumble Strip AMF.

Area Type	Crash Type Subset	Through Lanes	Subset Proportion
Rural	Single-vehicle run-off-road, either side	4	0.30
		6	0.39
Urban	Single-vehicle run-off-road, either side	4	0.31
		6	0.27
		8	0.24
		10	0.25

Roadside Design

This section describes AMFs related to the roadside design of a freeway segment. Topics specifically addressed are listed in Table 2-11. Many roadside design components or elements that are not listed in this table (e.g., ditch shape) are also likely to have some correlation with severe crash frequency on the freeway segment. However, a review of the literature did not reveal that their effect has been quantified by previous research. The list of available AMFs for freeway roadside design is likely to increase as new research in this area is undertaken.

Table 2-11. AMFs Related to Roadside Design of Freeways.

Section	Accident Modification Factor
Cross section	Utility pole density
Appurtenances	see text

AMFs for roadside safety appurtenances are not described in this chapter because they generally do not exist. The safety literature related to roadside appurtenances has focused on the information needed to evaluate the cost-effectiveness of installing individual appurtenances at specific locations. The information is very detailed due to the design and operational complexity of various appurtenances and the influence of site-specific conditions on their performance. Moreover, safety appurtenances may sometimes increase crash frequency while, more importantly, reducing the severity of the crash. For these reasons, the safety benefits derived from an appurtenance are typically estimated on an individual, case-by-case basis using the techniques described in the *Roadside Design Guide* (*13*).

Cross Section

This subsection is devoted to the presentation of AMFs related to the roadside cross section of a freeway. A review of the literature indicates that horizontal clearance, side slope, utility pole offset, and bridge width may have some influence on crash frequency. However, these correlations have not been quantified for freeways. Rather, the focus of previous research has been on rural, two-lane highways. The one exception is utility pole offset. This research included crashes on urban and rural divided highways in the database. This subsection describes an AMF for utility pole offset and "density" (where pole density relates to pole frequency per unit length of roadway).

The relationship between utility pole offset, pole density, and crash frequency was evaluated by Zegeer and Parker (*14*). They developed a safety prediction model using data from four states. The model included average pole offset, traffic volume, and pole density as independent variables. The AMF derived from this model is described in Equation 2-22. Details of the model are described in Table 2-12. The base condition is 25 poles/mi and a pole offset of 30 ft.

$$AMF_{pd} = (f_p - 1.0)\, P_s + 1.0 \qquad \textbf{(2-22)}$$

with,

$$f_p = \frac{(0.0000984\,ADT + 0.0354\,D_p)\,W_o^{-0.6} - 0.04}{0.0000128\,ADT + 0.075} \qquad \textbf{(2-23)}$$

where:

AMF_{pd} = utility pole accident modification factor;
D_p = utility pole density (two-way total), poles/mi; and
W_o = average pole offset from nearest edge of traveled way, ft.

Table 2-12. Coefficient Values for Utility Pole Density along Freeways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s
Zegeer & Parker *(14)*	Urban and rural roads	All	Single-vehicle collision with pole	0.022

The subset proportions appropriate for Texas freeway applications are provided in Table 2-13. Equation 2-22 yields an AMF that ranges from 1.08 at a pole density of 50 poles/mi and offset of 10 ft to 0.99 at a pole density of 10 poles/mi and offset of 30 ft. The AMF is more sensitive to pole offset than it is to density or average daily traffic volume.

Table 2-13. Crash Distribution for Freeway Utility Pole Density AMF.

Area Type	Crash Type Subset	Through Lanes	Subset Proportion
Rural	Single-vehicle collision with pole	4	0.030
		6	0.038
Urban	Single-vehicle collision with pole	4	0.046
		6	0.029
		8	0.016
		10	0.012

Appurtenances

As discussed previously, AMFs for roadside safety appurtenances are not described in this chapter. The safety literature related to roadside appurtenances has focused on the information needed to evaluate the cost-effectiveness of installing individual appurtenances at specific locations. Most of the research conducted in this area has been incorporated into a comprehensive procedure for evaluating appurtenances on a case-by-case basis. This procedure is outlined in a report by Mak and Sicking *(15)* and automated in the *Roadside Safety Analysis Program* (RSAP) *(16)*.

RSAP can be used to evaluate alternative roadside safety appurtenances on individual road segments. The program accepts as input information about the road segment geometry and traffic characteristics. It also allows the analyst to describe the roadside cross section, location of fixed objects, and safety appurtenance design. Table 2-14 summarizes the various RSAP inputs. The output from RSAP includes an estimate of annual crash frequency as well as the road-user costs associated with these crashes. The crash reduction potential realized by adding a roadside safety appurtenance (or changing the roadside cross section) can be estimated by specifying the changed condition as an "alternative."

Table 2-14. RSAP Input Data Requirements.

Design Category	Design Component	Design Element	
General	--	Area type (urban/rural) One-way/two-way Segment length	Functional class Speed limit
Traffic characteristics	--	Traffic volume (ADT) Traffic growth factor	Truck percentage
Geometric design	Horizontal alignment	Direction of curve	Radius
	Vertical alignment	Grade	
	Cross section	Divided/undivided Lane width Median type	Number of lanes Shoulder width Median width
Roadside design	Cross section	Foreslope Parallel ditches	Backslope Intersecting slopes
	Fixed object	Offset Type (wood pole, headwall, etc.) Width	Side of roadway Spacing
	Safety appurtenances	Offset Type (barrier, cushion, etc.) Length	Side of roadway Flare rate Width

Other Adjustment Factors

This section describes AMFs related to features of the freeway that are not categorized as related to geometric design or roadside design. At this time, the only AMF addressed in this section is speed limit.

Two safety prediction models developed for freeways in urban and rural settings were found to have a variable for speed limit. One of these models was developed by Hadi et al. (*2*). Another was that developed by Milton and Mannering (*9*). A generalized AMF form is shown in Equation 2-24 for a base speed limit of 55 mph. It can be used to represent the AMF obtained from both of the two prediction models by proper substitution of the coefficient *b* from Table 2-15.

$$AMF_{sl} = e^{b(V - 55)} \qquad \textbf{(2-24)}$$

where:

AMF_{sl} = speed limit accident modification factor; and
V = speed limit, mph.

The two AMFs are compared in Figure 2-10. The two coefficients reported by Milton and Mannering are so similar that they plot as one trend line. The trends in the figure indicate that higher speed limits are associated with fewer severe crashes. Although not shown in the figure, similar trends have been found in safety prediction models developed for urban streets. It is likely that an increase in speed limit does not make the freeway safer; rather, it probably indicates that freeways with a higher speed limit tend to have a more generous design.

Table 2-15. Coefficient Values for Speed Limit on Freeways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Coefficient *b*
Hadi et al. (*2*)	Urban, 4-lane, freeway	Injury	All	-0.0154
Milton & Mannering (*9*)	Urban & rural, principal arterials	All	All	-0.0111 [1] -0.0103
			Weighted Average:	**-0.012**

Note:
1 - Separate safety prediction models were developed for data from principal arterials in the eastern and western half of Washington State.

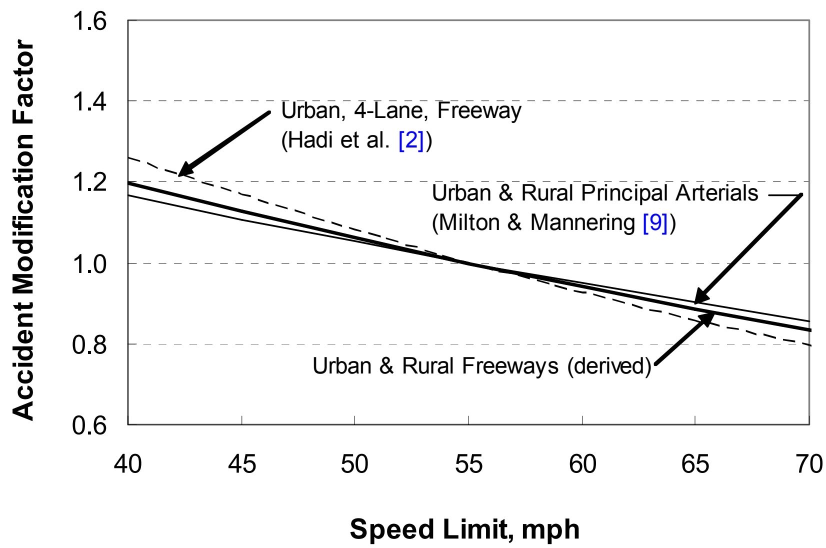

Figure 2-10. Speed Limit AMF for Freeways.

The derived AMF relationship is shown in Figure 2-10. It uses the weighted average of the *b* coefficients listed in Table 2-15 (where the weight *w* for each coefficient is equal to its reciprocal squared [i.e., $w = 1/b^2$]). There is insufficient data to determine if this coefficient varies by number-of-lanes or area type. Hence, the derived AMF is offered as approximate for all lanes and area types until new information becomes available.

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Chapter 3
Rural Highways

Page

INTRODUCTION

Rural highways provide a high speed network of roadways that serve traffic movements between urban areas. Between densely populated metropolitan areas, these highways often have a divided, multilane cross section. Highways serving smaller towns or shorter trip lengths typically have lower volume and a two-lane undivided cross section. About one-third of the vehicle miles of travel in the state of Texas occur on rural highways, yet these highways are associated with about one-half of all fatal crashes. In general, crashes on rural highways tend to be more severe than those on urban streets, due largely to the higher speeds associated with rural highways. For these reasons, efforts to accommodate "desirable" design elements in rural highway design, and to provide a forgiving roadside, are often cost-effective.

The development of a safe, efficient, and economical rural highway design typically reflects the consideration of a variety of design alternatives. A variety of techniques exist for estimating the operational benefits of alternatives; many are automated through software tools. Techniques for estimating construction and right-of-way costs are also available to the designer. Unfortunately, techniques for estimating the safety benefits of alternative designs are not as readily available. This chapter summarizes information in the literature that can be used to estimate the crash frequency associated with various highway design alternatives.

Objective

The objective of this chapter is to synthesize information in the literature that quantitatively describes the relationship between various rural highway design components and safety. This information is intended to provide a basis for the development of a procedure for estimating the safety benefit of alternative designs. This procedure is documented in Chapter 3 of the *Roadway Safety Design Workbook* (*1*).

The presentation consists of an examination of safety prediction models and accident modification factors (AMFs). Safety prediction models provide an estimate of the expected crash frequency for a typical highway segment. They include variables for traffic volume and segment length. They also include variables for other factors considered to be correlated with crash frequency (e.g., median type, number of lanes, etc.). One or more AMFs can be multiplied by the expected crash frequency obtained from the prediction model to produce an estimate of the expected crash frequency for a specific highway segment.

Scope

This chapter addresses the safety of rural highway segments. It does not address the safety of intersections on these highways. For this reason, the crashes addressed herein are referred to as "mid-block" crashes. Crashes that occur at, or are related to, rural intersections are the subject of Chapter 6.

When available, safety relationships that estimate the frequency of *severe* (i.e., injury or fatal) crashes are given preference for inclusion in this document. This preference is due to a wide variation in reporting threshold among cities and states. This variation complicates the extrapolation of crash trends found in one location to another location. Moreover, it can confound the development of safety prediction models using data from multiple agencies. Reporting threshold is strongly correlated with the number of property-damage-only (PDO) crashes found in a crash database. Agencies with a high reporting threshold include relatively few PDO crashes in their database and vice versa. As a consequence, the total crash frequency for a given roadway will be low if it is located in an area with a high reporting threshold. In contrast, this same roadway will have a high crash frequency if it is located in an area with a low reporting threshold. This problem is minimized when crash data analyses, comparisons, and models are based on data pertaining only to severe crashes.

Overview

This chapter documents a review of the literature related to rural highway safety. The focus is on quantitative information that relates severe crash frequency to various geometric design components of the highway. The review is not intended to be comprehensive in the context of referencing all works that discuss rural highway safety. Rather, the information presented herein is judged to be the most current information that is relevant to highway design in Texas. It is also judged to be the most reliable based on a review of the statistical analysis techniques used and the explanation of trends.

Where appropriate, the safety relationships reported in the literature are compared herein, with some interpretation offered to explain any differences noted. The relationships are typically presented as reported in the literature; however, the names or the units of some variables have been changed to facilitate their uniform presentation in this chapter.

This chapter is envisioned to be useful to design engineers who desire a more complete understanding of the relationship between various highway design components and severe crash frequency. As previously noted, it is also intended to serve as the basis for the development of the safety evaluation procedure described in Chapter 3 of the *Roadway Safety Design Workbook* (*1*).

This chapter consists of two main parts. In the first part to follow, several safety prediction models reported in the literature are described. In the next part, accident modification factors are described. Within this part, the various factors examined are organized into the following categories: roadway geometric design, roadside design, and "other" factors.

SAFETY PREDICTION MODELS

Described in this part of the chapter are several models that were developed to estimate the expected frequency of crashes on rural highway segments. Two of the models described were specifically developed for rural two-lane highway segments. Two other models were developed for multilane highway segments. After the models are summarized, they are then compared graphically for a range of common input conditions.

Rural Two-Lane Highways

This section addresses safety prediction models for rural two-lane highways. Two sets of models are described and are identified by the names of their developers. They include:

- Hadi Model
- Vogt and Bared Model

Hadi Model

The model described in this subsection is derived from the equations reported by Hadi et al. (*2*). They used regression analysis to calibrate a set of safety prediction models using data from Florida roadways. The models are categorized by crash severity, area type (i.e., urban, rural), and number of through lanes. The rural two-lane highway model developed by Hadi et al. is:

$$C = 0.001547\ ADT^{0.868}\ L^{0.8157}\ e^{B_1} \qquad \textbf{(3-1)}$$

with,

$$B_1 = -0.0787\ W_l - 0.0108\ V + 0.0601\ N_i - 0.021 \times 2\ W_s \qquad \textbf{(3-2)}$$

where:
C = frequency of mid-block injury crashes on rural two-lane highways, crashes/yr;
ADT = average daily traffic, veh/d;
L = highway segment length, mi;
W_l = lane width, ft;
V = speed limit, mph;
N_i = number of intersections on highway segment; and
W_s = shoulder width (paved or unpaved), ft.

The models developed by Hadi et al. (*2*) estimate injury crash frequency only; they do not include PDO or fatal crash frequency. Hadi et al. prepared separate models for estimating total crash frequency and fatal crash frequency. Analysis of these models indicates that the estimates obtained from Equation 3-1 should be inflated by 5 percent (i.e., multiplied by 1.05) to obtain an estimate of severe crash frequency.

The sign of the constant associated with any single variable in the model's linear terms (i.e., those variables in Equation 3-2) indicates the correlation between a change in the variable value and injury crash frequency. For example, the negative sign of the constant associated with speed limit *V* suggests that injury crash frequency is lower on roads with a higher speed limit. In this case, the trend is an aberration of the modeling approach because injuries are generally recognized to increase with speed. The trend is likely a reflection of the fact that higher speed roads are typically built to have more generous design element sizes and more forgiving roadside safety features. Hence, the speed limit constant is acting as a surrogate for several unspecified design elements and features.

The characteristics of the data used by Hadi et al. (*2*) are summarized in Table 3-1. They used four years of crash data to develop their safety prediction models. They did not report the number of road segments reflected in the database. However, the road segments used were on the Florida state highway system.

Table 3-1. Database Characteristics for Rural Two-Lane Highway Safety Prediction Models.

Model Developers	Database Characteristics			
	Data Source	Years of Crash Data	Number of Road Segments	Total Section Length, mi
Hadi et al. (*2*)	Florida DOT	4	not available	not available
Vogt & Bared (*3*)	Minnesota DOT	5	619	706
	Washington DOT	3	712	534

Vogt and Bared Model

Vogt and Bared (*3*) investigated the relationships between crash frequency and various rural highway geometry and operational attributes. They developed a safety prediction model for predicting the frequency of severe crashes on rural two-lane highways. The form of this model is:

$$C = 0.0005197 \; ADT \; L \; e^{B_1 + B_2} \tag{3-3}$$

with,

$$B_1 = -0.1306 \; W_l - 0.0784 \, W_s + 0.0598 \, R_{haz} + 0.0062 \, D_d \tag{3-4}$$

$$B_2 = 0.0457 \; D_r + 0.4694 \, V_r + 0.4309 \, I_{Wash} \tag{3-5}$$

where:

C = frequency of severe crashes on two-lane highways, crashes/yr;
R_{haz} = roadside hazard rating (1, 2, ..., 7; where 1 represents best and 7 is worst from a safety standpoint);
D_d = driveway density, driveways/mi;
D_r = weighted average degree of curvature, degrees/ft;
V_r = weighted average vertical curvature, %/ft; and
I_{Wash} = indicator variable for location of segment (1 for Washington State, 0 for Minnesota).

The characteristics of the data used by Vogt and Bared (*3*) are summarized in Table 3-1. The database they used to develop the safety prediction model represents five years of crash data for 619 highway segments in Minnesota and three years of data for 712 segments in Washington State.

Rural Multilane Highways

This section addresses safety prediction models for rural multilane divided highways. Two sets of models are described and are identified by the names of their developers. They include:

- Hadi Model
- Wang Model

Hadi Model

The model described in this subsection is developed from the equations reported by Hadi et al. (*2*). They used regression analysis to calibrate a set of safety prediction models using data from Florida roadways. The models are categorized by crash severity, area type (i.e., urban, rural), and number of through lanes. The four-lane divided highway model developed by Hadi et al. is:

$$C = 0.000956\ ADT^{0.6919}\ L^{0.6288}\ e^{0.1973\,N_i} \tag{3-6}$$

where:

C = frequency of mid-block injury crashes on four-lane divided highways, crashes/yr.

The models developed by Hadi et al. (*2*) estimate injury crash frequency only; they do not include PDO or fatal crash frequency. Hadi et al. prepared separate models for estimating total crash frequency and fatal crash frequency. Analysis of these models indicates that the estimates obtained from Equation 3-1 should be inflated by 5 percent (i.e., multiplied by 1.05) to obtain an estimate of severe crash frequency.

The characteristics of the data used by Hadi et al. (*2*) are summarized in Table 3-2. They used four years of crash data to develop their safety prediction models. They did not report the number of road segments reflected in the database. However, the road segments used were on the Florida state highway system.

Table 3-2. Database Characteristics for Rural Multilane Highway Safety Prediction Models.

Model Developers	Database Characteristics				
	Data Source	Years of Crash Data	Number of Road Segments	Total Section Length, mi	Number of Through Lanes
Hadi et al. (*2*)	Florida DOT	4	not available	not available	4
Wang et al. (*4*)	Minnesota DOT	5	622	432	3, 4

Wang Model

Wang et al. (*4*) developed a safety prediction model for rural multilane divided highways using crash and geometry data from Minnesota. Equations 3-7 and 3-8 describe this model.

$$C_{RH} = 0.000233\ ADT^{1.073}\ L^{1.073}\ e^{B_{RH}}\ (1 - 0.01\ PDO) \qquad \textbf{(3-7)}$$

with,

$$\begin{aligned} B_{RH} = {} & 0.131\ R_{haz} - 0.151\ I_{ac} + 0.034\ D_d + 0.163\ D_{it} + 0.052\ D_{wo} \\ & - 0.572\ I_{fc} - 0.094\ W_s - 0.003\ W_m + 0.429\ I_{at} \end{aligned} \qquad \textbf{(3-8)}$$

where:

C_{RH} = frequency of severe mid-ramp crashes on rural highways, crashes/yr;
R_{haz} = roadside hazard rating (1, 2, ..., 7; where 1 represents best and 7 is worst from a safety standpoint);
I_{ac} = access control (1 for partial control, 0 for no control);
D_d = driveway density, driveways/mi;
D_{it} = density of unsignalized intersections with turn lanes, intersections/mi;
D_{wo} = density of unsignalized intersections without turn lanes, intersections/mi;
I_{fc} = functional class (1 for rural principal arterial, 0 otherwise);
W_s = outside shoulder width, ft;
I_{at} = area location type (1 for highways in a rural municipality, 0 otherwise); and
PDO = percent property-damage-only crashes on rural multilane divided highways (= 62.5).

As indicated in Equation 3-8, the model contains a variable describing the relative degree of hazard associated with the roadside. Specifically, a roadside hazard rating is used to describe the safety of the highway cross section. A rating of "1" is assigned to roadsides with horizontal clearances of 30 ft or more and side slopes of 1V:4H or flatter.

Equation 3-7 was originally derived to predict total crashes (i.e., PDO plus injury and fatal). An adjustment multiplier of "1 − 0.01 *PDO*" was added to this equation to convert the prediction into severe crash frequency. This term requires an estimate of the percentage of PDO crashes. This percentage is estimated at 62.5 percent for rural multilane divided highways in Minnesota (*5*).

The characteristics of the data used by Wang et al. (*4*) are summarized in Table 3-2. Their model is based on five years of crash data for 622 multilane highway segments.

Comparison of Crash Models

The crash prediction models described in the previous sections are compared in this section. The objective of this comparison is to determine which model or models are reasonable in their prediction of severe crash frequency. To facilitate this comparison, the models are examined over a range of traffic volume levels. The values of other model variables were set at typical values for rural highways. These values are listed in Table 3-3.

Table 3-3. Typical Values Used for Rural Highway Model Comparison.

Model Variable	Typical Value	Safety Prediction Model Developer			
		Hadi et al. (*2*)		Vogt & Bared (*3*)	Wang et al. (*4*)
		Two Lane	Four Lane	Two Lane	Four Lane
Outside shoulder width, ft	8	✔	--	✔	✔
Median width, ft	40	--	--	--	--
Lane width, ft	12	✔	--	✔	--
Speed limit, mph	55	✔	--	--	--
Roadside hazard rating	3	--	--	✔	✔
Intersections per mile	0	✔	✔	--	✔
Driveway density, driveways/mi	5	--	--	✔	✔
Average degree of curvature	0.0	--	--	✔	--
Average vertical curvature	0.0	--	--	✔	--
Access control	no control	--	--	--	✔
Functional class	principal arterial	--	--	--	✔
Area location type	non-municipal	--	--	--	✔

The expected severe crash frequencies obtained from the various models are shown in Figures 3-1 and 3-2. The model developer and facility type for a given trend line is indicated in each figure. The trend lines in these figures indicate that severe crash frequency increases in a nearly linear manner with traffic volume. In fact, the crash frequency on multilane highways increases at about the same rate as the crash frequency on two-lane highways.

ACCIDENT MODIFICATION FACTORS

This part of the chapter describes various accident modification factors that are related to the design of a rural highway. The various factors examined are organized into the following categories: roadway geometric design, roadside design, and "other" factors.

Geometric Design

This section describes AMFs related to the geometric design of a rural highway. Topics specifically addressed are listed in Table 3-4. Many geometric design components or elements are not listed in Table 3-4 (e.g., pavement cross slope) that are also likely to have some effect on severe crash frequency. However, a review of the literature did not reveal useful quantitative information describing these effects. The list of available AMFs for rural highway geometric design is likely to increase as new research in this area is undertaken.

In some instances, an AMF is derived from a safety prediction model as the ratio of "segment crash frequency with a changed condition" to "segment crash frequency without the change." In other instances, the AMF is obtained from a before-after study. Occasionally, crash data reported in the literature were used to derive an AMF.

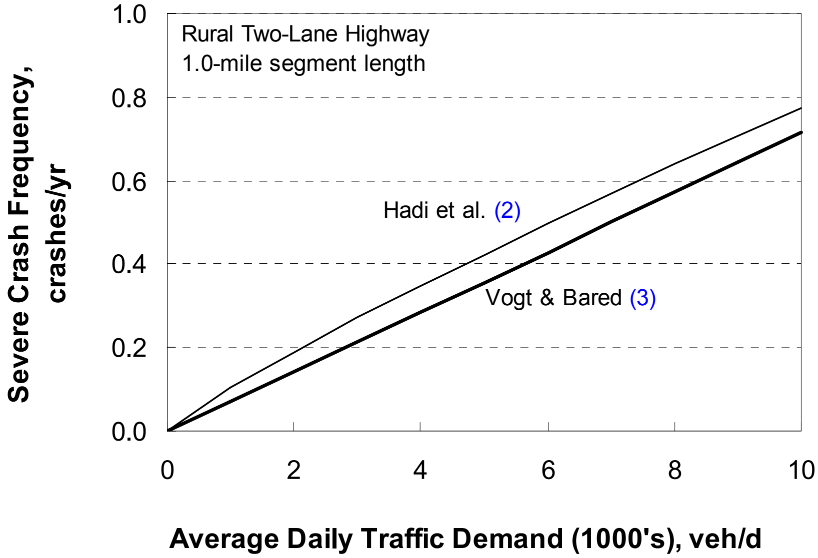


Figure 3-1. Severe Crash Frequency for Rural Two-Lane Highways.

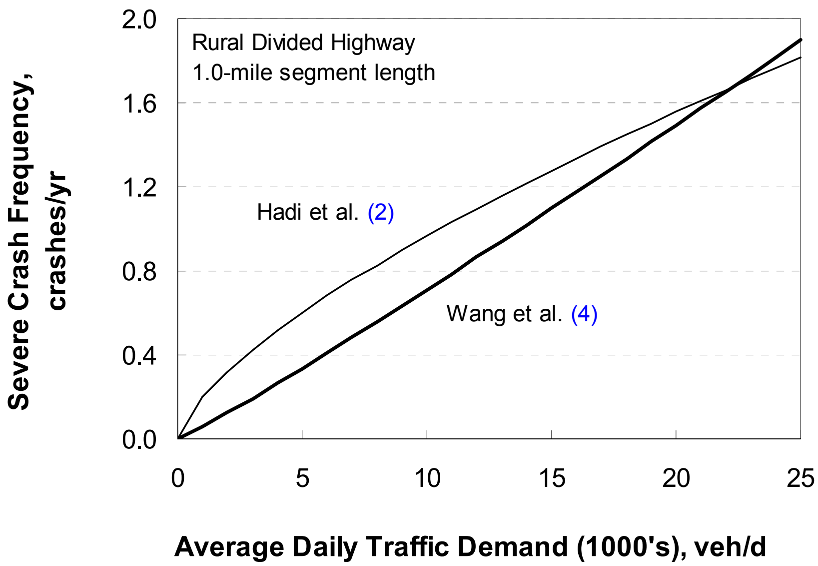


Figure 3-2. Severe Crash Frequency for Rural Multilane Divided Highways.

Table 3-4. AMFs Related to Geometric Design of Rural Highways.

Section	Accident Modification Factor		
Horizontal alignment	Horizontal curve radius	Spiral transition curve	
Vertical alignment	Grade		
Cross section	Lane width Median width TWLTL median type [1]	Outside shoulder width Shoulder rumble strips Superelevation	Inside shoulder width Centerline rumble strip Passing lanes

Note:
1 - TWLTL: two-way left-turn lane.

All of the AMFs described in this section are developed to yield a value of 1.0 when the associated design component or element represents a "typical" condition. Deviation from this base condition to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Horizontal Alignment

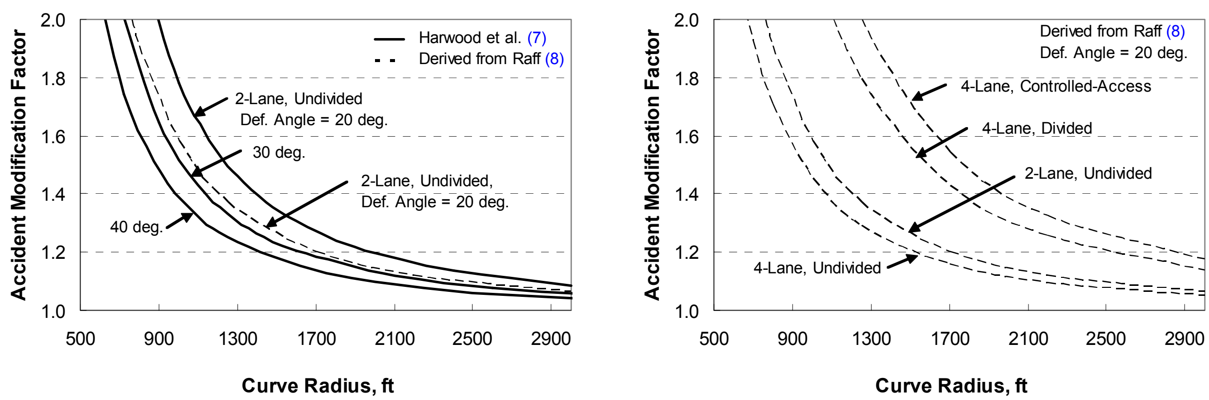
Horizontal Curve Radius. Zegeer et al. (*6*) developed a model for determining the safety of horizontal curves on rural two-lane highways. This model was subsequently modified by Harwood et al. (*7*) to produce a horizontal curve AMF. This AMF is shown in Equation 3-9.

$$AMF_{cr} = \frac{1.55\,L_c + \dfrac{80.2}{R} - 0.012\,I_s}{1.55\,L_c} \qquad \textbf{(3-9)}$$

where:

AMF_{cr} = horizontal curve accident modification factor;
L_c = length of horizontal curve (= $I_c \times R$ / 5280 / 57.3), mi;
I_c = curve deflection angle, degrees;
R = curve radius, ft; and
I_s = presence of a spiral transition curve (1 if a spiral transition is present, 0 otherwise).

The AMF computed using Equation 3-9 is shown in Figure 3-3a (using a solid trend line) for a typical range of two-lane highway curve radii and deflection angles. The trend lines indicate that the AMF converges to 1.0 as the curve radius increases. For a given radius, larger deflection angles correspond to lower AMF values. This trend suggests that most curve crashes are associated with the curve entry maneuver such that curves with a larger deflection angle (for a given radius) are easier for the driver to detect in advance and safely negotiate the entry maneuver.



a. Two-Lane Highways. *b. Multilane Highways.*

Figure 3-3. Horizontal Curve AMF for Rural Highways.

The numerator of Equation 3-9, when multiplied by the volume of vehicles per year (in millions), can be used to estimate the total number of annual crashes on a horizontal curve on a two-lane highway, as developed by Zegeer et al. (*6*). Their model also included an adjustment factor for total roadway width (i.e., shoulder plus lane width). The model suggested by the numerator of Equation 3-9 is applicable to roads with a typical, total width of 30 ft. Thus, the constant "1.55" represents the crash rate *CR* (in crashes/million-vehicle-miles [crashes/mvm]) for a tangent section of highway. More generally, for curves without spiral transitions, this model can also be written as:

$$CR = b_0 + \frac{b_1^2}{I_c \, R^2} \qquad \textbf{(3-10)}$$

where b_0 equals 1.55 and b_1 equals 4930.

Equation 3-10 can be converted to the following generalized AMF form by dividing both sides of the equation by the tangent crash rate term b_0:

$$AMF_{cr} = 1 + \frac{1}{I_c}\left(\frac{c_0}{R}\right)^2 \qquad \textbf{(3-11)}$$

where $c_0 = b_1 \times b_0^{-0.5}$.

Raff (*8*) examined curve crash rates for multilane and two-lane highways. He computed rates for four radius categories for each facility type. These rates are listed in Table 3-5.

The average curve deflection angles for each radius category were not reported by Raff (*8*). To compensate for this deficiency, geometric data for 29 rural, two-lane highway curves in six states were available from a report by Bonneson (*9*) to examine this relationship. An analysis of these data revealed the following relationship: $I_c = 11.3\ (5730/R)^{0.590}$. The coefficient of determination R^2 for this relationship is 0.50. This relationship was checked against curve data reported by Milton and Mannering (*10*) for rural minor arterials in eastern Washington State and found to yield very good agreement. However, its extrapolation to data reported for rural principal arterials in Washington State (i.e., multilane, divided highways) indicated that it overestimated the average deflection angle by 54 percent. Thus, the previous relationship was adjusted to yield the following equation for divided highways: $I_c = 7.30\ (5730/R)^{0.590}$. The deflection angle estimated for each radius using these equations is listed in Table 3-5.

Weighted linear regression was used with Equation 3-10 to quantify the relationship between crash rate and curve radius for the data in Table 3-5. The weight used for each radius category was the reported number of crashes associated with that category. The results of the analysis are reported in the last four columns of Table 3-5. Listed in the last row of the table are the regression constants that yield the equivalent of Equation 3-9. The resulting AMFs are shown in Figure 3-3b for a 20-degree deflection angle. The two-lane, undivided highway AMF obtained from the Raff (*8*) data and the Harwood et al. (*7*) AMF are compared in Figure 3-3a for a common, 20-degree deflection angle.

Table 3-5. Crash Rate Analysis for Various Rural Highway Cross Sections.

Road Type (Reference)	Variable	Radius, ft[1]				Statistics			
		3820	1270	716	477	b_o	b_1	c_o	R^2
Two-lane highway (8)	Deflection angle, deg	14.4	27.4	38.5	49.0	1.76	4530	3410	0.99
	Crash rate, crashes/mvm	1.6	2.5	2.8	3.5				
	Crashes	504	596	338	354				
Four-lane, undivided highway (8)	Deflection angle, deg	14.4	27.4	38.5	49.0	1.99	4250	3010	0.73
	Crash rate, crashes/mvm	1.9	2.6	3.3	1.2				
	Crashes	98	90	16	3				
Four-lane, divided highway (8)	Deflection angle, deg	9.3	17.7	24.9	31.6	1.40	5900	4980	0.98
	Crash rate, crashes/mvm	1.8	2.4	3.1	6.7				
	Crashes	95	65	5	12				
Four-lane, controlled-access (8)	Deflection angle, deg	9.3	17.7	24.9	--	1.20	6130	5590	0.96
	Crash rate, crashes/mvm	1.6	2.3	4.5	--				
	Crashes	180	162	38	--				
Two-lane highway (7)	--	--	--	--	--	1.55	4930	3960	--

Note:
1 - Radii listed represent the midpoint value for each radius category reported by Raff (8).

The comparison in Figure 3-3a suggests that the Raff AMF is in general agreement with the Harwood AMF. Differences between the two AMFs may be due to the use of estimated deflection angles. The trends in Figure 3-3b indicate that the four-lane, undivided roadway may have a slightly lower AMF value than the two-lane highway trend; however, the difference is small and may be attributable to differences in deflection angle present in Raff's database. A similar claim can be made when comparing the AMFs for divided and controlled-access highways. However, the large increase in AMF value for these two highway types, beyond that for the two-lane, undivided highway, cannot be explained by error in estimating the deflection angles.

Based on this analysis, the AMF developed by Harwood et al. (7) (i.e., Equation 3-9) is rationalized to be applicable to two-lane and four-lane, undivided highways. In contrast, the following AMF should be used for four-lane, divided highways:

$$AMF_{cr} = 1 + \frac{1}{I_c}\left(\frac{5800}{R}\right)^2 \quad \textbf{(3-12)}$$

where, the "5800" constant was computed by multiplying the constant c_0 of 4980 by the ratio of the c_0 constants for two-lane highways (i.e., 5800 = 4980 × 3960 / 3410). This adjustment is intended to maintain, in Equations 3-9 and 3-12, the relationship between the two-lane, undivided and four-lane, divided highways that is found in the Raff data.

Spiral Transition Curve. Equation 3-9 was used to derive an equation relating two-lane highway crash frequency and the presence of a spiral transition curve. The form of this equation is:

$$AMF_{sp} = \frac{1.55\, L_c + \dfrac{80.2}{R} - 0.012}{1.55\, L_c + \dfrac{80.2}{R}} \qquad \textbf{(3-13)}$$

where:

AMF_{sp} = spiral transition curve accident modification factor.

An examination of Equation 3-13 indicates that the spiral transition curve is associated with fewer crashes and that this effect varies with radius. The AMF value is about 0.95 for curves with a radius of 500 ft. It increases to 0.98 for a radius of 3000 ft. The literature review did not identify any research quantifying a similar relationship for spiral transitions on multilane highway curves.

Vertical Alignment

This subsection describes AMFs related to features of the highway's vertical alignment. At this time, the only AMF addressed in this subsection is grade.

The relationship between grade and crash frequency was derived from the safety prediction models developed by Milton and Mannering (*10*) and by Harwood et al. (*7*). The Milton and Mannering AMF is based on a mix of urban and rural multilane highways in Washington State. The highways were classified as principal arterials and had up to six lanes with ADTs of up to 18,000 veh/d per lane. In contrast, the AMF from Harwood et al. (*7*) is based on rural two-lane highways. They presented their AMF in tabular form. The two AMFs are compared in Figure 3-4. A best-fit trend line is shown for the tabulated data provided by Harwood et al. (*7*).

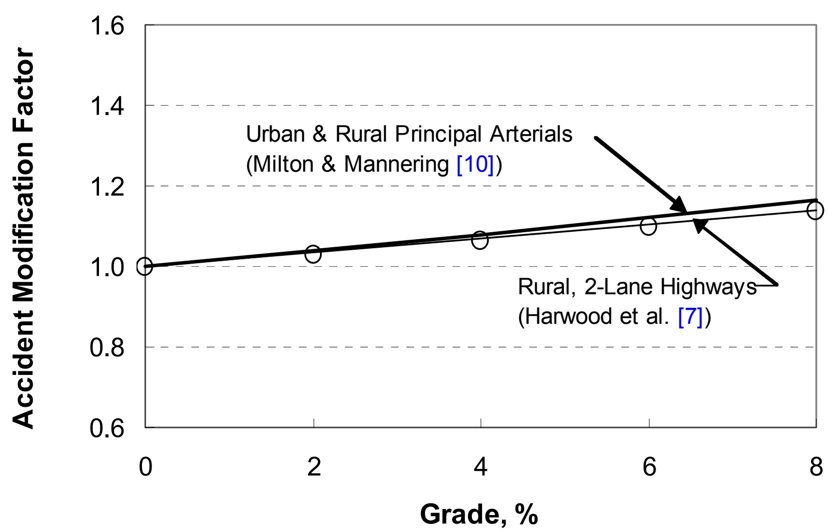


Figure 3-4. Grade AMF for Rural Highways.

The Harwood AMF and the Milton and Mannering AMF can be described using Equation 3-14. The structure of this equation implies that the base condition for grade is 0 percent (i.e., flat). That is, flat highway segments have a grade AMF equal to 1.0. It should also be noted that the grade variable is an absolute value implying that the AMF has the same value, regardless of whether the grade is uphill or downhill.

$$AMF_g = (e^{b P_g} - 1.0) P_s + 1.0 \qquad \textbf{(3-14)}$$

where:
AMF_g = grade accident modification factor;
b = regression coefficient;
P_g = percent grade (absolute value), %; and
P_s = proportion of crashes to which the AMF applies.

The values of variables b and P_s are provided in Table 3-6. For example, substitution of the values of 0.019 and 1.0 for b and P_s, respectively, results in the trend line shown in Figure 3-4 that is attributed to Milton and Mannering *(10)*.

Table 3-6. Coefficient Values for Grade on Rural Highways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s	Coefficient b
Milton & Mannering *(10)*	Urban & rural principal arterials	All	All	1.0	0.019
Harwood et al. *(7)*	Rural, 2-lane, undivided	All	All	1.0	0.016

The trend lines shown in Figure 3-4 indicate that the AMF for grade ranges from 1.0 for 0 percent to 1.14 for 8 percent. Based on this analysis, the Harwood and the Milton and Mannering AMFs are reasoned to be applicable to two-lane and multilane highways, respectively.

Cross Section

Lane Width. This section describes the analysis of various lane width and shoulder width AMFs derived from the literature. This analysis was broadened to include shoulder width because of its correlation with lane width. The findings from this analysis that relate to lane width are discussed in this subsection. The findings for shoulder width are discussed in the next subsection. Most of the AMFs derived from the literature can be represented by the following functional form, where the base lane width is expressed as 12 ft:

$$AMF_{lw,i} = e^{b_i (W_l - 12)} \qquad \textbf{(3-15)}$$

where:
$AMF_{lw,i}$ = lane width accident modification factor for roadway type i;
b_i = regression coefficient for roadway type i; and
W_l = lane width, ft.

Some AMFs found in the literature were not directly adaptable to the generalized form of Equation 3-15 (e.g., some specified discrete AMFs for specific lane widths). For these AMFs, log-linear regression was used with Equation 3-15 to quantify an equivalent value of b_i. The various coefficients identified in the literature are identified in column 8 of Table 3-7, as are the corresponding area type, road type, median type, and number of through lanes.

Table 3-7. Coefficient Analysis for Lane and Shoulder Width on Rural Highways.

Design Element	Area Type	Road Type	Median Type[1]	Thru. Lanes	Model Source	Crash Severity	Source Coeff. b_i[2, 3]	Proportion[4]		Base Coeff. b_b
								P_i	P_b	
Lane	Rural	Freeway	Restrictive	4	Hadi et al. (2)	Injury	-0.3070	0.37	0.37	-0.307
				6	Hadi et al. (2)	Injury	-0.3390	0.49	0.37	-0.245
		Highway	Restrictive	4	Harwood et al. (11)	All	<u>-0.029</u>	0.36	0.36	-0.029
			Non-Restrictive	2	Harwood et al. (7)	All	<u>-0.057</u>	0.42	0.36	-0.049
					Hadi et al. (2)	Injury	-0.0787	0.42	0.36	-0.067
					Vogt & Bared (3)	Severe	-0.1306	0.42	0.36	-0.111
				4	Harwood et al. (11)	All	<u>-0.043</u>	0.37	0.36	-0.042
	Urban	Street	Non-Restrictive	2	Hadi et al. (2)	Injury	-0.0489	0.25	0.24	-0.047
				4	Hadi et al. (2)	Injury	-0.1037	0.18	0.24	-0.141
Shoulder	Rural	Freeway	Restrictive	4	Hadi et al. (2)[5]	Injury	-0.0407	0.15	0.15	-0.041
		Freeway, Highway	Restrictive	4	Knuiman et al. (12)	All	-0.0352	0.16	0.16	-0.035
					Knuiman et al. (12)	All	-0.0460	0.16	0.16	-0.046
		Highway	Non-Restrictive	2	Harwood et al. (7)	All	<u>-0.028</u>	0.34	0.16	-0.013
					Vogt & Bared (3)	Severe	-0.0784	0.34	0.16	-0.036
	Urban	Freeway	Restrictive	6	Hadi et al. (2)	Injury	-0.0594	0.14	0.16	-0.068
		Street	Restrictive	4	Hadi et al. (2)	Injury	-0.0303	0.09	0.09	-0.030
			Mix	4	Harwood (13)	All	<u>-0.011</u>	0.10	0.09	-0.010
			Non-Rest.	2	Hadi et al. (2)	Injury	-0.0489	0.17	0.09	-0.025

Notes:

1 - Median type: non-restrictive - undivided, two-way left-turn lane, surfaced; restrictive - raised-curb, depressed.

2 - Source coefficient: coefficient obtained directly from the literature for a specified combination of area type, road type, median type, and through lanes.

3 - <u>Underlined</u> coefficients represent a best-fit exponential curve to the reported AMFs that were tabulated for a range of shoulder or lane widths by the researchers identified in the "Model Source" column.

4 - Proportions are obtained from the crash database maintained by the State of Texas.

5 - This coefficient applies to the inside shoulder; all other coefficients apply to the outside shoulder.

In a preliminary examination of the regression coefficients b_i in Table 3-7, it was noted that they tended to vary among the various area and road types. In recognition of this wide range, it was decided that the most appropriate method of AMF development was to combine all of the coefficients into one database and use linear regression to derive an equation for estimating b_i as a function of area type, road type, or both.

As noted by Harwood et al. (*7*), lane width tends to influence specific types of crashes on two-lane, undivided highways. These "related" crashes include: single-vehicle run-off-road, same direction sideswipe, and multiple vehicle opposite direction. It is likely that the multiple vehicle opposite direction crashes would not be influenced by lane width for depressed, or other types of restrictive, medians. Crashes related to shoulder width are most likely characterized as single-vehicle run-off-road crashes. More specifically, for outside shoulder width, the related crashes are likely the single-vehicle run-off-road on right side. Similarly, for inside shoulder width, the related crashes are likely single-vehicle run-off-road on left side. The proportion of these crash types P_i for the corresponding design element, area type, road type, median type, and through lanes are listed in column 9 of Table 3-7. These proportions were obtained from the crash database maintained by the State of Texas, Department of Public Safety.

Based on the assumption that the proportion of crashes P_i can explain much of the variation in the coefficients b_i due to median type and number of lanes, the following relationship was derived to facilitate the estimation of an AMF for various combinations of area type and road type:

$$AMF_i = (AMF_b - 1.0)\frac{P_i}{P_b} + 1.0 \qquad \textbf{(3-16)}$$

where:

AMF_i = adjusted accident modification factor for lane and median combination i;
AMF_b = accident modification factor for base combination of lanes and median type;
P_i = proportion of "related" crashes that occur on roadways with lane and median combination i; and
P_b = proportion of "related" crashes that occur on roadways with the base combination of lanes and median type.

The values of P_i and P_b correspond to the crashes influenced by the AMF, and are both represented as a proportion of all crashes. The crash distribution used for P_i is that reflecting the crash history of the roadway type corresponding to b_i. The distribution used for P_b is that reflecting a specified base combination of lanes and median type. For this analysis, a four-lane, depressed median highway was chosen to represent the base combination.

Equation 3-16 was combined with Equation 3-15 to obtain the following equation for estimating the "base coefficient" b_b for a four-lane, depressed median highway with a specified area type and road type:

$$b_b = \ln\left[(e^{b_i} - 1.0)\frac{P_b}{P_i} + 1.0\right] \qquad \textbf{(3-17)}$$

The change in lane (or shoulder) width is specified to be 1.0 ft for all lane and median combinations. This equation was applied to both the lane width and shoulder width coefficients b_i listed in Table 3-7 to estimate the base coefficient. Values of the base coefficient are listed in the last column of Table 3-7. Theoretically, these coefficients have the effect of median type and number of lanes removed and any systematic variability remaining can be explained by area type or road type.

A regression analysis was undertaken to determine if area type or road type could explain any of the variability in the base coefficients. Weighted regression was used with the weight w_j for each variable equal to the reciprocal of the square of the corresponding base coefficient (i.e., $w_j = 1/ b_{b,j}^2$). The following equation was developed as a result of this analysis:

$$b_b = -0.040 + 0.026\, I_s - 0.007\, I_{h,f} \qquad \textbf{(3-18)}$$

where:

b_b = base coefficient for a four-lane, depressed median highway;
I_s = indicator variable for shoulder width (1 for shoulder width, 0 for lane width); and
$I_{h,f}$ = indicator variable for road type (1 for highway or freeway, 0 for street).

The p-value for the I_s coefficient is 0.029 and that for the $I_{h,f}$ coefficient is 0.33. The weighted R^2 is 0.94. For rural highways, the value of b_b that is applicable to lane width is computed from Equation 3-18 as -0.047.

Based on the preceding analysis, the following equation represents the lane width AMF for the base combination of lanes and median type (i.e., four-lane, depressed median highway). The base condition lane width for this AMF is 12 ft.

$$AMF_{lw,b} = e^{-0.047\,(W_l\, -\, 12)} \qquad \textbf{(3-19)}$$

where:

$AMF_{lw,\,b}$ = lane width accident modification factor for the base combination of lanes and median type (i.e., four-lane, depressed median highway).

Equation 3-19 can be combined with Equation 3-16 to obtain the lane width AMF for other combinations of lanes and median type. The proportion of influential crashes for the base combination P_b is 0.36. The resulting AMF is:

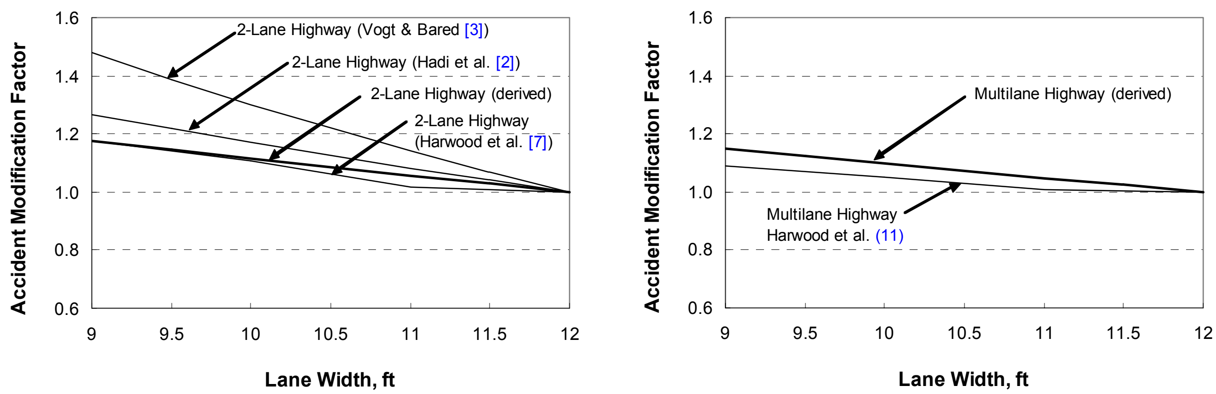
$$AMF_{lw,i} = (e^{-0.047\,(W_l\, -\, 12)} - 1.0)\,\frac{P_i}{0.36} + 1.0 \qquad \textbf{(3-20)}$$

It can be tailored to a desired number of lanes and median type by using the variable P_i. Values for this variable are listed in Table 3-8. The proportions in this table (and subsequent similar tables) were obtained from the crash database maintained by the State of Texas, Department of Public Safety.

Table 3-8. Crash Distribution for Rural Highway Lane Width AMF.

Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Depressed	Single-vehicle run-off-road, same direction sideswipe	4	0.36
		6	0.35
Undivided, TWLTL, or flush paved	Single-vehicle run-off-road, same direction sideswipe, multiple vehicle opposite direction	2	0.42
		4	0.37

The AMFs obtained from Equation 3-20 are illustrated in Figure 3-5. They are labeled "derived" and have a bold line weight. They are compared to the AMFs developed by other researchers. The trend is one of increasing AMF value with a reduction in lane width. For a given lane width, the derived AMF for two-lane highways is larger than that for multilane highways. There is general agreement between the derived AMFs and those reported in the literature. Significant differences between AMFs are likely a reflection of bias (due to colinearity) in the original models.



a. Two-Lane Highways. *b. Multilane Highways.*

Figure 3-5. Lane Width AMF for Rural Highways.

An expert panel convened by Harwood et al. *(7)* rationalized that lane width would not have as significant an effect on crash frequency at low volume as at higher volume. The general trend between lane width AMF and ADT suggested by this panel is shown in Figure 3-6. It has been modified to converge to the AMF obtained from Equation 3-20 for ADTs of 2000 veh/d or more.

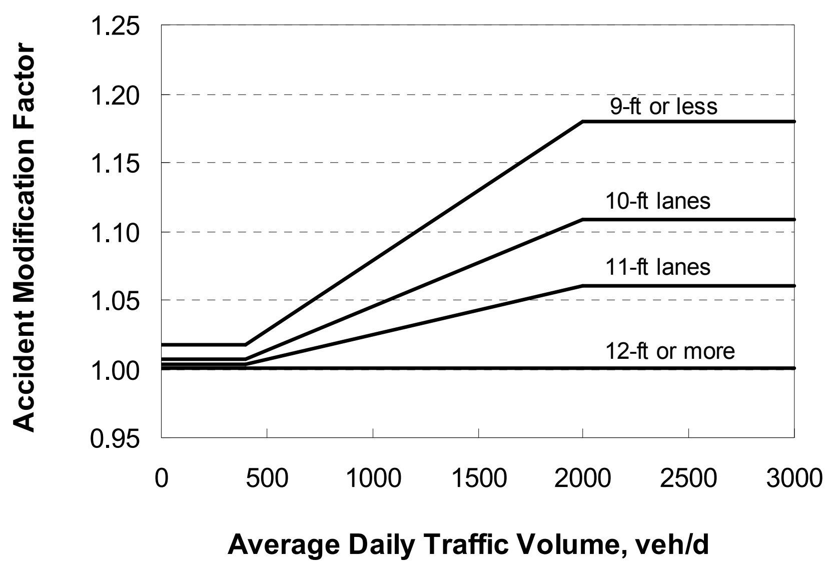


Figure 3-6. Lane Width AMF for Low Volume Rural Highways.

Outside Shoulder Width. The following equation represents the shoulder width AMF for the base combination of lanes and median type (i.e., four-lane, depressed median highway). It was derived in a similar manner as the lane width AMF. The base condition shoulder width is 8 ft.

$$AMF_{osw,b} = e^{-0.021\,(W_s - 8)} \qquad \textbf{(3-21)}$$

where:

$AMF_{osw,b}$ = outside shoulder width accident modification factor for the base combination of lanes and median type (i.e., four-lane, depressed median highway); and

W_s = outside shoulder width, ft.

Equation 3-21 can be combined with Equation 3-16 to obtain the outside shoulder width AMF for other combinations of lanes and median type. The proportion of influential crashes for the base combination P_b is 0.16. The resulting AMF is:

$$AMF_{osw,i} = \left(e^{-0.021\,(W_s - 8)} - 1.0 \right) \frac{P_i}{0.16} + 1.0 \qquad \textbf{(3-22)}$$

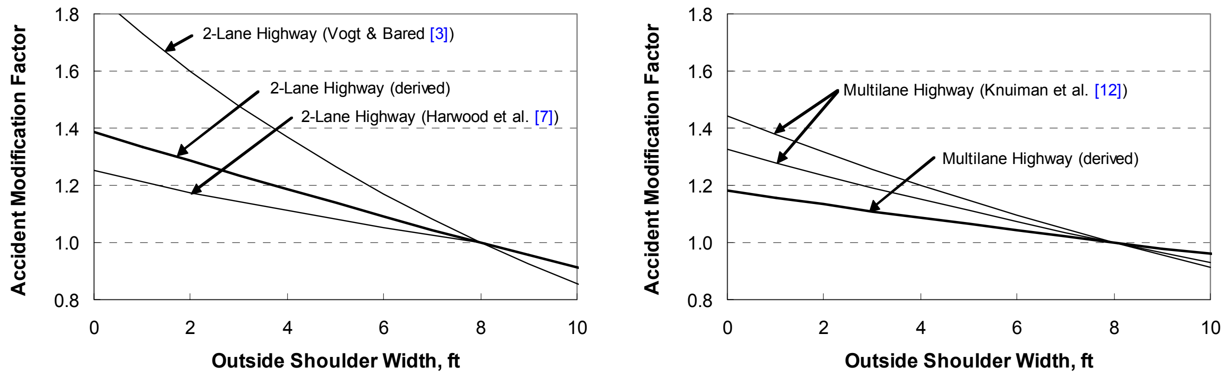
It can be tailored to the desired number of lanes and median type by using the variable P_i. The value of this variable is selected from Table 3-9.

Table 3-9. Crash Distribution for Rural Highway Outside Shoulder Width AMF.

Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Depressed	Single-vehicle run-off-road, right side	4	0.16
		6	0.15
Undivided, TWLTL, or flush paved	Single-vehicle run-off-road, either side	2	0.34
		4	0.27

The AMFs obtained from Equation 3-22 are illustrated in Figure 3-7. They are labeled "derived" and have a bold line weight. They are compared to the AMFs developed by other researchers. The trend is one of increasing AMF value with a reduction in shoulder width. For a given shoulder width, the AMF for two-lane highways is larger than that for multilane highways. There is general agreement between the derived AMFs and those reported in the literature. Significant differences between AMFs are likely a reflection of bias (due to colinearity) in the original models.

A shoulder width AMF was derived from the model developed by Wang et al. (*4*). However, it was very sensitive to shoulder width. In fact, it has a value of 2.1 when the shoulder width is 0.0 ft, which is much higher than the values obtained from other AMFs. An examination of the data evaluated by Wang et al. indicated that only 4 percent of the segments had shoulder widths less than 4 ft, yet these same segments account for 20 percent of the crashes. Hence, it is likely that there are unspecified factors that underlie this over-representation of crashes when categorized by shoulder width. For these reasons, the AMF derived from the Wang model was not considered further.



a. Two-Lane Highways. *b. Multilane Highways.*

Figure 3-7. Outside Shoulder Width AMF for Rural Highways.

An expert panel convened by Harwood et al. (*7*) used engineering judgment to limit the AMF they developed based on roadway volume. They rationalized that for very low traffic volumes, shoulder width would not have as significant an effect on crash frequency as at higher volumes. The relationship between shoulder width AMF and ADT suggested by this panel is based on a base shoulder width of 6 ft. It was converted to a base shoulder width of 8 ft for this document and is shown in Figure 3-8. This figure would be used instead of Equation 3-22 for ADTs of 2000 veh/d or less.

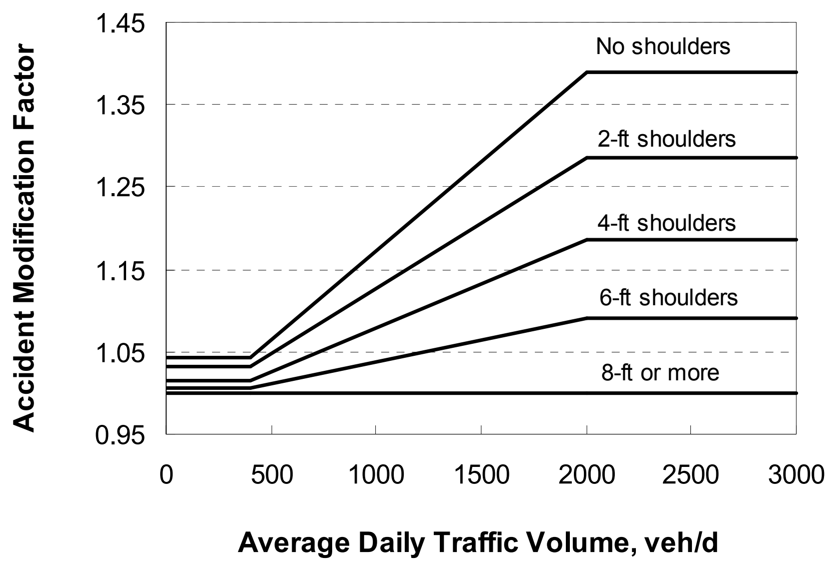


Figure 3-8. Outside Shoulder Width AMF for Low Volume Rural Highways.

Inside Shoulder Width. A review of the literature did not reveal any AMFs or safety prediction models that address the inside shoulder width of a rural highway. A model based on rural freeways was developed by Hadi et al. (*2*). It included a variable for inside shoulder width. However, when the regression coefficient associated with this variable was compared with coefficients for outside shoulder width (see Table 3-7), no significant difference was found between coefficients. Hence, Equation 3-21 is rationalized to be equally applicable to inside shoulder width, after adjustment for a base width of 4 ft. Equation 3-21 was combined with Equation 3-16 to obtain the following inside shoulder width AMF:

$$AMF_{isw,i} = \left(e^{-0.021\,(W_{is} - 4)} - 1.0 \right) \frac{P_i}{0.16} + 1.0 \qquad \textbf{(3-23)}$$

where:
AMF_{isw} = inside shoulder width accident modification factor; and
W_{is} = inside shoulder width, ft.

The proportion of influential crashes for the base combination P_b is 0.16. Equation 3-23 can be tailored to the desired number of lanes and median type by using the variable P_i. The value of this variable is selected from Table 3-10.

Table 3-10. Crash Distribution for Rural Highway Inside Shoulder Width AMF.

Facility Type	Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Rural highway	Depressed	Single-vehicle run-off-road, left side	4	0.16
			6	0.15

The AMF obtained from Equation 3-23 is illustrated in Figure 3-9. It is labeled "derived" and has a bold line weight. It is compared to the AMFs developed by Hadi et al. (*2*). The trend is one of increasing AMF value with a reduction in shoulder width. There is general agreement between the derived AMF and that derived from the Hadi model.

Median Width. AMFs for median width were developed from safety prediction models developed by Hadi et al. (*2*) and Wang et al. (*4*), and from data reported by Knuiman et al. (*12*). The Hadi and Wang AMFs are based on models developed for rural divided highways. Knuiman et al. examined crash data for a mix of rural freeways, urban freeways, and major highways. The crash rate adjustment factors (analogous to AMFs) they computed are shown in Figure 3-10, as are the "best-fit" trend lines. It should be noted that none of the researchers examined the relationship between median type (i.e., surfaced, depressed) and crash frequency.

Equation 3-24 was developed to represent the AMFs derived from the literature. The base condition for this AMF is a median width of 76 ft for depressed medians and 16 ft for surfaced medians (i.e., TWLTL or flush paved). The coefficients used in this model are listed in columns 4 through 6 of Table 3-11.

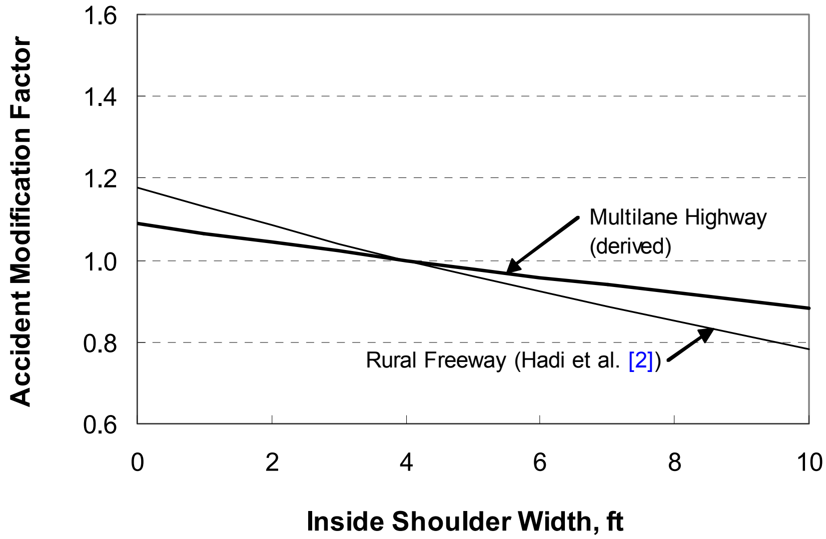


Figure 3-9. Inside Shoulder Width AMF for Rural Highways.

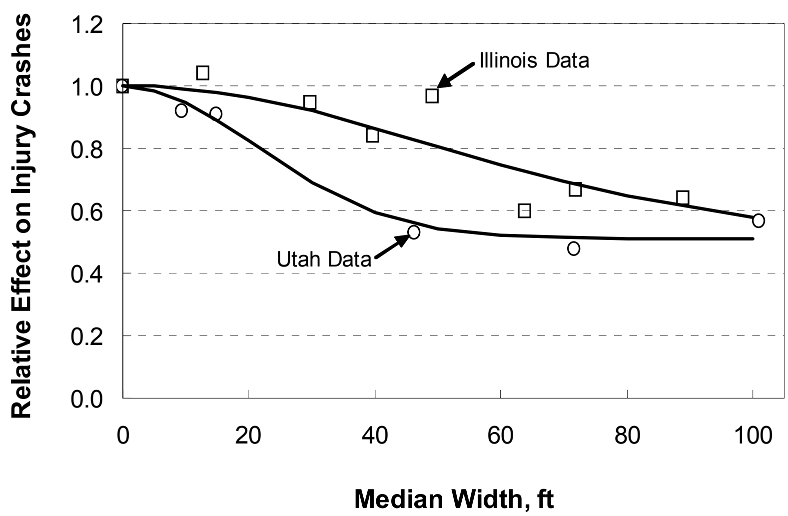


Figure 3-10. Relationship between Median Width and Severe Crash Frequency.

$$AMF_{mw} = \frac{b_0\,(e^{b_1 W_m^{b_2}} - 1.0) + 1.0}{b_0\,(e^{b_1 W_{mb}^{b_2}} - 1.0) + 1.0} \qquad \textbf{(3-24)}$$

where:

AMF_{mw} = median width accident modification factor;
W_{mb} = base median width (16 ft for surfaced median; 76 ft for depressed median), ft; and
W_m = median width, ft.

Table 3-11. Coefficient Values for Median Width on Rural Highways.

Model Source	Roadway Type	Crash Severity	Base Coefficients			Equivalent b_1 Coeff.	
			b_0	b_1	b_2	$W_m < 40$ft	$W_m > 40$ft
Hadi et al. (*2*)	Rural, 4-lane, highway	All	1.0	-0.0458	0.5	-0.046	-0.046
Knuiman et al. (*12*)	Urban & rural, 4-lane, highway (Utah)	Severe	0.488	-0.00111	2.0	-0.153	-0.055
	Urban & rural, 4-lane, highway (Illinois)	Severe	0.483	-0.00020	2.0	-0.042	-0.111
Wang et al. (*4*)	Rural, 4-lane, highway	All	1.0	-0.003	1.0	-0.029	-0.046
					Weighted Average:	**-0.038**	**-0.052**

Figure 3-11 illustrates the AMFs listed in Table 3-11. They have a base condition median width of 16 ft. Similar trends are obtained for a base width of 76 ft. The AMFs are in general agreement. The AMF from Knuiman et al. using Utah data is less in agreement and indicates a greater sensitivity to median width for widths of 40 ft or less.

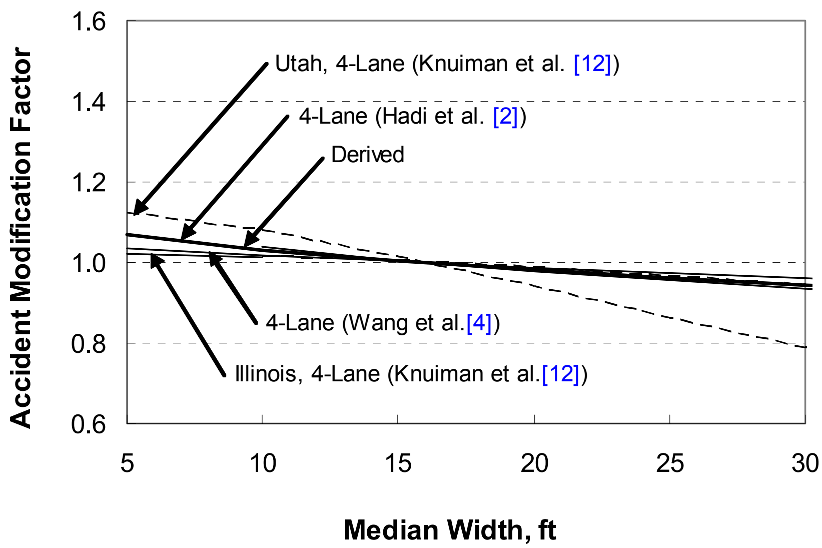


Figure 3-11. Median Width AMF for Rural Highways.

The derived AMF relationship is shown in Figure 3-11. It is based on Equation 3-24 with $b_0 = 1.0$, $b_2 = 0.5$, and the weighted average of the equivalent b_1 coefficients listed in columns 7 and 8 of Table 3-11 (where the weight w for each coefficient is equal to its reciprocal squared [i.e., $w = 1/b^2$]). The equivalent b_1 coefficient for each model source was derived from a regression analysis using Equation 3-24 with $b_0 = 1.0$ and $b_2 = 0.5$ for median widths ranging from 10 to 40 ft. The analysis was repeated for median widths ranging from 40 to 80 ft. The AMFs obtained from Equation 3-24 (using the base coefficients) served as the dependent variable.

Shoulder Rumble Strips. Griffith (*14*) investigated the correlation between the presence of continuous rolled-in rumble strips and crash frequency on rural freeways in Illinois. The focus of his investigation was on single-vehicle run-off-road crashes. The reported AMFs are shown in column 5 of Table 3-12. It is rationalized that these factors are also applicable to rural highways. The standard deviation for the AMF value of 0.93 in Table 3-12 indicates the AMF is not significantly different from 1.0. However, the AMF value is only slightly larger than that used for freeways in Chapter 2 (and which is more confidently known) and is reasoned to be a conservative estimate of the relationship between rumble strips and severe crashes on rural highways.

Table 3-12. AMFs for Shoulder Rumble Strips on Rural Highways.

Data Source	State	Crash Severity	Crash Type Subset	Base AMF (f_{rs})	Standard Deviation
Griffith (*14*)	Illinois	All	Single-vehicle run-off-road	0.79	0.10
		Injury	Single-vehicle run-off-road	0.93	0.15

The shoulder rumble strip AMF can be computed using Equation 3-25. It is based on the overall base AMF for severe crashes in Table 3-12. For Texas applications, it requires the appropriate crash distribution proportion for the roadway type of interest. These proportions are listed in Table 3-13.

$$AMF_{srs} = (0.93 - 1.0)\,P_i + 1.0 \qquad \textbf{(3-25)}$$

where:

AMF_{srs} = shoulder rumble strip accident modification factor; and

P_i = proportion of influential crashes that occur on roadway type *i* (from Table 3-13).

Table 3-13. Crash Distribution for Rural Highway Shoulder Rumble Strip AMF.

Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Depressed	Single-vehicle run-off-road, either side	4	0.32
		6	0.31
Undivided, TWLTL, or flush paved	Single-vehicle run-off-road, right side	2	0.17
		4	0.13

Centerline Rumble Strip. Persaud et al. (*15*) investigated the effect of continuous centerline rumble strips on crash frequency for rural two-lane highways in seven states. The reported crash reduction factors were converted to AMFs for this chapter; these values are shown in column 4 of Table 3-14. It is rationalized that these factors are also applicable to multilane, undivided highways.

Table 3-14. AMFs for Centerline Rumble Strip on Rural Two-Lane Highways.

Data Source	Crash Severity	Crash Type Subset	Base AMF	Standard Deviation
Persaud (*15*)	Injury	All	0.86	0.07
	All	All	0.88	0.06

Given the preference for AMFs based on severe crash data, the two-lane highway centerline rumble strip AMF is:

$$AMF_{crs} = 0.86 \qquad \textbf{(3-26)}$$

where:
AMF_{crs} = centerline rumble strip accident modification factor.

TWLTL Median Type. Harwood et al. (*7*) developed an AMF to capture the effect of driveway activity on crash frequency, as it relates to the addition of a TWLTL to an undivided two-lane highway. This AMF is shown as Equation 3-27.

$$AMF_T = 1.0 - 0.7\, P_D\, P_{LT/D} \qquad \textbf{(3-27)}$$

with,

$$P_D = \frac{0.0047\, D_d + 0.0024\, D_d^2}{1.199 + 0.0047\, D_d + 0.0024\, D_d^2} \qquad \textbf{(3-28)}$$

where:
AMF_T = TWLTL accident modification factor;
P_D = driveway-related crashes as a proportion of total crashes; and
$P_{LT/D}$ = left-turn accidents susceptible to correction by a TWLTL as a proportion of driveway-related crashes (estimated as 0.50).

Equation 3-28 represents the ratio of driveway-related crash rate to total crash rate.

The AMF for adding a TWLTL to a rural highway is shown in Figure 3-12. The trend line attributed to Harwood et al. (*7*) is based on Equations 3-27 and 3-28. It indicates that the TWLTL is more effective at reducing crashes as driveway density increases.

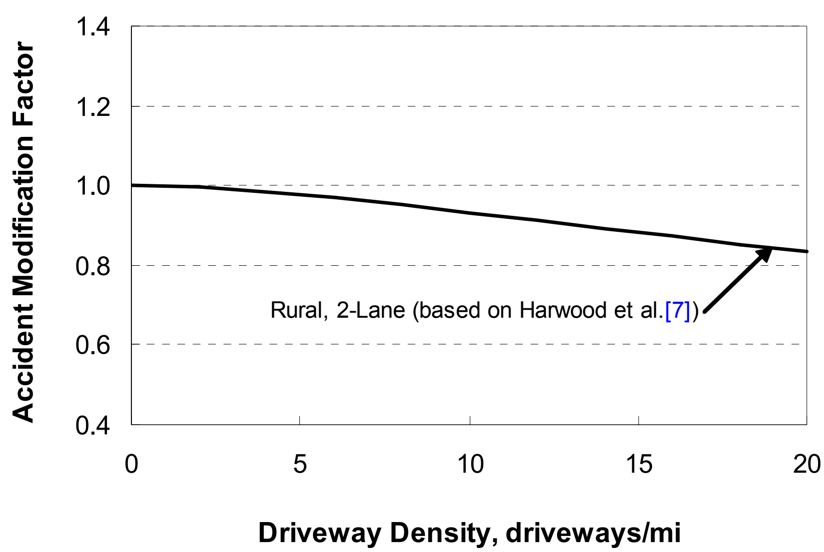


Figure 3-12. TWLTL Median Type AMF for Rural Highways.

Superelevation. Harwood et al. (*7*) developed an AMF to describe the relationship between superelevation rate and crash frequency. Their analysis indicated that crash frequency was higher on highways that were deficient in the amount of superelevation provided. Superelevation deficiency was computed as the difference between the superelevation provided on the curve and that specified in the AASHTO document *A Policy on Geometric Design of Highways and Streets* (*16*) (*Green Book*). Crash frequency was found to be higher with increasing superelevation deficiency.

Harwood et al. (*7*) convened an expert panel to review the relationship between superelevation deficiency and crash frequency. They determined that curves with a superelevation deficiency of 1.0 percent or less would not likely experience an increase in crashes, as suggested by the crash data analysis. The AMF values for superelevation deficiency that are recommended by the expert panel are listed in Table 3-15. If the existing (or proposed) superelevation rate equals or exceeds that recommended in the AASHTO *Green Book*, then the AMF is 1.0.

Table 3-15. AMFs for Superelevation on Rural Highways.

	Superelevation Deficiency,[1] %					
	0.0 or less	1.0	2.0	3.0	4.0	5.0
AMF_e:	1.00	1.00	1.06	1.09	1.12	1.15

Note:
1 - Superelevation deficiency = AASHTO Policy superelevation rate − existing (or proposed) superelevation rate.

Passing Lanes. Harwood et al. (*7*) developed an AMF to describe the relationship between passing lane presence and crash frequency. Their analysis indicated that crash frequency was lower on two-lane highways to which a passing lane was added. They focused their evaluation on two types of passing lanes: (1) conventional passing lane or climbing lane added in one direction of a two-lane highway and (2) short four-lane section specifically built to provide passing opportunities in both directions. The climbing lane or passing lane is assumed to satisfy warrants for such lanes and has a length that is sufficient to provide safe and efficient passing opportunities. Guidelines describing justification criteria and length considerations for climbing lanes and passing lanes are provided in the *Green Book* (*16*).

The AMFs for passing lanes AMF_{pass} are listed in Table 3-16. They are not applicable to three- or four-lane cross sections having a length that extends beyond that needed to provide safe and efficient passing operation.

Table 3-16. AMFs for Passing Lanes on Rural Two-Lane Highways.

	Climbing Lane or Passing Lane Type[1]	
	One Direction (three-lane cross section)	Two Directions (four-lane cross section)
AMF_{pass}:	0.75	0.65

Note:
1 - Only applicable to highway segments where the passing lane is warranted and has a length that is sufficient to provide safe and efficient passing opportunities (no more and no less).

Roadside Design

This section describes AMFs related to the roadside design of a rural highway segment. Topics specifically addressed are listed in Table 3-17. Many roadside design components or elements that are not listed in this table (e.g., ditch shape) are also likely to have some correlation with severe crash frequency on the highway segment. However, a review of the literature did not reveal that their effect has been quantified by previous research. The list of available AMFs for highway roadside design is likely to increase as new research in this area is undertaken.

Table 3-17. AMFs Related to Roadside Design of Rural Highways.

Section	Accident Modification Factor		
Cross section	Horizontal clearance Bridge width	Side slope	Utility pole density
Appurtenances	see text		

AMFs for roadside safety appurtenances are not described in this chapter because they generally do not exist. The safety literature related to roadside appurtenances has focused on the information needed to evaluate the cost-effectiveness of installing individual appurtenances at specific locations. The information is very detailed due to the design and operational complexity of various appurtenances and the influence of site-specific conditions on their performance. Moreover, safety appurtenances may sometimes increase crash frequency while, more importantly, reducing the severity of the crash. For these reasons, the safety benefits derived from an appurtenance are typically estimated on an individual, case-by-case basis using the techniques described in the *Roadside Design Guide* (*17*).

Cross Section

Horizontal Clearance. The relationship between horizontal clearance distance and single-vehicle run-off-road crashes was evaluated by Miaou (*18*). He developed a safety prediction model for rural highways that included this distance as a variable. The AMF derived from this model is described in Equation 3-29. The base condition for this AMF is a horizontal clearance of 30 ft.

$$AMF_{hc} = (e^{b(W_{hc} - 30)} - 1.0)\, P_s + 1.0 \qquad \textbf{(3-29)}$$

where:
AMF_{hc} = horizontal clearance accident modification factor; and
W_{hc} = horizontal clearance (average for length of segment), ft.

The value of variable *b* is provided in Table 3-18. The subset proportions appropriate for Texas rural highway applications are provided in Table 3-19. For four-lane divided highways, Equation 3-29 yields an AMF that ranges from 1.05 at 10 ft clearance to 0.95 at 60 ft clearance.

Table 3-18. Coefficient Values for Horizontal Clearance on Rural Highways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s	Coefficient b
Miaou (*18*)	Rural, 2-lane, undivided	All	Single-vehicle run-off-road	unreported	-0.0137

Table 3-19. Crash Distribution for Rural Highway Horizontal Clearance Width AMF.

Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Depressed	Single-vehicle run-off-road, right side	4	0.16
		6	0.15
Undivided, TWLTL, or flush paved	Single-vehicle run-off-road, either side	2	0.34
		4	0.27

Side Slope. The relationship between side slope and crash frequency was evaluated by Miaou (*18*). He developed a safety prediction model for rural highways that included side slope as a variable. The AMF derived from this model is described in Equation 3-30. The base condition for this AMF is a side slope of 1V:4H (i.e., S_s = 4.0).

$$AMF_{ss} = (e^{b(1/S_s - 1/4)} - 1.0)\,P_s + 1.0 \tag{3-30}$$

where:

AMF_{ss} = side slope accident modification factor; and

S_s = horizontal run for a 1 ft change in elevation (average for length of segment), ft.

The value of variable b is provided in Table 3-20. The subset proportions appropriate for Texas rural highway applications are provided in Table 3-19. For four-lane divided highways, Equation 3-30 yields an AMF that ranges from 1.00 at 1V:4H to 1.01 at 1V:3H.

Table 3-20. Coefficient Values for Side Slope on Rural Highways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s	Coefficient b
Miaou (*18*)	Rural, 2-lane, undivided	All	Single-vehicle run-off-road	unreported	0.692

Utility Pole Density. The relationship between utility pole density and crash frequency was evaluated by Zegeer and Parker (*19*). They developed a safety prediction model for rural and urban roads in four states. The model included average pole offset, traffic volume, and pole density as independent variables. The AMF derived from this model is described in Equation 3-31. Details of the model are described in Table 3-21. The base condition is 25 poles/mi and a pole offset of 30 ft.

$$AMF_{pd} = (f_p - 1.0)\,P_s + 1.0 \qquad \textbf{(3-31)}$$

with,

$$f_p = \frac{(0.0000984\,ADT + 0.0354\,D_p)\,W_o^{-0.6} - 0.04}{0.0000128\,ADT + 0.075} \qquad \textbf{(3-32)}$$

where:
- AMF_{pd} = utility pole accident modification factor;
- D_p = utility pole density (two-way total), poles/mi; and
- W_o = average pole offset from nearest edge of traveled way, ft.

Table 3-21. Coefficient Values for Utility Pole Density on Rural Highways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s
Zegeer & Parker *(19)*	Urban and rural roads	All	Single-vehicle collision with pole	0.022

The subset proportions appropriate for Texas rural highway applications are provided in Table 3-22. Equation 3-31 yields an AMF that ranges from 1.08 at a pole density of 50 poles/mi and offset of 10 ft to 0.99 at a pole density of 10 poles/mi and offset of 30 ft. The AMF is more sensitive to pole offset than it is to density or average daily traffic volume.

Table 3-22. Crash Distribution for Rural Highway Utility Pole Density AMF.

Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Depressed	Single-vehicle collision with pole	4	0.054
		6	0.046
Undivided, TWLTL, or flush paved	Single-vehicle collision with pole	2	0.038
		4	0.048

Bridge Width. Turner *(20)* evaluated the relationship between bridge width and bridge crash rate for rural two-lane highways in Texas. "Relative bridge width" was defined as the difference between the bridge width and the approach traveled-way width. Bridge width was measured between the face of the bridge rails. Traveled-way width is the pavement width available for vehicle movement, exclusive of paved shoulders and auxiliary lanes. *Approach* traveled-way width is the traveled-way width of the normal highway cross section, prior to any change in cross section associated with the bridge approach.

The relationship between relative bridge width and crash rate, as reported by Turner *(20)*, is shown in Figure 3-13 using circular data points. Crash rate is defined as bridge-related crashes per million vehicles (mv). A best-fit trend line and corresponding equation are also shown. The AMF derived from this model is described in Equation 3-33. The base condition is a relative bridge width of 12 ft, which is consistent with an 8 ft outside shoulder, a 4 ft inside shoulder, and no reduction in traveled-way width across the bridge.

$$AMF_{bw} = (e^{b\, I_{br}\,(W_b\, -\, 12)} - 1.0)\, P_s + 1.0 \qquad \textbf{(3-33)}$$

where:

AMF_{bw} = bridge width accident modification factor;
I_{br} = presence of bridges (1 if one or more bridges present, 0 if not); and
W_b = relative bridge width (= bridge width − approach traveled-way width), ft.

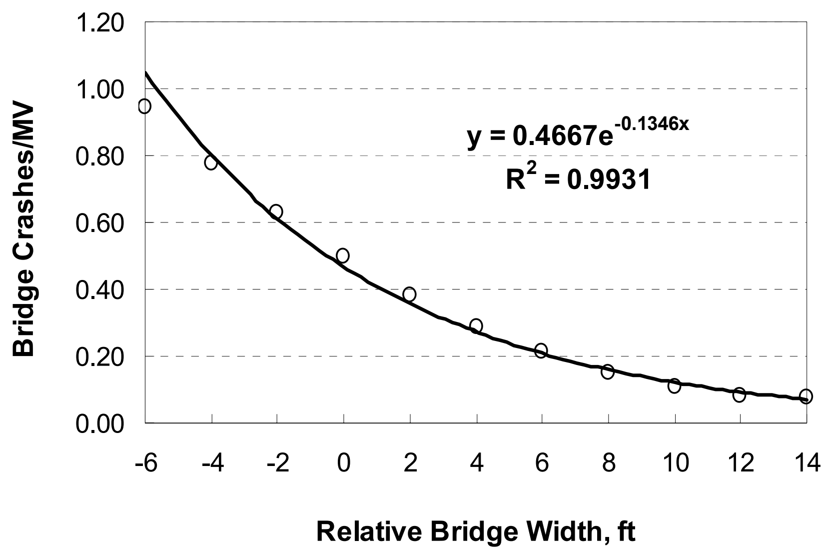


Figure 3-13. Correlation between Relative Bridge Width and Bridge Crash Rate.

The value of variable *b* is provided in Table 3-23. The subset proportions appropriate for two-lane highway applications are provided in Table 3-24.

Table 3-23. Coefficient Values for Bridge Width on Rural Highways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s	Coef-ficient *b*
Turner (20)	Rural, 2-lane, undivided	All	Single-vehicle collision with bridge	unreported	-0.135

Table 3-24. Crash Distribution for Rural Highway Bridge Width AMF.

Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Undivided, TWLTL, or flush paved	Single-vehicle collision with bridge	2	0.017
		4	0.011

Appurtenances

As discussed previously, AMFs for roadside safety appurtenances are not described in this chapter. The safety literature related to roadside appurtenances has focused on the information needed to evaluate the cost-effectiveness of installing individual appurtenances at specific locations. Most of the research conducted in this area has been incorporated into a complex and comprehensive procedure for evaluating appurtenances on a case-by-case basis. This procedure is outlined in a report by Mak and Sicking *(21)* and automated in the *Roadside Safety Analysis Program* (RSAP) *(22)*.

RSAP can be used to evaluate alternative roadside safety appurtenances on individual road segments. The program accepts as input information about the road segment geometry and traffic characteristics. It also allows the analyst to describe the roadside cross section, location of fixed objects, and safety appurtenance design. Table 3-25 summarizes the various RSAP inputs. The output from RSAP includes an estimate of annual crash frequency as well as the road-user costs associated with these crashes. The crash reduction potential realized by adding a roadside safety appurtenance (or changing the roadside cross section) can be estimated by specifying the changed condition as an "alternative."

Table 3-25. RSAP Input Data Requirements.

Design Category	Design Component	Design Element	
General	--	Area type (urban/rural) One-way/two-way Segment length	Functional class Speed limit
Traffic characteristics	--	Traffic volume (ADT) Traffic growth factor	Truck percentage
Geometric design	Horizontal alignment	Direction of curve	Radius
	Vertical alignment	Grade	
	Cross section	Divided/undivided Lane width Median type	Number of lanes Shoulder width Median width
Roadside design	Cross section	Foreslope Parallel ditches	Backslope Intersecting slopes
	Fixed object	Offset Type (wood pole, headwall, etc.) Width	Side of roadway Spacing
	Safety appurtenances	Offset Type (barrier, cushion, etc.) Length	Side of roadway Flare rate Width

Access Control

This section is devoted to AMFs related to the access control elements of a rural highway. A review of the literature indicates that median type and driveway density have an influence on crash frequency. The correlation between median type and crashes is addressed previously in the section titled TWLTL Median Type. This section on access control describes the correlation between driveway density and crash frequency.

Four accident modification factors for driveway density were identified during the literature review. The first is offered by Harwood et al. (*7*). It was developed for rural two-lane highways and is based on an analysis by Hauer (*23*) of crash rates published by others. The equation described by Harwood et al. (*7*) is listed herein as Equation 3-34. The constant "0.622" was added to reflect the fact that the original form of the model used a driveway density variable that was based on units of "driveways/km."

$$AMF_{dd} = \frac{0.20 + [0.05 - 0.005\ln(ADT)]\, D_d\, 0.622}{0.20 + [0.05 - 0.005\ln(ADT)]\, D_{base}\, 0.622} \qquad \textbf{(3-34)}$$

where:
AMF_{dd} = driveway density accident modification factor; and
D_{base} = base driveway density, driveways/mi.

The remaining three AMFs for driveway density were derived from safety prediction models developed by Vogt and Bared (*3*), Harwood et al. (*7*), and Wang et al. (*4*). The models by Vogt and Bared and by Harwood et al. are based on two-lane, undivided highway segments. The model by Wang et al. is based on four-lane, divided highways. The generalized AMF is shown below. Values of the variables b and P_s are provided in Table 3-26.

$$AMF_{dd} = (e^{b\,(D_d - D_{base})} - 1.0)\, P_s + 1.0 \qquad \textbf{(3-35)}$$

Table 3-26. Coefficient Values for Driveway Density for Rural Highways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s	Coefficient b
Vogt & Bared (*3*)	Rural, 2-lane, undivided	Severe	All	1.0	0.0062
Harwood et al. (*7*)	Rural, 2-lane, undivided	All	All	1.0	0.0084
				Weighted Average:	**0.0070**
Wang et al. (*4*)	Rural, 4-lane, divided	All	All	1.0	0.034

All four AMFs are compared in Figure 3-14 for a base driveway density of five driveways/mi. The trends in the data indicate that the relationship between driveway density and crashes varies widely. This variation is likely due to the fact that driveway activity level (i.e., driveway traffic demand) is not used in the models. Equation 3-34 exhibits an illogical trend such that, for a given driveway density, the AMF value decreases with increasing daily traffic demand. Logically, the driveway volume should increase with increasing daily traffic demand, which should *increase* the risk of a collision.

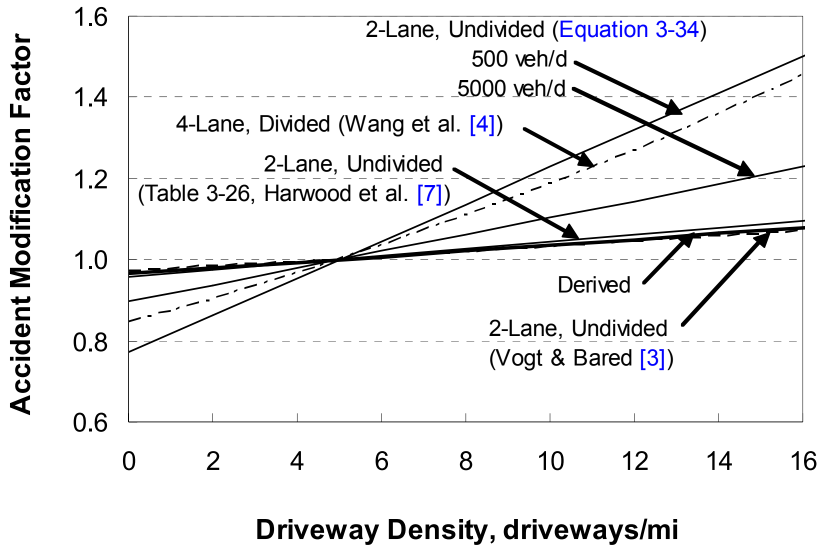


Figure 3-14. Driveway Density AMF for Rural Highways.

The two, two-lane highway AMFs in Table 3-26, show less sensitivity to driveway density than the four-lane highway AMF attributed to Wang et al. It is possible that crash frequency on four-lane highways is more sensitive to driveway density. However, it is also possible that the data from which these AMFs were derived inadvertently introduced some bias due to unspecified variables that are correlated with driveway density.

A weighted average of the coefficients for the two AMFs based on two-lane, undivided highways yields a value of 0.007. The weight w used for this average equals the reciprocal of the coefficient squared (i.e., $w = 1/b^2$). It is unclear whether the correlation between driveway density and AMF value varies with highway cross section. Based on this analysis, the best-fit AMF for driveway density on rural two-lane highways is:

$$AMF_{dd} = e^{0.007\,(D_d - 5)} \tag{3-36}$$

At this time, it rationalized that this AMF represents the best estimate of the relationship between driveway density and crash frequency for multilane rural highways.

Other Adjustment Factors

This section describes AMFs related to features of the highway that are not categorized as related to geometric design or roadside design. At this time, the only AMF addressed in this section is speed limit.

Two safety prediction models developed for rural highways were found to have a variable for speed limit. One of these models was developed by Hadi et al. (*2*). Another was that developed by Milton and Mannering (*10*). A generalized AMF form is shown in Equation 3-37 for a base speed limit of 55 mph. It can be used to represent the AMF obtained from both of the two prediction models by proper substitution of the coefficient *b* from Table 3-27.

$$AMF_{sl} = e^{b(V - 55)} \qquad \textbf{(3-37)}$$

where:

AMF_{sl} = speed limit accident modification factor; and
V = speed limit, mph.

The two AMFs are compared in Figure 3-15. The two coefficients for all model sources are so similar that they plot as one trend line. The trends in the figure indicate that higher speed limits are associated with fewer severe crashes. Although not shown in the figure, similar trends have been found in safety prediction models developed for urban streets. It is likely that an increase in speed limit does not make the highway safer; rather, it probably indicates that highways with a higher speed limit tend to have a more generous design.

Table 3-27. Coefficient Values for Speed Limit for Rural Highways.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Coefficient *b*
Hadi et al. (*2*)	Rural, 2-lane, highway	Injury	All	-0.0108
Milton & Mannering (*10*)	Urban & rural, principal arterials	All	All	-0.0111 [1] -0.0103
			Weighted Average:	**-0.011**

Note:
1 - Separate safety prediction models were developed for data from principal arterials in the eastern and western half of Washington State.

The derived AMF relationship is shown in Figure 3-15. It uses the weighted average of the *b* coefficients listed in Table 3-27 (where the weight *w* for each coefficient is equal to its reciprocal squared [i.e., $w = 1/b^2$]). There is insufficient data to determine if this coefficient varies by number-of-lanes or median type. Hence, the derived AMF is offered as approximate for all lanes and median types until new information becomes available.

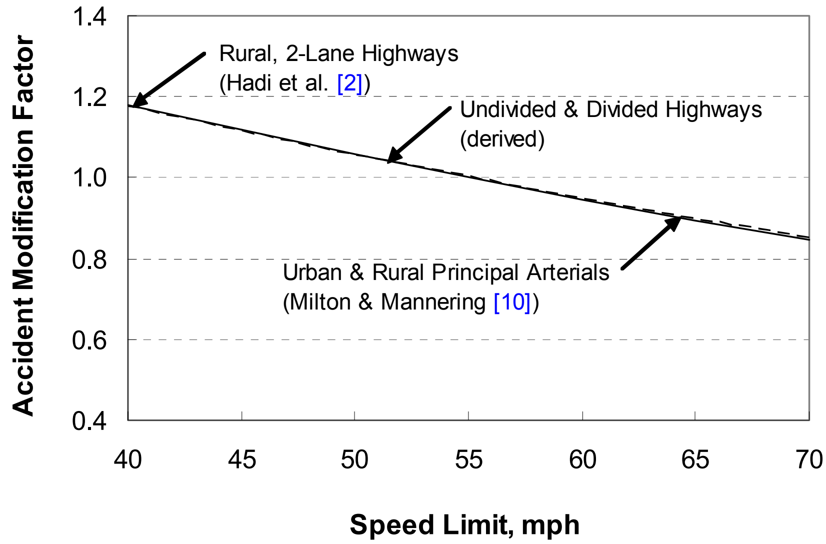


Figure 3-15. Speed Limit AMF for Rural Highways.

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Chapter 4
Urban Streets

Page

INTRODUCTION

The urban street is intended to provide both mobility to through travelers and access to adjacent properties. It is also intended to serve motorists, transit riders, pedestrians, and bicyclists within the right-of-way. As consequence of this mixture of function and travel mode, the urban street has a complex design that is challenged to serve all modes in an equally safe and efficient manner. The high cost of right-of-way presents additional challenges to the development of a design that is not only safe and efficient but also cost-effective.

The development of a safe, efficient, and economical urban street design typically reflects the consideration of a wide variety of design alternatives. A variety of techniques exist for estimating the operational benefits of alternatives; many are automated through software tools. Techniques for estimating construction and right-of-way costs are also available to the designer. Unfortunately, techniques for estimating the safety benefits of alternative designs are not as readily available. This chapter summarizes information in the literature that can be used to estimate the crash frequency associated with various urban street design alternatives.

Objective

The objective of this chapter is to synthesize information in the literature that quantitatively describes the relationship between various urban street design components and safety. This information is intended to provide a basis for the development of a procedure for estimating the safety benefit of alternative designs. This procedure is documented in Chapter 4 of the *Roadway Safety Design Workbook* (*1*).

The presentation consists of an examination of safety prediction models and accident modification factors (AMFs). Safety prediction models provide an estimate of the expected crash frequency for a typical urban street segment. They include variables for traffic volume and segment length. They also include variables for other factors considered to be correlated with crash frequency (e.g., median type, land use, etc.). One or more AMFs can be multiplied by the expected crash frequency obtained from the prediction model to produce an estimate of the expected crash frequency for a specific street segment.

Scope

This chapter addresses the safety of urban street segments. It does not address the safety of intersections on these streets. For this reason, the crashes addressed herein are referred to as "mid-block" crashes. Crashes that occur at, or are related to, urban intersections are the subject of Chapter 7.

When available, safety relationships that estimate the frequency of *severe* (i.e., injury or fatal) crashes are given preference for inclusion in this document. This preference is due to a wide variation in reporting threshold among cities and states. This variation complicates the extrapolation of crash trends found in one location to another location. Moreover, it can confound the development of safety prediction models using data from multiple agencies. Reporting threshold is strongly correlated with the number of property-damage-only (PDO) crashes found in a crash database. Agencies with a high reporting threshold include relatively few PDO crashes in their database and vice versa. As a consequence, the total crash frequency for a given street will be low if it is located in an area with a high reporting threshold. In contrast, this same street will have a high crash frequency if it is located in an area with a low reporting threshold. This problem is minimized when crash data analyses, comparisons, and models are based on severe crash data.

The relationships described in this chapter address the occurrence of vehicle-related crashes on urban streets. They assume that the distribution of pedestrian and bicycle crashes remains unchanged, regardless of a change in street design or traffic volume. Relationships that specifically focus on vehicle-pedestrian and vehicle-bicycle crashes on streets are not addressed.

Overview

This chapter documents a review of the literature related to urban street safety. The focus is on quantitative information that relates severe crash frequency to various geometric design components of the urban street. The review is not intended to be comprehensive in the context of referencing all works that discuss urban street safety. Rather, the information presented herein is judged to be the most current information that is relevant to urban street design in Texas. It is also judged to be the most reliable based on a review of the statistical analysis techniques used and the explanation of trends.

Where appropriate, the safety relationships reported in the literature are compared herein, with some interpretation offered to explain any differences noted. The relationships are typically presented as reported in the literature; however, the names or the units of some variables have been changed to facilitate their uniform presentation in this chapter.

This chapter is envisioned to be useful to design engineers who desire a more complete understanding of the relationship between various urban street design components and severe crash frequency. As noted previously, it is also intended to serve as the basis for the development of the safety evaluation procedure described in Chapter 4 of the *Roadway Safety Design Workbook* (*1*).

This chapter consists of two main parts. In the first part to follow, several safety prediction models reported in the literature are described. In the second part, accident modification factors are described. Within this part, the various factors examined are organized into the following categories: roadway geometric design, roadside design, and access control.

SAFETY PREDICTION MODELS

Described in this part of the chapter are several models that were developed to estimate the expected frequency of crashes on urban street segments. The first three models were selected primarily because they explicitly exclude crashes associated with intersections (both signalized and unsignalized). A fourth model that excludes only signalized intersection crashes is also included in the summary. This model was added because it includes several factors included in the three other models and, thereby, provides some confirmation of the trends they exhibit.

Harwood Models

The models described in this section are derived from the crash rates reported by Harwood (*2*) for suburban highways. The criteria used to identify a "suburban highway" were sufficiently broad (e.g., speeds between 35 and 50 mph; signal spacing of 0.25 miles or more, etc.) that the resulting models are likely to reflect the safety experience of most urban streets with reasonable accuracy. The reported crash rates are categorized by median type, number of through lanes, and adjacent land use. They are listed in Table 4-1.

Table 4-1. Base Crash Rates and Adjustment Factors Reported by Harwood (*2*).

Base Crash Rate, crashes/million-veh-mi					
Adjacent Land Use	**Through Traffic Lanes and Median Type [1]**				
	2 Lanes		**4 Lanes**		
	Undivided	**TWLTL**	**Undivided**	**Raised Curb**	**TWLTL**
Commercial	2.39	1.56	2.85	2.90	2.69
Residential	1.88	1.64	0.97	1.39	1.39
Adjustment Factors					
Driveway density [2]	Driveways/mile crash rate adjustment factor		Under 30 -0.41	30 to 60 -0.03	Over 60 +0.35
Truck presence	Percent Trucks crash rate adjustment factor		Under 5 +0.18	5 to 10 -0.07	Over 10 -0.33
Property-Damage-Only Crash Distribution, %					
Adjacent Land Use	**Through Traffic Lanes and Median Type [1]**				
	2 Lanes		**4 Lanes**		
	Undivided	**TWLTL**	**Undivided**	**Raised Curb**	**TWLTL**
Commercial	60.9	70.1	62.3	66.0	66.7
Residential	57.8	56.3	62.0	55.2	60.4

Notes:
1 - TWLTL - two-way left-turn lane.
2 - Driveway density is based on a count of driveways on both sides of the street (i.e., a two-way total).

The rates in Table 4-1 can be used in the following equation to compute the expected frequency of severe crashes.

$$C_i = 0.000365\ ADT\ L\ (Base_i + Drive + Truck)\ AMF_{sw}\ (1.0\ -\ 0.01\ PDO_i) \qquad \textbf{(4-1)}$$

where:

C_i = frequency of severe crashes on urban streets with median type and land use combination i, crashes/yr;
ADT = average daily traffic, veh/d;
L = street segment length, mi;
$Base_i$ = base crash rate for urban street type i (see Table 4-1), crashes/million-veh-mi;
$Drive$ = adjustment for driveway density (see Table 4-1), crashes/million-veh-mi;

Truck = adjustment for truck presence (see Table 4-1), crashes/million-veh-mi;
AMF_{sw} = shoulder width accident modification factor (1.05 if curb-and-gutter is used instead of shoulder or if W_s is 4 ft or less; 0.95 if W_s more than 8 ft; 1.00 otherwise);
W_s = outside shoulder width (paved or unpaved), ft; and
PDO_i = percent property-damage-only crashes on urban streets of type *i*.

Equation 4-1 was developed using the guidance offered by Harwood (*2*). The adjustment factor for shoulder width in this equation was derived from the statement that "accident rates should be decreased by 5 percent for highway sections with full shoulders and increased by 5 percent for highway sections with no shoulders" (*2*, p. 9) where "full shoulders" is defined as shoulders of 8 ft or more and "no shoulders" is defined as shoulders of 4 ft or less. Given that the research included cross sections both with and without curbs, it is inferred that the "no shoulders" case includes cross sections with curb and gutter.

The PDO term in Equation 4-1 was added to convert the total crash estimate into a severe crash estimate. The use of this adjustment is based on the assumption that the relationship between the various model variables and total crash frequency is the same for severe crashes. The PDO percentages needed for this adjustment are provided in Table 4-1.

The base crash rates in Table 4-1 for two-lane streets are consistent with the commonly held belief that the TWLTL is safer than an undivided cross section. However, the base crash rates for four-lane streets do not maintain this consistency. They suggest that divided cross sections are less safe than undivided cross sections, a trend that is contrary to most research findings. Harwood (*2*) did not offer an explanation for these trends.

The characteristics of the data used by Harwood are summarized in Table 4-2. He used five years of crash data from two states to compute the crash rates for three median types. All total, his database reflected the crash history of 420 suburban highway segments representing a total length of 254.8 miles.

Table 4-2. Database Characteristics for Various Safety Prediction Models.

Model Developers	Database Characteristics				
	Median Type	**Years of Crash Data**	**Number of Street Segments**	**Total Section Length, mi**	**Number of Through Lanes**
Harwood (*2*)	Undivided	5	222	129.4	2, 4
	Raised curb	5	44	21.8	4
	TWLTL	5	154	103.6	2, 4
Hadi et al. (*3*)	Undivided	4	not available	not available	2, 6
Hauer et al. (*4*)	Undivided	4	not available	122.0	4
Bonneson & McCoy (*5*)	Undivided	3	62	26.0	2, 4
	Raised curb	3	59	25.1	4, 6
	TWLTL	3	68	27.5	4, 5

Hadi Models

The models described in this section are developed from the equations reported by Hadi et al. (*3*). They used negative binomial regression analysis to calibrate a set of safety prediction models. The models are categorized by crash severity, area type (i.e., urban, rural), number of through lanes, and median type. Roads with a divided median were defined to include the following median types: flush unpaved, raised curb, barrier median, and TWLTL.

In recognition of the varying influence of median type on crash frequency (as suggested by the rates in Table 4-1), it was determined that only the Hadi models for undivided urban streets were appropriate for inclusion in this chapter. These models are shown as Equations 4-2 and 4-4; they can be used to compute the expected frequency of crashes on undivided two-lane and four-lane streets, respectively.

$$C_{U,2} = 0.25\ ADT^{0.9137}\ (1000\ L)^{0.9330}\ e^{B_{U,2}} \qquad \textbf{(4-2)}$$

with,

$$B_{U,2} = -11.415 - 0.0489\,(W_l + W_{ps}) - 0.0201\ V + 0.056\ N_i - 0.0342\ W_{us} \qquad \textbf{(4-3)}$$

where:
$C_{U,2}$ = frequency of injury crashes on undivided two-lane urban streets, crashes/yr;
W_l = lane width, ft;
W_{ps} = paved shoulder width, ft;
V = speed limit, mph;
N_i = number of intersections; and
W_{us} = unpaved shoulder width, ft.

$$C_{U,4} = 0.25\ ADT^{0.8317}\ (1000\ L)^{0.8831}\ e^{B_{U,4}} \qquad \textbf{(4-4)}$$

with,

$$B_{U,4} = -9.584 - 0.1037\ W_l - 0.0150\ V + 0.0395\ N_i - 0.3318\ I_{curb} \qquad \textbf{(4-5)}$$

where:
$C_{U,4}$ = frequency of injury crashes on undivided four-lane urban streets, crashes/yr; and
I_{curb} = presence of curb (1 if curb present, 0 otherwise).

The models developed by Hadi et al. (*3*) estimate injury crash frequency only; they do not include PDO or fatal crash frequency. Hadi et al. prepared separate models for estimating total crash frequency and fatal crash frequency. Analysis of these models indicates that the estimates obtained from Equations 4-2 and 4-4 should be inflated by 1 percent (i.e., multiplied by 1.01) to obtain an estimate of severe crash frequency.

The sign of the constant associated with any single variable in the model's linear terms (i.e., those variables in Equations 4-3 and 4-5) indicates the correlation between a change in the variable value and crash frequency. For example, the negative sign of the constant associated with speed limit V in either equation indicates that crash frequency is lower on roads with a higher speed limit. This trend is likely a reflection of the fact that higher speed roads are typically built to have more generous design element sizes and more forgiving roadside safety features.

The characteristics of the data used by Hadi et al. (*3*) are summarized in Table 4-2. They used four years of crash data to develop their safety prediction models. They did not report the number of road segments reflected in the database; however, the discussion suggests that it reflects road segments on the Florida state highway system.

Hauer Models

The models described in this section are developed from the equations reported by Hauer et al. (*4*). They used negative binomial regression analysis to develop safety prediction models for urban, four-lane undivided streets. These models are categorized by crash severity and crash location (i.e., off-the-road or on-the-road). The models, shown as Equations 4-6 through 4-12, can be used to compute the expected frequency of severe crashes.

$$C = C_{off-road} + C_{on-road} \tag{4-6}$$

where:

$C_{off\text{-}road}$ = frequency of severe off-road crashes on undivided streets, crashes/yr; and
$C_{on\text{-}road}$ = frequency of severe on-road crashes on undivided streets, crashes/yr.

The frequency of off-road crashes is computed using Equations 4-7 through 4-9. The equation as published was intended to estimate crashes in one direction of travel. The constant "0.36" in Equation 4-7 has been increased by a factor of 2.0 over the value recommended by Hauer et al. such that the estimated quantity reflects total crashes for *both* directions of travel.

$$C_{off-road} = 0.36 \left(\frac{ADT}{10{,}000} \right)^{0.815} L\, e^{B + C} \tag{4-7}$$

with,

$$B = -0.069 \frac{ADT}{10{,}000} + 0.148\, I_{curb} + (1.0 - I_{curb}) \ln(0.76 + 0.083\, SWC) - 0.184\, I_{twltl} \tag{4-8}$$

and,

$$C = 0.230\,(1 - I_{park}) + \frac{320.9}{R} + 0.252\, I_{V \le 30} + 0.318\, I_{V \ge 45} \tag{4-9}$$

where:

SWC = shoulder width category (1 if 0 to 1 ft, 2 if 2 to 3 ft, 3 if 4 to 6 ft, 4 if 7 to 9 ft, 5 if 10 to 11 ft, 6 if over 11 ft);
R = curve radius, ft;
ln(x) = natural log of x;
I_{twltl} = presence of two-way left-turn lane (1 if lane is present, 0 otherwise);
I_{park} = presence of on-street parking (1 if parking is present, 0 otherwise);
$I_{V \leq 30}$ = low speed limit (1 if speed limit is 30 mph or less, 0 otherwise); and
$I_{V \geq 45}$ = high speed limit (1 if speed limit is 45 mph or more, 0 otherwise).

The frequency of on-road crashes is computed using Equations 4-10 through 4-12. The constant "0.90" in Equation 4-10 has been increased by a factor of 2.0 over the value reported by Hauer et al. such that the estimated quantity reflects total crashes for *both* directions of travel.

$$C_{on-road} = 0.90\ L \left[\left(\frac{ADT}{10{,}000} \right)^{1.830} A\ e^{B} \ + \ 0.0205\ D_{d,b} \right] \qquad \textbf{(4-10)}$$

with,

$$B = -0.111 \frac{ADT}{10{,}000} + 0.113\ I_{curb} + (1.0 - I_{curb}) \ln(0.96 + 0.040\ SWC) - 0.229\ I_{twltl} \qquad \textbf{(4-11)}$$

and,

$$A = \frac{1}{150} \left(2\ e^{-0.059\ P_t} + 0.017\ P_t \right) \left(e^{-2298/R} + \frac{343.8}{R} \right) \left(V^{2.066}\ e^{-0.0689\ V} \right) \qquad \textbf{(4-12)}$$

where:

$D_{d,b}$ = density of business or commercial driveways (two-way total), driveways/mi; and
P_t = percent trucks represented in ADT, %.

As with the previous models, the signs of the linear terms in Equations 4-8, 4-9, 4-11, and 4-12 can be interpreted to indicate the change in crash frequency associated with a change in the corresponding variable. An increase in the value of any variable with a positive coefficient corresponds to an increase in crash frequency.

The characteristics of the data used by Hauer et al. (*4*) are summarized in Table 4-2. Specifically, they used four years of crash data from one state to develop their safety prediction models. Their database reflected the crash history for 122 miles of roadway.

Bonneson and McCoy Models

The models described in this section were developed from the equations reported by Bonneson and McCoy (5). They used negative binomial regression analysis to develop models for estimating crash frequency for urban streets. These models differ from those previously described because they yield the combined frequency of crashes that occur on the segment *and* at unsignalized public street intersections. For this reason, the estimates obtained from these models are referred to herein as "mid-signal" crash frequencies.

Three models were developed by Bonneson and McCoy–one model for each of the following median types: undivided, TWLTL, or raised curb. These models are shown as Equations 4-13 through 4-20; they can be used to compute the expected frequency of severe mid-signal crashes.

$$C_R^* = ADT^{0.910}\,(5280\,L)^{0.852}\,e^{B_R}\,(1.0 - 0.01\,PDO) \tag{4-13}$$

with,

$$B_R = -15.162 - 0.296\,I_{b/o} - 0.596\,(1.0 - I_{b/o}) + 0.00478\,(D_d + D_{uia})\,I_{b/o} + 0.0255\,PDO \tag{4-14}$$

where:

C_R^* = frequency of severe, mid-signal crashes on urban streets with a raised-curb median, crashes/yr;
$I_{b/o}$ = business or office land use (1 if business or office, 0 otherwise);
D_d = driveway density (two-way total), driveways/mi;
D_{uia} = unsignalized public street approach density (two-way total), approaches/mi; and
PDO = property-damage-only crashes as a percentage of total crashes (= 68 percent).

$$C_T^* = ADT^{0.910}\,(5280\,L)^{0.852}\,e^{B_T}\,(1.0 - 0.01\,PDO) \tag{4-15}$$

with,

$$B_T = -15.162 + 0.018\,I_{b/o} - 0.093\,(1.0 - I_{b/o}) + 0.00478\,(D_d + D_{uia})\,I_{b/o} + 0.0255\,PDO \tag{4-16}$$

where:

C_T^* = frequency of severe, mid-signal crashes on urban streets with a TWLTL, crashes/yr.

$$C_{U,b/o}^* = ADT^{0.910}\,(5280\,L)^{0.852}\,e^{B_{U,b/o}}\,(1.0 - 0.01\,PDO) \tag{4-17}$$

with,

$$B_{U,b/o} = -15.162 + 0.570\,I_{park} + 0.00478\,(D_d + D_{uia}) + 0.0255\,PDO \tag{4-18}$$

where:

$C^*_{U,b/o}$ = frequency of severe, mid-signal crashes on undivided urban streets serving a business or office land use, crashes/yr.

$$C^*_{U,r/i} = ADT^{1.931}\,(5280\,L)^{0.852}\,e^{B_{U,r/i}}\,(1.0 - 0.01\,PDO) \qquad \textbf{(4-19)}$$

with,

$$B_{U,r/i} = -25.666 + 0.570\,I_{park} + 0.0255\,PDO \qquad \textbf{(4-20)}$$

where:

$C^*_{U,r/i}$ = frequency of severe, mid-signal crashes on undivided urban streets serving residential or industrial land use, crashes/yr.

The PDO terms in Equations 4-13, 4-15, 4-17, and 4-19 were added to convert the total crash estimate into a severe crash estimate. The data used by Bonneson and McCoy to calibrate their model consisted of 68 percent PDO crashes. This factor should be used in the aforementioned equations to yield the desired severe crash frequency.

As with the previous models, the signs of the linear terms in Equations 4-14, 4-16, 4-18, and 4-20 can be interpreted to indicate the change in crash frequency associated with a change in the corresponding variable. An increase in the value of any variable with a positive coefficient corresponds to an increase in crash frequency.

The characteristics of the data used by Bonneson and McCoy *(5)* are summarized in Table 4-2. Specifically, they used three years of crash data from cities in two states to develop their safety prediction models. Their database reflected the crash history for 189 street segments totaling 78.6 miles.

Comparison of Crash Models

The models described in the previous sections are compared in this section. The objective of this comparison is to determine which model or models are reasonable in their prediction of severe crash frequency. To facilitate this comparison, the models are examined over a range of traffic volume levels. The models were grouped by median type and adjacent land use. The values of other model variables were set at typical values for urban streets. These values are listed in Table 4-3.

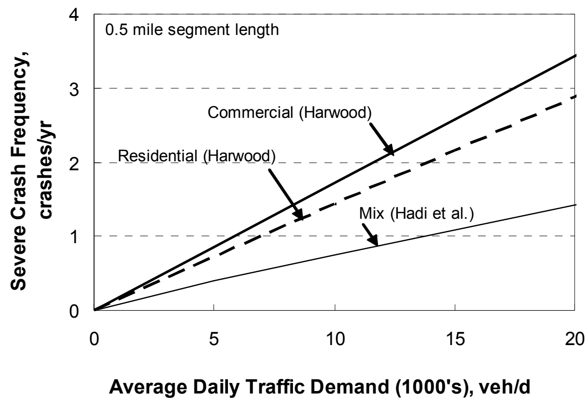
Table 4-3. Typical Values Used for Urban Street Model Comparison.

Model Variable	Typical Value	Safety Prediction Model Developer			
		Harwood (2)	Hadi et al. (3)	Hauer et al. (4)	Bonneson& McCoy (5)
Segment length, mi	0.5	✔	✔	✔	✔
Driveway density, driveways/mile	50 for undivided 50 for TWLTL 25 for raised curb	✔	--	✔	✔
Number of intersections	5	--	✔	--	--
Truck percentage	6.0	✔	--	✔	--
Adjacent land use [1]	varied	✔	--	✔	✔
Outside edge of pavement	curb	--	✔	✔	--
Shoulder width, [2] ft	1.5	✔	✔	✔	--
Lane width, ft	12	--	✔	--	--
Curve radius, ft	infinite (i.e., tangent)	--	--	✔	--
Presence of curb parking	no	--	--	✔	✔
Presence of TWLTL	varied	✔	--	✔	✔
Presence of median	varied	✔	--	--	✔
Speed limit, mph	35 residential and industrial; 40 business and office	--	✔	✔	--

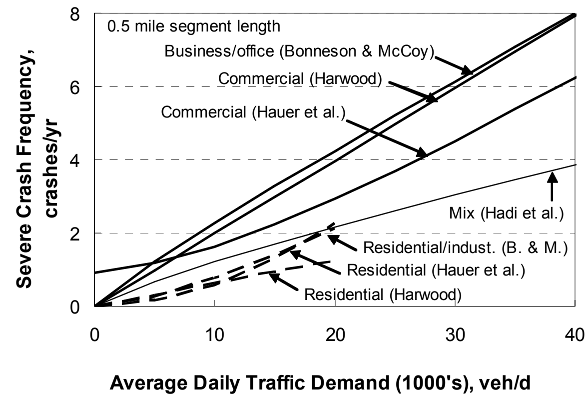
Notes:
1 - Land use categories: business (or commercial), office, industrial, and residential.
2 - It is assumed that curb-and-gutter is provided on the typical urban street instead of a shoulder. This cross section is assumed to have an equivalent "shoulder" width of 1.5 ft.

The expected crash frequencies obtained from the various models are shown in Figures 4-1, 4-2, and 4-3. The model developer and land use adjacent to the street for a given trend line is indicated in each figure. Several trends can be seen in these figures. First, the figures indicate that crash frequency increases in a nearly linear manner with traffic volume. Second, streets in residential areas typically experience fewer crashes than those in business (or commercial) areas. In this regard, there is a mild correlation between land use and driveway traffic demand, where driveways in business areas tend to have more traffic than those in residential areas.

When comparing across Figures 4-1, 4-2, and 4-3, the relationship between land use and crash frequency can be seen to vary widely. This variability reflects the mild correlation between land use and driveway activity. Finally, there is a tendency for more crashes to occur on a four-lane street than a two-lane street for common land use and traffic volume. This trend is likely due to the four-lane cross section's increased exposure to adjacent-lane crashes.

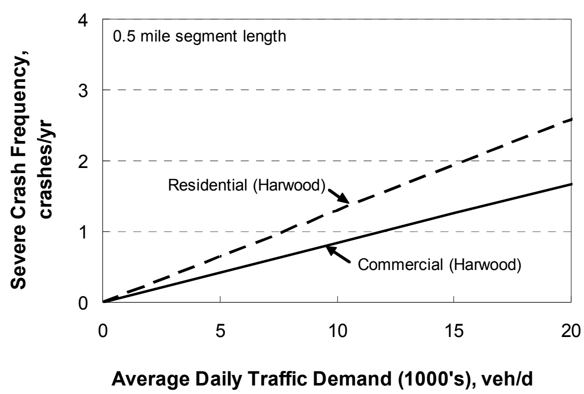


a. Two-Lane Streets.

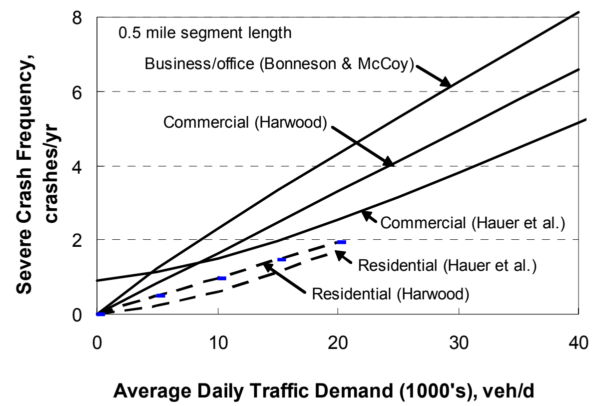


b. Four-Lane Streets.

Figure 4-1. Severe Crash Frequency for Undivided Streets.



a. Two-Lane Streets.



b. Four-Lane Streets.

Figure 4-2. Severe Crash Frequency for Streets with TWLTL.

The correlation between median type and crash frequency was evaluated by examination of all the models shown in Figure 4-2. For a common volume, land use, and lanes, streets with a raised-curb median consistently have fewer severe crashes than those with undivided cross sections. In contrast, crash frequencies for undivided streets and those with a TWLTL are fairly similar. If an overall average of the trend lines is taken, a street in a business or commercial area with a TWLTL may experience a slightly lower crash frequency than an undivided street in the same area. These trends do not extend to streets with a TWLTL that are located in residential areas. They are likely a reflection of the significant amount of driveway activity that occurs in business or commercial areas (relative to residential areas) and the fact that the TWLTL provides storage for turning vehicles.

The models listed in Table 4-3 estimate the frequency of crashes that involve at least one vehicle. Hence, the estimate obtained includes crashes involving pedestrians or bicycles with automobiles, trucks, or buses. Given that these crashes represent a relatively small portion of the total number of crashes, it is likely that AMFs derived from these models do not accurately reflect the relationship between a design element and the frequency of pedestrian or bicycle-related crashes.

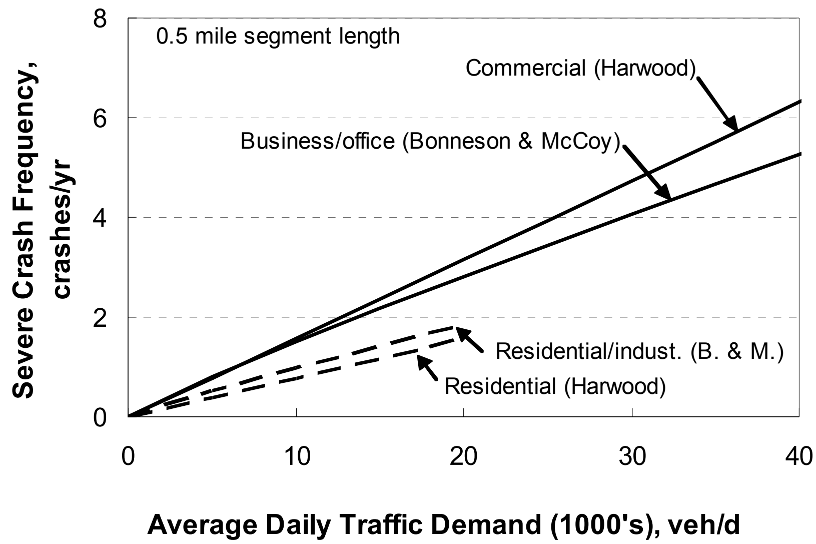


Figure 4-3. Severe Crash Frequency for Four-Lane Streets with Raised-Curb Median.

ACCIDENT MODIFICATION FACTORS

This part of the chapter describes various accident modification factors that are related to the design of an urban street. The various factors examined are organized into the following categories: roadway geometric design, roadside design, and access control.

Geometric Design

This section describes AMFs related to the geometric design of an urban street. Topics specifically addressed are listed in Table 4-4. Many geometric design components or elements are not listed in this table (e.g., grade) that are also likely to have some correlation with severe crash frequency. However, a review of the literature did not reveal useful quantitative information describing these effects. The list of available AMFs for urban street geometric design is likely to increase as new research in this area is undertaken.

Table 4-4. AMFs Related to Geometric Design of Urban Streets.

Section	Accident Modification Factor		
Horizontal alignment	Horizontal curve radius		
Cross section	Lane width TWLTL median type	Shoulder width Curb parking	Median width Uncurbed cross section

In some instances, an AMF is derived from a safety prediction model as the ratio of "segment crash frequency with a changed condition" to "segment crash frequency without the change." In other instances, the AMF is obtained directly from a before-after study. Occasionally, crash rates reported in the literature were used to derive an AMF.

All of the AMFs described in this section are developed to yield a value of 1.0 when the associated design component or element represents a "typical" condition. Deviation from this base condition to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Horizontal Alignment

This subsection describes AMFs related to the urban street's horizontal alignment. The only horizontal alignment AMF found in the literature applies to horizontal curve radius for undivided urban streets. This AMF was derived from the safety prediction model developed by Hauer et al. (*4*). It is shown as Equation 4-21 and reflects one modification from the original model form. Specifically, the constants "2.30" and "0.781" are added for the reasons offered in the next paragraph. The use of Equation 4-21 requires knowledge of the proportion of off-road crashes. The crash data analyzed by Hauer et al. reflect a proportion of off-road crashes equal to 0.14.

$$AMF_{cr} = 2.30\left(e^{-2298/R} + \frac{343.8}{R}\right)[1 - P_{off-road}] + 0.781\,(e^{320.9/R})\,P_{off-road} \quad \textbf{(4-21)}$$

where:
AMF_{cr} = curve radius accident modification factor; and
$P_{off\text{-}road}$ = proportion of crashes that occur off the roadway.

The structure of Hauer's original equation is such that the use of a tangent as the base condition yields AMF values of less than 1.0 for curved segments, which is illogical. Equation 4-21 represents a modified version of the original equation. The modification reflects the use of a 1300 ft base condition such that AMF values equal 1.0 or more for all radii. The constants "2.30" and "0.781" were added to Equation 4-21 to reflect this modification. This relationship is shown in Figure 4-4 for a proportion of off-road crashes $P_{off\text{-}road}$ equal to 0.14.

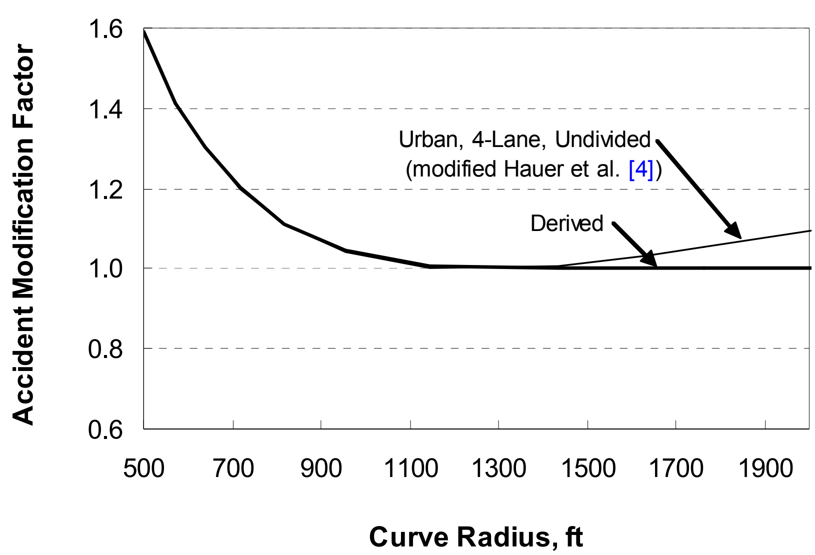


Figure 4-4. Curve Radius AMF for Urban Streets.

For radii less than 1300 ft, Equation 4-21 is consistent with AMFs for rural highways in that crash frequency increases with decreasing radii. However, for radii larger than 1300 ft, it suggests that crash frequency increases as radius increases. This increase is not consistent with rural highway AMFs and additional research is needed to determine if it is true. Until then, Equation 4-21 is recommended for radii less than 1300 ft, and an AMF of 1.0 is recommended for larger radii. The recommended AMF is shown in Figure 4-4 as the thick bold line labeled “derived.”

Cross Section

Lane Width. A relationship between lane width and crash frequency was derived from the safety prediction models developed by several researchers. This derivation is described in the Geometric Design section of Chapter 3. When applied to urban streets, the following equation was developed for computing the lane width AMF for the base combination of lanes and median type (i.e., four-lane, raised-curb median street). The base condition lane width for this AMF is 12 ft.

$$AMF_{lw,b} = e^{-0.040\,(W_l\,-\,12)} \tag{4-22}$$

where:

$AMF_{lw,b}$ = lane width accident modification factor for the base combination of lanes and median type (i.e., four-lane, raised-curb median street).

Equation 4-22 can be combined with the following equation to obtain the lane width AMF for other combinations of lanes and median type.

$$AMF_i = (AMF_b - 1.0)\,\frac{P_i}{P_b} + 1.0 \tag{4-23}$$

where:

AMF_i = adjusted accident modification factor for lane and median combination *i*;
AMF_b = accident modification factor for base combination of lanes and median type;
P_i = proportion of “related” crashes that occur on roadways with lane and median combination *i*; and
P_b = proportion of “related” crashes that occur on roadways with the base combination of lanes and median type.

The values of P_i and P_b correspond to the crashes influenced by the AMF, and are both represented as a proportion of all crashes. The crash distribution used for P_i is that reflecting the crash history of the roadway type corresponding to b_i. The distribution used for P_b is that reflecting a specified base combination of lanes and median type. For this analysis, a street with a four-lane, raised-curb median street was chosen to represent the base combination. The proportion of influential crashes for the base combination P_b is 0.24.

The AMF that results from the combination of Equations 4-22 and 4-23 is:

$$AMF_{lw,i} = (e^{-0.040\,(W_l\,-\,12)} - 1.0)\,\frac{P_i}{0.24} + 1.0 \tag{4-24}$$

It can be tailored to a desired number of lanes and median type by using the variable P_i. Values for this variable are listed in Table 4-5. The proportions in this table (and subsequent similar tables) were obtained from the crash database maintained by the State of Texas, Department of Public Safety.

Table 4-5. Crash Distribution for Urban Street Lane Width AMF.

Area Type	Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Urban	Undivided or TWLTL	Single-vehicle run-off-road, either side same direction sideswipe, multiple vehicle opposite direction	2	0.25
			4	0.18
			6	0.14
	Raised curb	Single-vehicle run-off-road, either side same direction sideswipe	4	0.24
			6	0.27

The AMFs obtained from Equation 4-24 are illustrated in Figure 4-5. They are labeled "derived" and have a bold line weight. They are compared to the AMFs developed by other researchers. The trend is one of increasing AMF value with a reduction in lane width. For a given lane width, the derived AMF for two-lane streets is larger than that for four-lane streets. There is general agreement between the derived AMFs and those reported in the literature. Significant differences between AMFs are likely a reflection of bias (due to colinearity) in the original models.

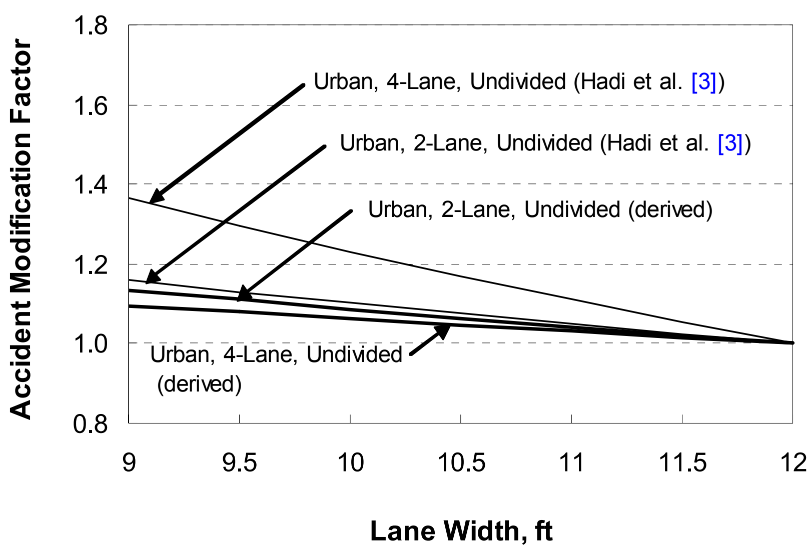


Figure 4-5. Lane Width AMF for Urban Streets.

Shoulder Width. The following equation represents the shoulder width AMF for the base combination of lanes and median type (i.e., four-lane, raised-curb median street). It was derived in a similar manner as the lane width AMF (as described in Chapter 3). The base condition shoulder width is 1.5 ft.

$$AMF_{sw,b} = e^{-0.014\,(W_s\,-\,1.5)} \qquad \textbf{(4-25)}$$

where:

$AMF_{sw,\,b}$ = shoulder width accident modification factor for the base combination of lanes and median type (i.e., four-lane, raised-curb median street); and

W_s = shoulder width, ft.

Equation 4-25 can be combined with Equation 4-23 to obtain the shoulder width AMF for other combinations of lanes and median type. The proportion of influential crashes for the base combination P_b is 0.088. The resulting AMF is:

$$AMF_{sw,i} = \left(e^{-0.014\,(W_s\,-\,1.5)} - 1.0 \right) \frac{P_i}{0.088} + 1.0 \qquad \textbf{(4-26)}$$

It can be tailored to the desired number of lanes and median type by using the variable P_i. The value of this variable is selected from Table 4-6.

Table 4-6. Crash Distribution for Urban Street Shoulder Width AMF.

Area Type	Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Urban	Undivided or TWLTL	Single-vehicle run-off-road, either side	2	0.17
			4	0.10
			6	0.054
	Raised curb	Single-vehicle run-off-road, right side	4	0.088
			6	0.087

The AMFs obtained from Equation 4-26 are illustrated in Figures 4-6 and 4-7. They are labeled “derived” and have a bold line weight. They are compared to the AMFs developed by other researchers.

Figure 4-6. Shoulder Width AMF for Two-Lane Urban Streets.

Figure 4-7. Shoulder Width AMF for Four-Lane Urban Streets.

The Harwood (*2*) AMF was derived from Equation 4-1. Values for this AMF vary by shoulder width as follows: 1.0 for 0 to 4 ft, 0.95 for 5 to 7 ft, 0.90 for 8 ft or more. The AMF extracted from the Hauer et al. (*4*) model was derived from Equations 4-8 and 4-11. Values for this AMF vary by shoulder width as follows: 1.0 for 2 to 3 ft; 1.05 for 4 to 6 ft; 1.09 for 7 to 9 ft; and 1.14 for 10 to 11 ft.

With one exception, the trends in Figures 4-6 and 4-7 show a decreasing AMF value with an increase in shoulder width. For a given shoulder width, the AMF for two-lane streets is smaller than that for four-lane streets. There is general agreement between the derived AMFs and those reported in the literature. The one exception is the AMF developed by Hauer et al. (*4*) for four-lane streets. It suggests that wider shoulders are associated with increased crashes which is contrary to common belief. It is likely that this trend is indirectly a result of a negative correlation between shoulder width and other variables that were not included in the safety prediction model.

Median Width. AMFs for median width were developed from safety prediction models developed by Hadi et al. (*3*) and Bowman et al. (*6*). All AMFs are based on models developed for urban, four-lane and six-lane, divided streets. Equation 4-27 was developed to represent these two AMFs. The base condition for this AMF is a 16 ft median width. The coefficients used in the model are listed in columns 4 through 6 of Table 4-7.

$$AMF_{mw} = \frac{b_0\,(e^{b_1 W_m^{b_2}} - 1.0) + 1.0}{b_0\,(e^{b_1 16^{b_2}} - 1.0) + 1.0} \qquad \textbf{(4-27)}$$

where:

AMF_{mw} = median width accident modification factor; and
W_m = median width, ft.

Table 4-7. Coefficient Values for Median Width on Urban Streets.

Model Source	Roadway Type	Crash Severity	Base Coefficients			Equivalent b_1 Coefficients
			b_0	b_1	b_2	
Hadi et al. (*3*)	Urban, 4-lane, divided	Injury	1.0	-0.0325	0.5	-0.0325
	Urban, 6-lane, divided	Injury	1.0	-0.0501	0.5	-0.0501
Bowman et al. (6)	Urban, 4 & 6-lane, divided	All	1.0	-0.0275	1.0	-0.2008
					Weighted Average:	**-0.041**

Figure 4-8 illustrates the AMFs listed in Table 4-7. The two AMFs attributed to Hadi et al. (*3*) are in good agreement. They are also in good agreement with the median width AMF derived for rural highways (see Chapter 3). The AMF derived from the Bowman et al. (*6*) model is in less agreement and indicates a much greater sensitivity to median width.

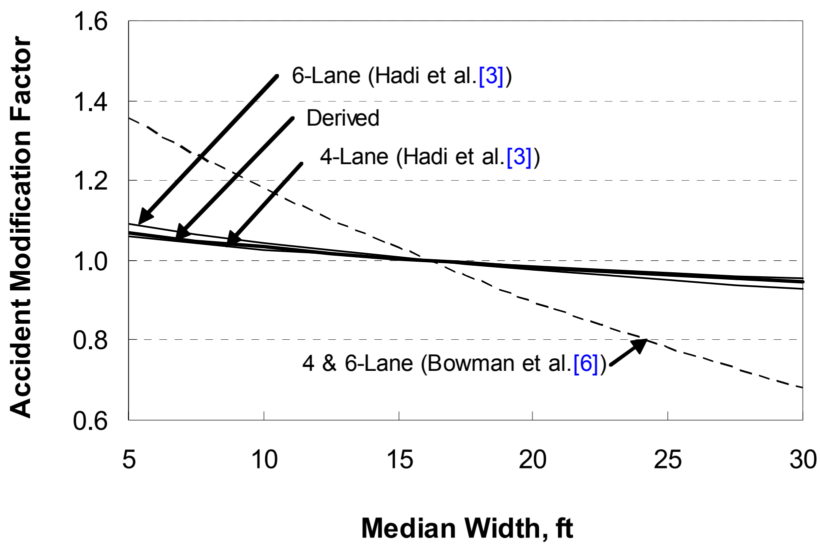


Figure 4-8. Median Width AMF for Urban Streets.

The derived AMF relationship is shown in Figure 4-8. It is based on Equation 4-27 with b_0 = 1.0, b_2 = 0.5, and the weighted average of the equivalent b_1 coefficients listed in columns 7 and 8 of Table 4-7 (where the weight w for each coefficient is equal to its reciprocal squared [i.e., $w = 1/b^2$]). The equivalent b_1 coefficient for the Bowman AMF was derived from a regression analysis using Equation 4-27 with b_0 = 1.0 and b_2 = 0.5 for median widths ranging from 5 to 30 ft. The AMFs obtained from Equation 4-27 (using the base coefficients) served as the dependent variable.

TWLTL Median Type. As noted in the discussion of Figures 4-1 and 4-2, the difference between a street with a TWLTL and an undivided street is not readily apparent in terms of their expected crash frequency for a given traffic volume and land use. This lack of a clear difference is likely due partly to differences in driveway traffic activity. Harwood et al. (*7*) developed an AMF to capture this effect as it relates to the addition of a TWLTL to a two-lane rural highway. This AMF is shown as Equation 4-28.

$$AMF_T = 1.0 - AMF_{target} \, P_D \, P_{LT/D} \tag{4-28}$$

with,

$$P_D = \frac{0.0047\, D_d + 0.0024\, D_d^2}{1.199 + 0.0047\, D_d + 0.0024\, D_d^2} \tag{4-29}$$

where:

AMF_T = TWLTL accident modification factor;
AMF_{target} = AMF for crash types directly influenced by the addition of a TWLTL (= 0.70);
P_D = driveway-related crashes as a proportion of total crashes; and
$P_{LT/D}$ = left-turn accidents susceptible to correction by a TWLTL as a proportion of driveway-related crashes (estimated as 0.50).

The effectiveness of the TWLTL, when added to an undivided street, has been researched by many individuals. In their review of previously conducted before-after studies, Bonneson and McCoy *(5)* noted that crash reduction factors associated with the TWLTL ranged from 28 to 44 percent (i.e., AMFs of 0.56 to 0.72). The AMFs for target crashes (i.e., left-turn, rear end, etc.) were found to vary from 0.53 to 0.67. One of the reports they reviewed was a before-after study conducted by Thakkar *(8)*. His database included information for 15 sites that had four through lanes and 16 sites that had two through lanes. However, 11 of the four-through-lane sites and four of the two-through-lane sites were selected for conversion because they were high-accident locations (HALs). Table 4-8 lists the data reported by Thakkar for the sites that were *not* HALs.

The AMFs listed in Table 4-8 suggest that a TWLTL has a smaller safety benefit (i.e., larger AMF) when applied to a street with two through lanes than when applied to a street with four through lanes. They also confirm that certain crash types (i.e., left-turn, rear end, and sideswipe) are more likely to be reduced by a TWLTL than other crash types (e.g., fixed object).

Equation 4-30 was derived to combine the concepts expressed in Equation 4-28 with the AMFs listed in Table 4-8. Equation 4-31 is used to estimate the proportion of target crashes P_{target}.

$$AMF_T = (AMF_{target} - 1.0)\, P_{target} + 1.0 \tag{4-30}$$

with,

$$P_{target} = 1 - e^{-0.008\, D_{d,b/o}\,(n_l + 1)} \tag{4-31}$$

where:

P_{target} = target crashes as a proportion of total crashes;
n_l = number of through lanes; and
$D_{d,\,b/o}$ = density of driveways serving business or office land uses (two-way total), driveways/mi.

Table 4-8. Before-After TWLTL Crash Data Analysis.

Site	Thru. Lanes	ADT		Total Crashes			Target Crashes		
		Before	After	Before	After	Predicted	Before	After	Predicted
4	4	12,300	13,500	116	58	127.3	30	11	32.9
7	4	15,000	15,000	52	42	52.0	13	3	13.0
11	4	23,000	22,000	116	56	111.0	46	30	44.0
12	4	23,000	24,200	66	89	69.4	17	24	17.9
				Total:	245	359.7	**Total:**	68	107.8
					***AMF$_T$*:**	**0.68**		***AMF$_{target}$*:**	**0.63**
1	2	4200	3800	4	0	3.6	3	0	2.7
2	2	6900	6900	15	11	15.0	2	1	2.0
3	2	8200	8550	48	43	50.0	13	11	13.6
4	2	8400	8500	9	5	9.1	3	2	3.0
5	2	8500	9200	14	4	15.2	13	3	14.1
6	2	8550	8600	7	6	7.0	3	4	3.0
7	2	10,000	11,050	27	23	29.8	18	11	19.9
8	2	10,600	11,500	8	13	8.7	2	4	2.2
9	2	10,700	10,900	19	18	19.4	0	1	0.0
11	2	11,700	10,300	45	52	39.6	7	10	6.2
13	2	15,900	16,700	108	109	113.4	29	29	30.5
14	2	16,150	16,700	52	35	53.8	21	13	21.7
				Total:	319	364.7	**Total:**	89	118.8
					***AMF$_T$*:**	**0.87**		***AMF$_{target}$*:**	**0.75**

As in Equation 4-29, the driveway density term is included in Equation 4-31 to reflect the correlation between driveway density and the distribution of crashes. The number-of-lanes term is included for similar reasons. Logically, a wider cross section is likely to be correlated with an increase in target crashes. The $P_{b/o}$ term is included because an analysis of driveway influence (see the section titled Access Control) indicated that driveways associated with a residential land use do not have a significant correlation with crash frequency. The constant "-0.008" represents a best-fit calibration coefficient that was derived using the AMFs in Table 4-8 and an estimated average driveway density of 30 and 50 driveways/mi for the two-lane and four-lane cross sections, respectively. These average densities were obtained from the report by Harwood *(2)*.

The AMF for adding a TWLTL to an urban street is shown in Figure 4-9. The trend line attributed to Harwood et al. *(7)* is based on Equations 4-28 and 4-29. It indicates that the TWLTL is more effective at reducing crashes as driveway density increases. The thick trend lines are based on Equations 4-30 and 4-31. A similar trend with driveway density is reflected in these equations.

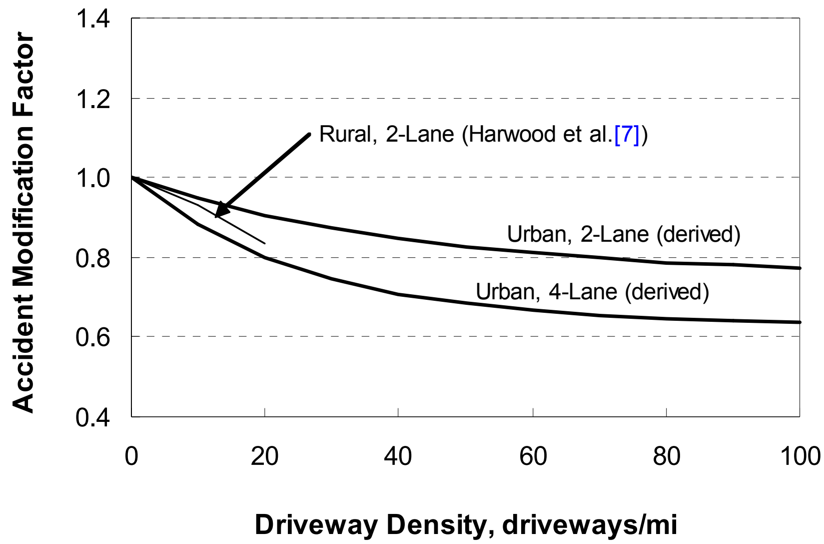


Figure 4-9. TWLTL Median Type AMF for Urban Streets.

Curb Parking. AMFs for curb parking were developed from the safety prediction model developed by Bonneson and McCoy *(5)* and from data reported by McCoy et al. *(9)*. The safety prediction model was described previously. The AMF from the McCoy data was developed for this chapter. McCoy et al. examined the relationship between number of through lanes, adjacent land use, and crash frequency on urban streets in Nebraska. The crash frequencies they reported for streets with parallel parking are provided in Table 4-9. The corresponding AMFs for parallel parking are shown in the last column of the table.

Table 4-9. AMFs for Parallel Parking on Urban Streets.

Data Source	Through Lanes	Percent Business or Office Land Use [1]	Crash Frequency by Type		AMF_{pp} [2]
			Parallel Parking	Total	
Bonneson & McCoy *(5)*	4	100	not available	not available	1.77
McCoy et al. *(9)*	2	75	19	41	1.86
		20	51	130	1.65
	>2	69	66	198	1.50
		35	66	307	1.27

Notes:
1 - "Business or Office Land Use" is defined to include retail, service, office, and medical facilities.
2 - AMF for Bonneson & McCoy *(5)* was computed as $e^{0.57}$. For the other data source, it is computed as *total*/(*total* − *parallel parking*).

The data in Table 4-9 indicate that there is a relationship between land use, through lanes, and the parallel parking AMF. This relationship is shown in Figure 4-10. The AMFs in the last column of Table 4-9 are shown as data points. The trends indicate that crash frequency increases linearly with the percentage of business or office land use. Crashes are also more frequent on two-lane streets than multilane streets.

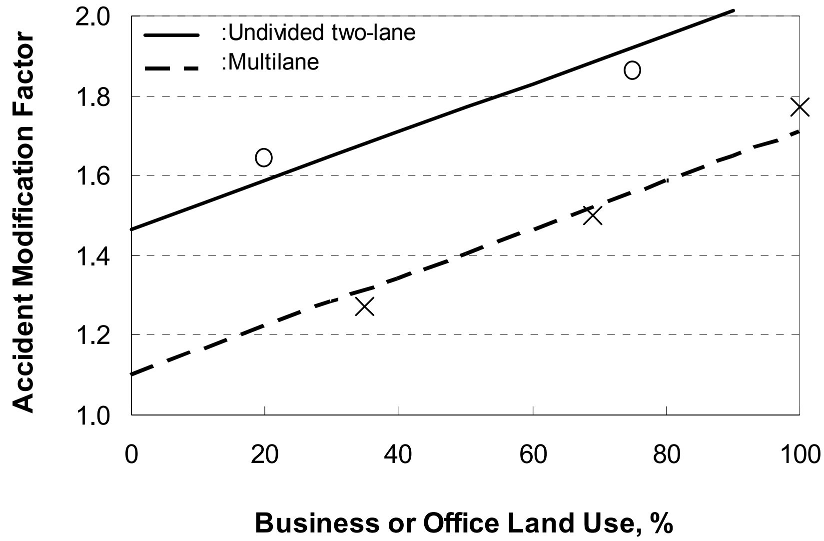


Figure 4-10. Parking AMF for Urban Streets.

The "business or office" land use percentages in Table 4-9 for McCoy et al. (*9*) reflect the percentage of painted stalls at each study site. The locations with a low percentage of business or office use almost always had unpainted parking stalls. Those locations with a high percentage of business or office land use almost always had painted stalls. Regardless, the variation among AMFs in Table 4-9 is more likely correlated with land use, and associated parking activity, than whether the stall was painted.

Box (*10*) recently reviewed several studies that contrasted angle parking with parallel parking. He concluded that streets with angle parking had crash rates that were 1.5 to 3 times larger than those streets with parallel parking. Some of the data he used to form this conclusion are summarized in Table 4-10.

Based on the trends described in the preceding paragraphs, the following AMF was derived to reflect the correlation between curb parking and crash frequency. The base condition for this AMF is "no parking."

$$AMF_{pk} = 1 + P_{pk}\,(B_{pk} - 1) \qquad \textbf{(4-32)}$$

$$B_{pk} = (1.10 + 0.365\, I_{u2} + 0.609\, P_{b/o})\ [(f_{ap/pp} - 1.0)\, P_{ap} + 1.0] \qquad \textbf{(4-33)}$$

where:

AMF_{pk} = parking presence accident modification factor;

P_{pk} = proportion of street segment length with parallel or angle parking (= 0.5 L_{pk} / L);

L_{pk} = curb miles allocated to parking, mi;
I_{u2} = indicator variable for cross section (1 for two-lane street; 0 otherwise);
$P_{b/o}$ = for that part of the street with parking, the proportion that has business or office as an adjacent land use;
$f_{ap/pp}$ = ratio of crashes on streets with angle parking to those on streets with parallel parking (see Table 4-10); and
P_{ap} = for that part of the street with parking, the proportion with angle parking.

Table 4-10. Comparison of Angle Parking with Parallel Parking.

Data Source	Roadway Type	Parking Crash Rate		Units	$f_{ang/pp}$
		Angle Crash Rate	Parallel Crash Rate		
Box (*10*) Table 6	Collector	211	84	parking crash/mi	2.51
	Local	147	56	parking crash/mi	2.63
Box (*10*) Table 9	Retail	15.0	7.5	crash/mi	2.00
	Retail/mixed	9.9	5.4	crash/mi	1.83
	Residential	5.3	1.9	crash/mi	2.79
Box (*10*) Table 10	Collector	13.5	9.4	crash/mi	1.44
	Local	14.6	4.8	crash/mi	3.04
McCoy et al. (*9*) Tables 12 & 13	Painted stalls	14.4	5.0	crash/bvmh/stall [1]	2.88
	Unpainted stalls	5.4	2.8	crash/bvmh/stall [1]	1.93
				Average:	**2.34**

Note:
1 - Crashes per 10 billion vehicle-mile-hours/stall.

Uncurbed Cross Section. The relationship between the presence of an outside curb and crash frequency was derived from models developed by Hadi et al. (*3*) and by Hauer et al. (*4*). In addition to the models shown as Equations 4-2 and 4-4, Hadi et al. (*3*) developed models for estimating: (1) total crash frequency for two-lane and four-lane undivided streets, (2) total crash frequency for six-lane divided streets, and (3) injury crash frequency for six-lane divided streets. Each of these models included an indicator variable for curb presence. The AMFs extracted from the urban, four-lane, undivided models indicated that crashes would be *higher* on an uncurbed cross section. This trend is illogical and likely the result of some correlation among model variables. This AMF was not considered further. The AMFs extracted from the remaining Hadi models are represented by Equation 4-34, in combination with Table 4-11.

$$AMF_{no\ curb} = (e^{b(I_{curb} - 1)} - 1.0)\,P_s + 1.0 \qquad \textbf{(4-34)}$$

where:
$AMF_{no\ curb}$ = uncurbed cross section accident modification factor, and
I_{curb} = presence of curb (1 if curb present, 0 otherwise).

Table 4-11. Coefficient Values for Uncurbed Cross Section on Urban Streets.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s	Coefficient b	AMF
Hadi et al. (3)	Urban, 2-lane, undivided	All	All	1.0	0.1707	0.84
	Urban, 6-lane, divided	Injury	All	1.0	0.2202	0.80
		All	All	1.0	0.1671	0.85
Hauer et al. (4)	Urban, 4-lane, undivided	Injury	Off-road crashes	0.14	0.225 [1]	0.91
			On-road crashes	--	0.074 [1]	
		All	Off-road crashes	0.14	0.291 [2]	0.86
			On-road crashes	--	0.132 [2]	

Notes:
1 - Values listed are derived from Equations 4-8 and 4-11 using a curb as the base condition and a flush shoulder of 2 to 3 ft representing the no curb condition (i.e., $SWC = 2$) (e.g., $0.225 = \ln(1.16) - \ln[0.76 + 0.083 \times SWC]$).
2 - Values listed are derived from equations reported by Hauer et al. (4).

The models presented by Hauer et al. were used to develop the following equation for estimating an uncurbed cross section AMF. The equation coefficients and subset proportions are listed in Table 4-11.

$$AMF_{no\ curb} = e^{b_{on\ road}(I_{curb} - 1)}(1 - P_s) + e^{b_{off\ road}(I_{curb} - 1)} P_s \qquad \textbf{(4-35)}$$

Column 4 of Table 4-12 lists AMFs that are computed using the coefficients and subset proportions listed in Table 4-11. Equation 4-34 was used with the Hadi coefficients and proportions. Equation 4-35 was used with the Hauer coefficients and proportions. The AMF values are in general agreement and range from 0.80 to 0.91.

Table 4-12. AMFs for Uncurbed Cross Section on Urban Streets.

Model Source	Roadway Type	Crash Severity	AMFs from Table 4-11 [1]	Subset Proportion, P_s [2]	AMFs from Equ. 4-35 [3]
Hadi et al. (3)	Urban, 2-lane, undivided	Injury	not available	0.17	0.91
		All	0.84		0.85
Hauer et al. (4)	Urban, 4-lane, undivided	Injury	0.91	0.10	0.92
		All	0.86		0.86
Hadi et al. (3)	Urban, 6-lane, divided	Injury	0.80	0.17	0.91
		All	0.85		0.85

Notes:
1 - AMFs are computed using the coefficients and subset proportions listed in Table 4-11 with the corresponding equation (i.e., Equation 4-34 or 4-35).
2 - Proportions are obtained from Table 4-6 and represent single vehicle run-off-road crashes in Texas.
3 - AMFs are computed using Equation 4-35 with the subset proportions in column 5 and the coefficients (in Table 4-11) from the Hauer model.

The subset distribution of run-off-road crashes listed in Table 4-6 was used to facilitate comparison of the Hauer coefficients with those from the Hadi models. Specifically, this distribution was used with Equation 4-35 and Hauer's coefficients, to estimate the AMFs in the last column of Table 4-12. The AMFs shown in this column compare favorably with those obtained from the Hadi models. This finding suggests that there is general agreement among the AMF model sources on the correlation between curb exclusion and crash frequency—provided that differences in median type and number-of-lanes are accounted for using the appropriate subset crash proportions. Thus, the following equation is rationalized to offer a reasonable method for estimating the uncurbed cross section AMF for urban streets:

$$AMF_{no\ curb} = e^{-0.074}\,(1 - P_{off\text{-}road}) + e^{-0.225}\,P_{off\text{-}road} \qquad \textbf{(4-36)}$$

The proportion of off-road crashes $P_{off\text{-}road}$ is obtained from Table 4-6.

Roadside Design

This section describes AMFs related to the roadside design of an urban street. Topics specifically addressed are listed in Table 4-13. Many roadside design components or elements that are not listed in this table (e.g., ditch shape) are also likely to have some correlation with crash frequency on the urban street segment. However, a review of the literature did not reveal that their effect has been quantified by previous research. The list of available AMFs for urban street roadside design is likely to increase as new research in this area is undertaken.

Table 4-13. AMFs Related to Roadside Design of Urban Streets.

Section	Accident Modification Factor
Cross section	Utility pole offset
Appurtenances	see text

AMFs for roadside safety appurtenances are not described in this chapter because they generally do not exist. The safety literature related to roadside appurtenances has focused on the information needed to evaluate the cost-effectiveness of installing individual appurtenances at specific locations. The information is very detailed due to the design and operational complexity of various appurtenances and the influence of site-specific conditions on their performance. Moreover, safety appurtenances may sometimes increase crash frequency while, more importantly, reducing the severity of the crash. For these reasons, the safety benefits derived from an appurtenance are typically estimated on an individual, case-by-case basis using the techniques described in the *Roadside Design Guide* (*11*).

Cross Section

This subsection is devoted to the presentation of AMFs related to the roadside cross section of an urban street. A review of the literature indicates that horizontal clearance, side slope, utility pole offset, and bridge width may have some influence on crash frequency. However, with one exception, these correlations have not been quantified for urban streets. Rather, the focus of previous research has been on rural, two-lane highways. The one exception is utility pole offset. This research included crashes on urban roadways in the database. This subsection describes an AMF for utility pole offset and "density" (where pole density relates to pole frequency per unit length of roadway).

The correlation between utility pole offset, pole density, and crash frequency was evaluated by Zegeer and Parker (*12*). They developed a safety prediction model using data from four states. The model included average pole offset, traffic volume, and pole density as independent variables. The AMF derived from this model is described in Equation 4-37. Details of the model are described in Table 4-14. The base condition is 50 poles/mi and a pole offset of 2.0 ft.

$$AMF_{pd} = (f_p - 1.0)\, P_s + 1.0 \tag{4-37}$$

with,

$$f_p = \frac{(0.0000984\, ADT + 0.0354\, D_p)\, W_o^{-0.6} - 0.04}{0.0000649\, ADT + 1.128} \tag{4-38}$$

where:

AMF_{pd} = utility pole accident modification factor;
D_p = utility pole density (two-way total), poles/mi; and
W_o = average pole offset from nearest edge of traveled way, ft.

Table 4-14. Coefficient Values for Utility Pole Offset on Urban Streets.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Subset Proportion, P_s
Zegeer & Parker (*12*)	Urban and rural roads	All	Single-vehicle collision with pole	0.022

The subset proportions appropriate for Texas urban street applications are provided in Table 4-15. Equation 4-37 yields an AMF that ranges from 1.02 at a pole density of 90 poles/mi and offset of 2.0 ft to 0.96 at a pole density of 20 poles/mi and offset of 30 ft. The AMF is more sensitive to pole offset than it is to density or average daily traffic volume.

Table 4-15. Crash Distribution for Urban Street Utility Pole Offset AMF.

Area Type	Median Type	Crash Type Subset	Through Lanes	Subset Proportion
Urban	Undivided or TWLTL	Single-vehicle collision with pole	2	0.042
			4	0.036
			6	0.017
	Raised curb	Single-vehicle collision with pole	4	0.045
			6	0.046

Appurtenances

As discussed previously, AMFs for roadside safety appurtenances are not described in this chapter. The safety literature related to roadside appurtenances has focused on the information needed to evaluate the cost-effectiveness of installing individual appurtenances at specific locations. Most of the research conducted in this area has been incorporated into a complex and comprehensive procedure for evaluating appurtenances on a case-by-case basis. This procedure is outlined in a report by Mak and Sicking (*13*) and automated in the *Roadside Safety Analysis Program* (RSAP) (*14*).

RSAP can be used to evaluate alternative roadside safety appurtenances on individual street segments. The program accepts as input information about the street segment geometry and traffic characteristics. It also allows the analyst to describe the roadside cross section, location of fixed objects, and safety appurtenance design. Table 4-16 summarizes the various RSAP inputs. The output from RSAP includes an estimate of annual crash frequency as well as the road-user costs associated with these crashes. The crash reduction potential realized by adding a roadside safety appurtenance (or changing the roadside cross section) can be estimated by specifying the changed condition as an "alternative."

Table 4-16. RSAP Input Data Requirements.

Design Category	Design Component	Design Element	
General	--	Area type (urban/rural) One-way/two-way Segment length	Functional class Speed limit
Traffic characteristics	--	Traffic volume (ADT) Traffic growth factor	Truck presence
Geometric design	Horizontal alignment	Direction of curve	Radius
	Vertical alignment	Grade	
	Cross section	Divided/undivided Lane width Median type	Number of lanes Shoulder width Median width
Roadside design	Cross section	Foreslope Parallel ditches	Backslope Intersecting slopes
	Fixed object	Offset Type (wood pole, headwall, etc.) Width	Side of roadway Spacing
	Safety appurtenances	Offset Type (barrier, cushion, etc.) Length	Side of roadway Flare rate Width

Access Control

This section is devoted to AMFs related to the access control elements of an urban street. A review of the literature indicates that median type, adjacent land use (as a surrogate for driveway activity), and driveway density have an influence on crash frequency. The correlation between median type and crashes is addressed previously in the sections titled Safety Prediction Models and TWLTL Median Type. The correlation between land use and crashes is also addressed in the part titled Safety Prediction Models. This section on access control describes the correlation between driveway density and crash frequency.

Three accident modification factors for driveway density were identified during the literature review. The first AMF for driveway density was derived from the Bonneson and McCoy model described previously. It is based on driveways in business or office areas. The form of this AMF is:

$$AMF_{dd} = e^{0.00478\,(D_d - D_{base})} \tag{4-39}$$

where:

AMF_{dd} = driveway density accident modification factor; and
D_{base} = base driveway density, driveways/mi.

A second AMF was derived from the crash rates developed by Harwood (*2*). These rates were reported previously in Table 4-1. Trends in the driveway density adjustment factors suggest that they reflect a base condition of about 50 driveways/mi. The AMF was computed from the rates in Table 4-1 as:

$$AMF_{dd} = 1.0 + \frac{Drive}{Base_i} \tag{4-40}$$

A third AMF for driveway density was derived from a model developed by Sawalha and Sayed (*15*). It is based on driveways in business areas. The form of this AMF is:

$$AMF_{dd} = e^{0.0105\,(D_d - D_{base})} \tag{4-41}$$

The three AMFs are compared in Figure 4-11 for a base driveway density of 50 driveways/mi. The trends in the data indicate that the relationship between driveway density and crashes varies widely. This variation is likely due to the fact that driveway activity level (i.e., traffic demand) is not used in the models. A surrogate for this activity is land use such that driveways in business and office areas are likely to have significant traffic activity as to be associated with frequent driveway-related crashes. In contrast, driveways in residential and office areas tend to have relatively low volume and a corresponding minimal association with crash frequency.

A best-fit AMF for driveway density was derived from the relationships shown in Figure 4-11. The form of this AMF is:

$$AMF_{dd} = e^{0.008\,(D_{d,b/o} - 50)} \tag{4-42}$$

where:

$D_{d,\,b/o}$ = density of driveways serving business or office land uses, driveways/mi.

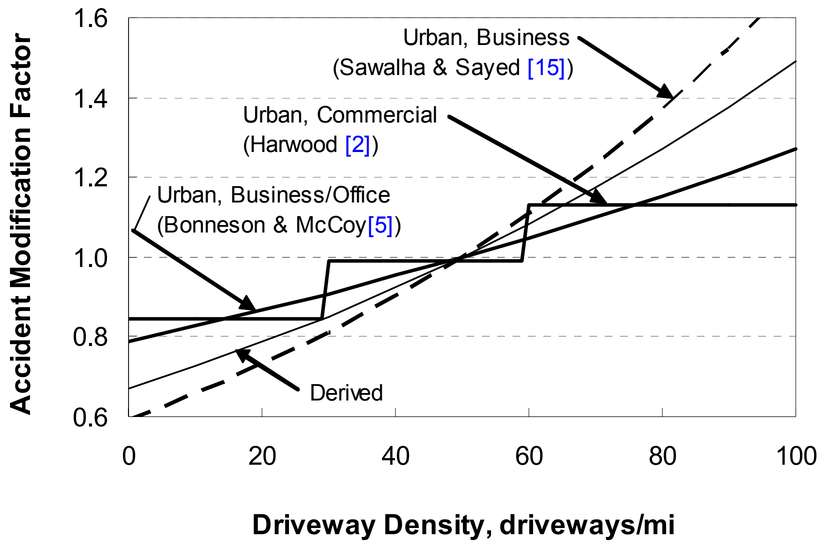


Figure 4-11. Driveway Density AMF for Urban Streets.

Other Adjustment Factors

This section describes AMFs related to features of the street that are not categorized as related to geometric design, roadside design, or access control. Specifically, the AMFs described in this section address truck presence and speed limit.

Truck Presence

AMFs for truck presence were derived from the Hauer and the Harwood safety prediction models. The AMF derived from the Hauer model is shown in Equation 4-43. It reflects a base truck percentage of 6.0 percent. The proportion of off-road crashes in the database used by Hauer et al. is 0.14.

$$AMF_{tk} = (f_{tk} - 1.0)\,(1 - P_{off\text{-}road}) + 1.0 \qquad \textbf{(4-43)}$$

with,

$$f_{tk} = \frac{2\,e^{-0.059\,P_t} + 0.017\,P_t}{1.506} \qquad \textbf{(4-44)}$$

where:

AMF_{tk} = truck presence accident modification factor; and
$P_{off\text{-}road}$ = proportion of crashes that occur off the roadway.

An AMF was also developed from the Harwood model using the rates reported previously in Table 4-1. Trends in this truck presence adjustment factor suggest that it reflects a base condition of about 6.0 percent. The AMF computed from the rates in Table 4-1 is:

$$AMF_{tk} = 1.0 + \frac{Truck}{Base_i} \qquad \textbf{(4-45)}$$

The two AMFs described above are compared in Figure 4-12 for a range of truck percentages. The trends in the data indicate that increasing truck percentage is associated with fewer crashes. It is likely that an increase in trucks does not make the street safer; rather, it probably indicates that drivers are more cautious when there are many trucks in the traffic stream.

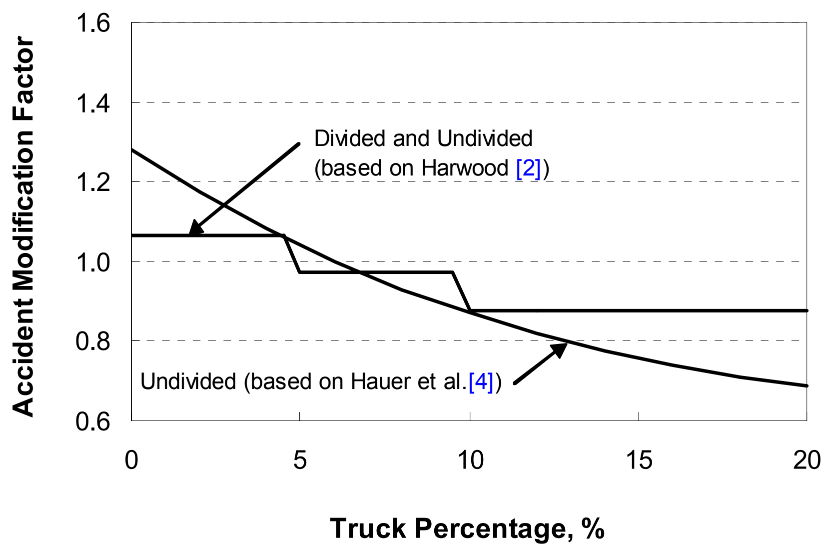


Figure 4-12. Truck Presence AMF for Urban Streets.

Speed Limit

AMFs for speed limit were derived from the Hauer and the Hadi safety prediction models. The AMF from the Hauer model is shown as Equation 4-46. It reflects a base speed limit of 40 mph. The proportion of off-road crashes in the database used by Hauer et al. is 0.14.

$$AMF_{sl} = e^{(0.252\, I_{V \le 30} + 0.318\, I_{V \ge 45})}\, P_{off-road} + 1.15\left(V^{2.066}\, e^{-0.0689\, V}\right)(1 - P_{off-road}) \qquad \textbf{(4-46)}$$

where:

AMF_{sl} = speed limit accident modification factor.

An AMF for speed limit was also derived from the Hadi model. It is shown in Equation 4-47 for a base speed limit of 40 mph. The value of each coefficient *b* is provided in Table 4-17.

$$AMF_{sl} = e^{b(V - 40)} \qquad \textbf{(4-47)}$$

Table 4-17. Coefficient Values for Speed Limit for Urban Streets.

Model Source	Roadway Type	Crash Severity	Subset of Influenced Crash Types	Coefficient *b*
Hadi et al. (*3*)	Urban, 2-lane, undivided	Injury	All	-0.0201
	Urban, 4-lane, undivided	Injury	All	-0.0155
	Urban, 4-lane, divided	Injury	All	-0.0295

The two AMFs are compared in Figure 4-13. The trends in the data indicate that higher speed limits are associated with fewer severe crashes. Although not shown in the figure, a similar trend was found by Bowman et al. (*6*). It is likely that an increase in speed limit does not make the street safer; rather, it probably indicates that streets with a higher speed limit tend to have a more generous design. Moreover, if just vehicle-pedestrian crashes were evaluated, their frequency would likely be found to *increase* with an increase in speed limit.

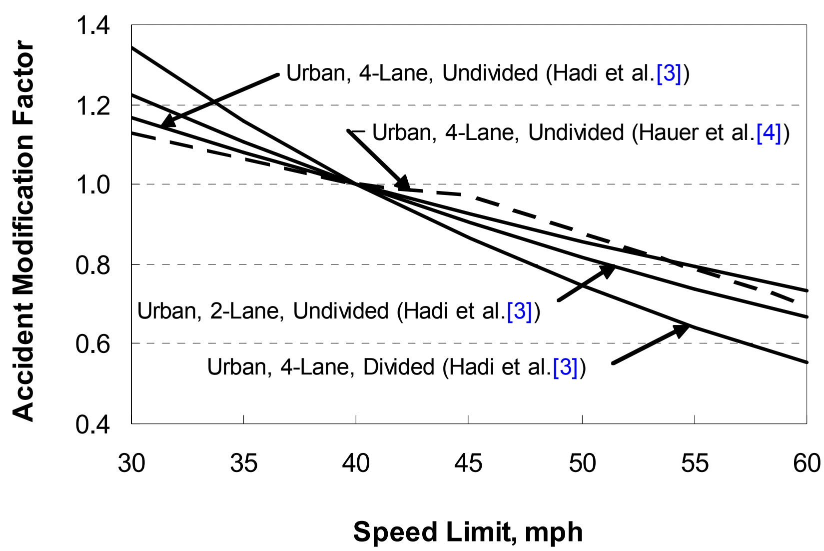


Figure 4-13. Speed Limit AMF for Urban Streets.

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Chapter 5
Interchange Ramps

Page

INTRODUCTION

Access to and from grade-separated facilities is achieved using interchange ramps. These ramps are essentially free-flow facilities with one or more lanes that allow ramp traffic to merge with freeway traffic while maintaining a relatively high speed. Ramps can connect two freeway facilities, a freeway to an arterial, or two arterial roadways. Ramps are configured in a variety of shapes to accommodate heavy turn movements and topography. They are associated with more significant crash risk because of the significant speed change that occurs along their length, often coupled with horizontal curves of near-minimum radius and relatively steep grade changes. These attributes complicate the ramp driving task. In recognition of this complexity, driveways and intersections are rarely allowed along the length of a ramp because the associated turning traffic would unnecessarily compound the complexity of the ramp driving task.

The development of a safe, efficient, and economical interchange ramp design typically reflects the consideration of a variety of design alternatives. Interchange design is particularly challenging because of the interchange's significant right-of-way requirement and construction cost. A variety of techniques exist for estimating the operational benefits of interchange alternatives; many are automated through software tools. Techniques for estimating construction and right-of-way costs are also available to the designer. Unfortunately, techniques for estimating the safety benefits of alternative designs are not as readily available. This chapter summarizes information in the literature that can be used to estimate the crash frequency associated with various ramp design alternatives.

Objective

The objective of this chapter is to synthesize information in the literature that quantitatively describes the relationship between various interchange ramp design components and safety. This information is intended to provide a basis for the development of a procedure for estimating the safety benefit of alternative designs. This procedure is documented in Chapter 5 of the *Roadway Safety Design Workbook* (*1*).

The presentation consists of an examination of safety prediction models and accident modification factors (AMFs). Safety prediction models provide an estimate of the expected crash frequency for a typical interchange ramp. They include variables for the volume of the conflicting traffic streams. They also include variables for other factors considered to be correlated with crash frequency (e.g., lane width, grade, etc.). One or more AMFs can be multiplied by the expected crash frequency obtained from the prediction model to produce an estimate of the expected crash frequency for a specific interchange ramp.

Scope

This chapter addresses the safety of the interchange ramp and its terminal with the main lanes (i.e., the speed-change lane). For this reason, the crashes addressed herein are referred to as "ramp-related" or "speed-change-lane-related" crashes. Crashes that occur on the ramp approach to the ramp-crossroad terminal are considered ramp related. However, crashes that occur within the curb-line limits of the ramp-crossroad terminal are not considered to be ramp related. At this time, the safety of frontage road segments is not addressed.

When available, safety relationships that estimate the frequency of *severe* (i.e., injury or fatal) crashes are given preference for inclusion in this document. This preference is due to a wide variation in reporting threshold among cities and states. This variation complicates the extrapolation of crash trends found in one location to another location. Moreover, it can confound the development of safety prediction models using data from multiple agencies. Reporting threshold is strongly correlated with the number of property-damage-only (PDO) crashes found in a crash database. Agencies with a high reporting threshold include relatively few PDO crashes in their database and vice versa. As a consequence, the total crash frequency for a given ramp will be high if it is located in an area with a low reporting threshold. This problem is minimized when crash data analyses, comparisons, and models are based on data pertaining only to severe crashes.

The relationships described in this chapter address the aggregation of all vehicle-related crashes on interchange ramps and speed-change lanes. Relationships that focus on vehicle-pedestrian and vehicle-bicycle crashes on ramps will be added in future updates to this chapter.

Most interchanges in use today are one of two types: diamond or partial cloverleaf (i.e., "parclo"). Typical variations of both types are shown in Figure 5-1.

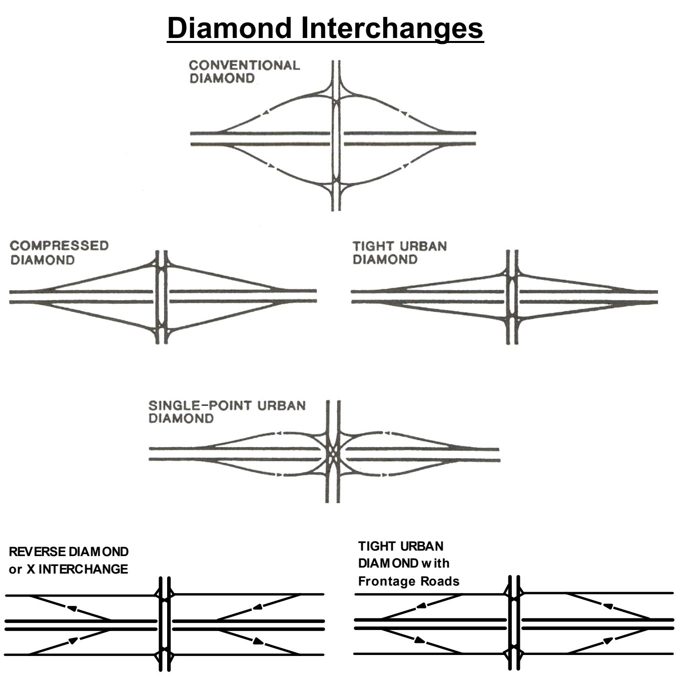


Figure 5-1. Commonly Used Interchange Types.

The conventional, compressed, and tight urban diamond interchange types are commonly used in Texas. In most instances, frontage roads are used with this interchange type and the ramps join with the frontage road in advance of, or just beyond, the crossroad. The more common arrangement is to have the ramp merge with the frontage road in advance of its intersection with the crossroad such that the combined traffic stream is served by the frontage-road intersection.

Parclo interchanges are much more common in states not having frontage roads; however, several are in service in Texas. The parclo A and parclo B types have two loop ramps that each serve one turn movement and ramps in all four quadrants. In contrast, the 2-quad parclo variations serve two turn movements on each loop ramp and, thereby, minimize right-of-way requirements in two quadrants. The generous allocation of ramps at the parclo A and parclo B results in a high capacity for all interchanging movements. On the other hand, the 2-quad parclo variations tend to have fewer ramps but at the cost of lower traffic carrying capacity.

Overview

This chapter documents a review of the literature related to interchange ramp safety. The focus is on quantitative information that relates severe crash frequency to various geometric design components of the ramp. The review is not intended to be comprehensive in the context of referencing all relevant works that discuss ramp safety. Rather, the information presented herein is judged to be the most current information that is relevant to ramp design in Texas. It is also judged to be the most reliable based on a review of the statistical analysis techniques used and the explanation of trends.

Where appropriate, the safety relationships reported in the literature are compared herein, with some interpretation offered as an explanation for any differences noted. The relationships are typically presented as reported in the literature; however, the names or the units of some variables have been changed to facilitate their uniform presentation in this document.

This chapter is envisioned to be useful to design engineers who desire a more complete understanding of the relationship between various ramp design components and severe crash frequency. As noted previously, it is also intended to serve as the basis for the development of the safety evaluation procedure described in Chapter 5 of the *Roadway Safety Design Workbook* (*1*).

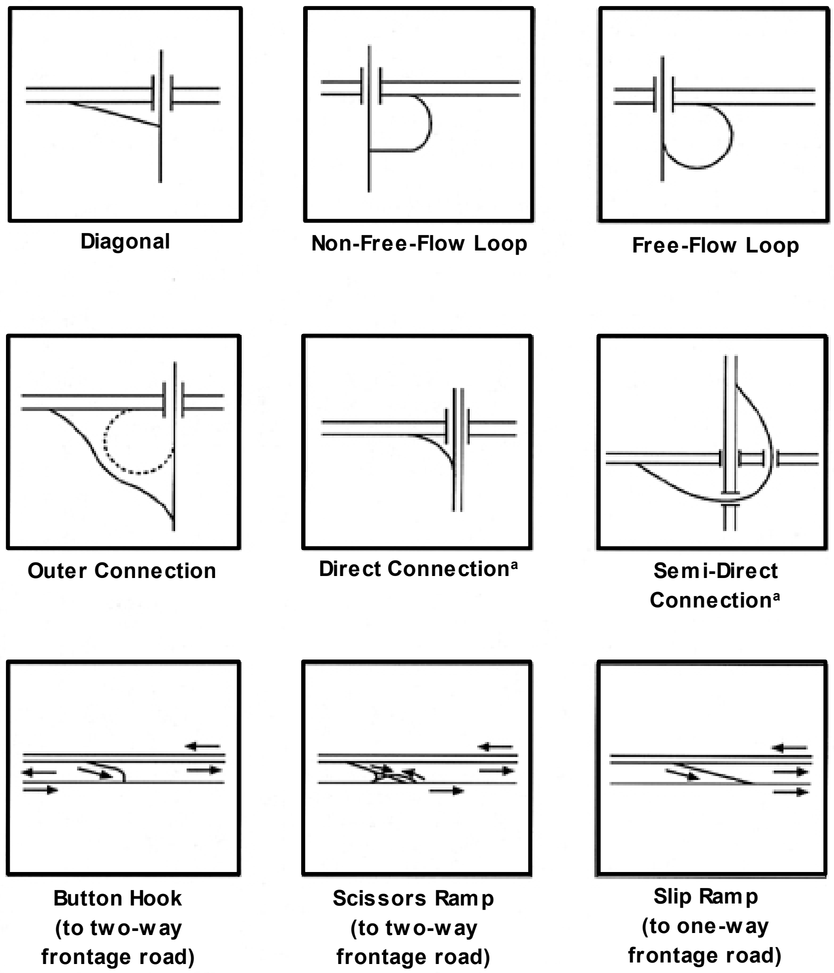
This chapter consists of two main parts. In the first part to follow, several safety prediction models are described. The second part is devoted to the discussion of AMFs. However, there are currently no accident modification factors available for interchange ramps, primarily because of the paucity of safety research on this topic. Hence, this latter part is relatively brief.

SAFETY PREDICTION MODELS

This part of the chapter describes safety prediction models that are applicable to interchange ramps. The first section in this part describes models related to the ramp proper. The second section describes models related to ramp speed-change lanes.

Interchange Ramps

Nine ramp configurations are addressed in this section. Exit ramp variations of each configuration are illustrated in Figure 5-2. Entrance ramp versions have a similar alignment.



[a] When used in directional interchanges

Figure 5-2. Basic Interchange Ramp Configurations. (2)

This section summarizes the findings from two recent research projects that evaluated the safety of interchange ramp configurations. Both of these studies examined the relationship between various elements of the ramp geometry and the interchange environment. In general, they did not find significant correlations between ramp geometry, or interchange environment, and crash frequency. The most significant correlations were found to be with ramp configuration and ramp volume. Of particular note is the finding that ramp length is not correlated with the frequency of ramp-related crash frequency.

Bauer and Harwood Model

The safety prediction model developed by Bauer and Harwood (*2*) is based on data obtained from Washington State for the years 1993 to 1995. It includes data for 533 crashes at 551 interchange ramps. Their model predicts the frequency of severe ramp-related crashes. It is shown below as Equation 5-1.

$$C = 0.0957 \, f_{type} \left(\frac{ADT_R}{1000} \right)^{0.85} \qquad \textbf{(5-1)}$$

where:
C = frequency of severe ramp-related crashes, crashes/yr;
ADT_R = average daily traffic on the ramp, veh/d; and
f_{type} = crash adjustment factor for area type, ramp type, and ramp configuration (see Table 5-1).

Equation 5-1 predicts only those severe crashes that occur on the ramp. The predicted crash frequency does not include crashes associated with the ramp speed-change lanes or those that occur within the curb-line limits of the ramp junction at the crossroad.

The adjustment factor f_{type} in Equation 5-1 is used to adapt the model to alternative combinations of area type, ramp type, and ramp configuration. The regression analysis reported by Bauer and Harwood (*2*) indicated that the factors listed in columns 3 and 6 of Table 5-1 are appropriate for this purpose. The free-flow loop, semi-direct connection, and direct connection ramps were not found to be significantly different, so they are grouped together in the table.

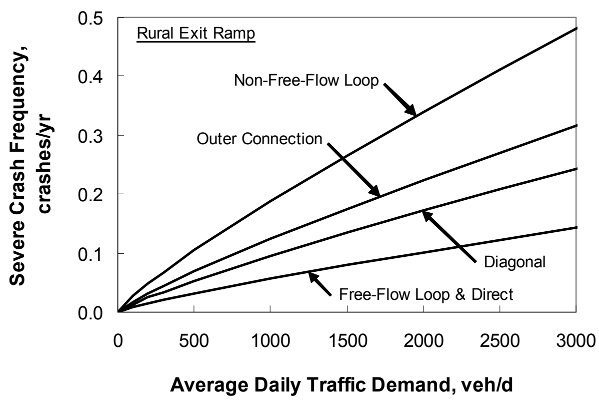
Table 5-1. Crash Adjustment Factors for Equation 5-1.

Area Type: Rural			Area Type: Urban		
Ramp Type	**Ramp Configuration**	f_{type}	**Ramp Type**	**Ramp Configuration**	f_{type}
Exit	Diagonal	1.00	Exit	Diagonal	1.40
	Non-free-flow loop	1.97		Non-free-flow loop	2.77
	Free-flow loop & direct [1]	0.59		Free-flow loop & direct [1]	0.83
	Outer connection	1.30		Outer connection	1.82
Entrance	Diagonal	0.58	Entrance	Diagonal	0.81
	Non-free-flow loop	1.14		Non-free-flow loop	1.60
	Free-flow loop & direct [1]	0.34		Free-flow loop & direct [1]	0.48
	Outer connection	0.75		Outer connection	1.05

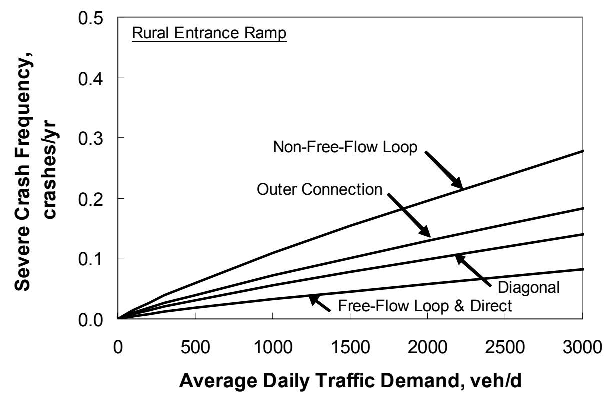
Note:
1 - Category includes free-flow loop, semi-direct connection, and direct connection ramps.

The adjustment factors f_{type} in Table 5-1 vary from 0.34 to 2.77, depending on the combination of attributes associated with a specific ramp. This range is relatively wide and suggests that ramp type and configuration can have a significant impact on safety.

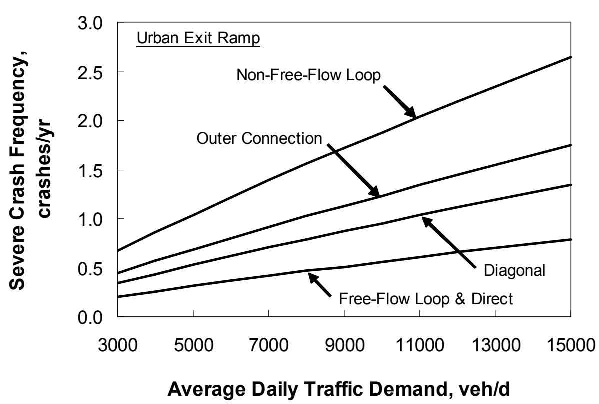
The relationship between severe crash frequency and ramp configuration is shown in Figure 5-3. The trends shown are based on Equation 5-1 and the factors in Table 5-1. The ramp volume range used in the figures reflects typical volume levels in urban and rural environments and are consistent with the volume levels reported by Bauer and Harwood.



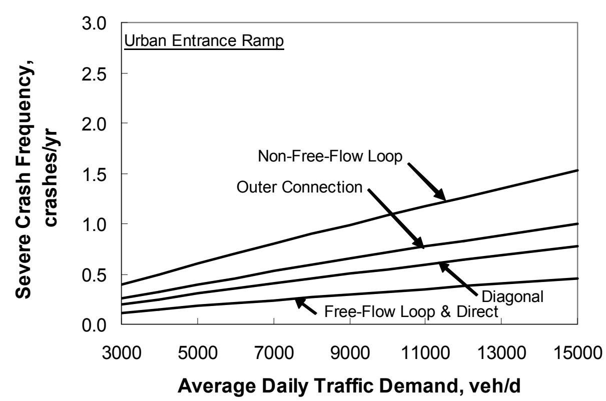
a. Rural Exit Ramps.



b. Rural Entrance Ramps.



c. Urban Exit Ramps.



d. Urban Entrance Ramps.

Figure 5-3. Severe Crash Frequency Predicted by Bauer and Harwood Model.

The trends in Figure 5-3 indicate that non-free-flow loop ramps experience the most crashes. Outer connection ramps experience fewer crashes, followed by diagonal ramps and free-flow-loop ramps. A comparison across ramp types indicate that exit ramps are associated with between 35 and 65 percent more crashes than entrance ramps.

Khorashadi Model

Khorashadi (*3*) examined severe crash frequency at 13,325 interchanges in California. He assembled a database that included information about ramp configuration, ramp volume, and crash frequency. The database represented the crash history for three years and included 24,143 severe crashes. Khorashadi did not develop a regression model similar to Equation 5-1. Rather, he computed the severe crash rate for each ramp configuration. These rates are summarized in Table 5-2; they were obtained from Appendix B of the report by Khorashadi. In general, crash data for 240 interchange ramps underlie each of the crash rates reported in Table 5-2; however, for any single rate, the number ranges from 1 to 2000 ramps.

Table 5-2. Severe Crash Rates Reported by Khorashadi (*3*).

Area Type: Rural			**Area Type: Urban**		
Ramp Type	**Ramp Configuration**	**Crash Rate [1] crashes/mv**	**Ramp Type**	**Ramp Configuration**	**Crash Rate [1] crashes/mv**
Exit	Diagonal	0.183	Exit	Diagonal	0.467
	Non-free-flow loop	0.887		Non-free-flow loop	0.387
	Free-flow loop	0.239		Free-flow loop	0.318
	Semi-direct connection	0.691		Semi-direct connection	0.188
	Direct connection	0.235		Direct connection	0.307
	Button hook	1.670		Button hook	0.568
	Scissor	0.553		Scissor	0.468
	Slip	not available		Slip	0.356
Entrance	Diagonal	0.186	Entrance	Diagonal	0.186
	Non-free-flow loop	0.228		Non-free-flow loop	0.317
	Free-flow loop	0.150		Free-flow loop	0.196
	Semi-direct connection	0.488		Semi-direct connection	0.192
	Direct connection	0.141		Direct connection	0.268
	Button hook	0.254		Button hook	0.229
	Scissor	0.077		Scissor	0.212
	Slip	not available		Slip	0.225

Note:
1 - Crash rate is in units of severe crashes per million ramp vehicles.

Examination of the rates in Table 5-2 indicates that the button hook ramp experiences the most crashes. This configuration is closely followed by the non-free-flow loop ramp. Thereafter, in order of decreasing crash rate, are the scissor ramp and the slip ramp. The diagonal, semi-direct connection, and direct connection ramps tend to experience about the same low crash rate. A comparison across ramp types indicates that exit ramps are associated with between 55 and 65 percent more crashes than entrance ramps. There is no apparent pattern between the crash rates in urban and rural environments.

The severe crash frequency for a specific ramp can be estimated using Equation 5-2 and the rates in Table 5-2.

$$C = 0.000365 \ CR \ ADT_R \tag{5-2}$$

where:

C = frequency of severe ramp-related crashes, crashes/yr; and

CR = severe crash rate (see Table 5-2), crashes/million ramp vehicles.

Comparison of Crash Models

The crash models described in the previous subsections are examined in this subsection to determine if there is general agreement about the relative safety of common ramp configurations. To facilitate this analysis, Equation 5-1 was used to compute severe crash frequencies for a range of volumes typically found in urban and rural environments. These frequencies, shown in Figure 5-3, were used to compute an equivalent crash rate for each area type, ramp type, and ramp configuration. These rates were then compared to those in Table 5-2 for common ramp configurations.

A regression model was used to facilitate the comparison of crash rates. The model uses indicator variables for the various ramp configurations as well as for the two ramp types (i.e., exit/entrance) and the two area types (i.e., urban/rural). The following model form was used:

$$\ln(CR) = b_0 + b_1 I_{diag} + b_2 I_{nff} + b_3 I_{ff} + b_4 I_{oc} + b_5 I_{sd} + b_6 I_{exit} \tag{5-3}$$

where:

$\ln(CR)$ = natural log of severe crash rate;

I_{diag} = indicator variable for diagonal ramp (1 if diagonal ramp; 0 otherwise);

I_{nff} = indicator variable for non-free-flow ramp (1 if non-free-flow ramp; 0 otherwise);

I_{ff} = indicator variable for free-flow ramp (1 if free-flow ramp; 0 otherwise);

I_{sd} = indicator variable for semi-direct connection (1 if semi-direct connection; 0 otherwise);

I_{oc} = indicator variable for outer connection (1 if outer connection; 0 otherwise); and

I_{exit} = indicator variable for exit ramp (1 if exit ramp; 0 if entrance ramp).

An indicator variable for area type was originally included in Equation 5-3 but was not found to be statistically significant, so it was removed from the model. Indicator variables for the button hook, scissor, and slip ramps were also originally included in Equation 5-3 but the predicted rates were not intuitive, so they were separately evaluated. This evaluation is described later in this subsection.

The natural log function is used in Equation 5-3 because it bounds the predicted rates to non-negative values, as is appropriate for the analysis of crash data. The predictive capability of the model is shown in Figure 5-4. The coefficient of determination R^2 of the regression model is 0.55.

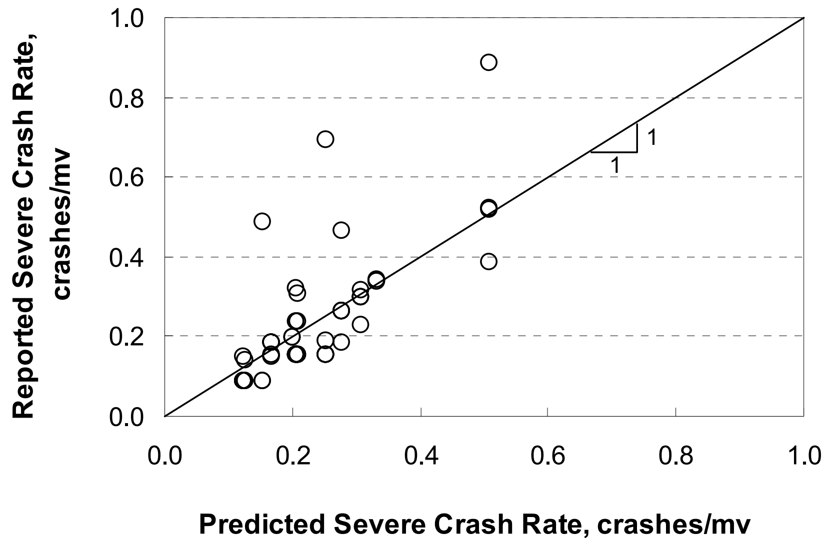


Figure 5-4. Comparison of Predicted and Reported Crash Rates.

As the trends in the data in Figure 5-4 suggest, there is reasonably good agreement between the predicted crash rates and those obtained from the models developed by the two researchers. Three of the crash rates from the Khorashadi database were found to be much higher than predicted by the regression model. These three rates apply to the exit non-free-flow loop, exit semi-direct connection, and the entrance semi-direct connection ramps; all of which are located in a rural environment. These rates were obtained from a total of 85 ramps, which is a relatively small number compared to the other configurations. In fact, the highest rate of 0.887 is based on crashes on only four non-free-flow loop ramps. Hence, these rates likely reflect the occurrence of an atypically large number of crashes on the corresponding ramps in the three-year period. Crash frequency likely returned to values nearer that predicted by the regression model in subsequent years.

The severe crash rates predicted by the calibrated form of Equation 5-3 are listed in Table 5-3. In application, these rates would be used with Equation 5-2 to estimate the severe crash frequency associated with a specific ramp type, ramp configuration, and ramp volume. These rates relate to only those severe crashes that occur on the ramp. The predicted crash frequency does not include crashes associated with the ramp speed-change lanes or those that occur within the curb-line limits of the ramp junction at the crossroad.

These rates listed in Table 5-3 confirm the trends noted in the evaluation of the previous two models. Specifically, that crash rates for entrance ramps are about 60 percent of those for exit ramps. The button hook ramp is associated with the largest crash rate, followed closely by that for the non-free-flow loop. The scissor and slip ramps have the next highest rates followed by the outer connection ramp and the diagonal ramp. The semi-direct connection ramp, the direct connection, and the free-flow loop ramp have the lowest crash rates. It should be noted that crash rates associated with frontage-road ramps have crash rates that are 30 to 100 percent larger than those for the diagonal ramp, a ramp that is common to diamond interchanges in non-frontage-road settings.

Table 5-3. Severe Crash Rates for Various Ramp Configurations.

Ramp Type	Ramp Configuration	Crash Rate [1] crashes/mv	Ramp Type	Ramp Configuration	Crash Rate [1] crashes/mv
Exit	Diagonal	0.277	Entrance	Diagonal	0.168
	Non-free-flow loop	0.507		Non-free-flow loop	0.308
	Free-flow loop	0.205		Free-flow loop	0.124
	Outer connection	0.332		Outer connection	0.201
	Semi-direct connection	0.252		Semi-direct connection	0.153
	Direct connection	0.209		Direct connection	0.127
	Button hook	0.573		Button hook	0.275
	Scissor	0.484		Scissor	0.208
	Slip	0.356		Slip	0.225

Note:
1 - Crash rate is in units of severe crashes per million ramp vehicles.

The crash rates for the button hook, scissor, and slip ramps were not part of the regression analysis using Equation 5-3. These ramps did not follow the trends apparent in the rates for the other ramps so they were evaluated separately. The estimate of the button hook and the scissor ramp crash rates was based on the following equation:

$$CR = \frac{\dfrac{CR_{rural}}{s^2_{rural}} + \dfrac{CR_{urban}}{s^2_{urban}}}{\dfrac{1}{s^2_{rural}} + \dfrac{1}{s^2_{urban}}} \qquad \textbf{(5-4)}$$

where:
s = standard deviation of the average crash rate, crashes/yr.

Equation 5-4 provides a weighted average of the crash rates for the rural and urban area types, for a common ramp type and ramp configuration. The weighting factor is that of the inverse of the variance of the average rate. Thus, the rates with a large amount of uncertainty are given less weight in the computed average rate. The original rates and their corresponding standard deviations are shown in Table 5-4. These standard deviations were obtained from Appendix B of the report by Khorashadi (*3*).

The rates for rural and urban interchanges were combined based on the aforementioned regression analysis wherein it was found that there was no significant difference between the crash rates in rural and in urban areas. Equation 5-4 was applied to the button hook and the scissor ramps because each ramp configuration was found in urban and rural locations. The weighted average rates are listed in Table 5-3. Equation 5-4 could not be applied to the slip ramps because they were only found in urban areas. Hence, the original rates for slip ramps are repeated in Table 5-3.

Table 5-4. Severe Crash Rate Analysis for Frontage-Road Ramps.

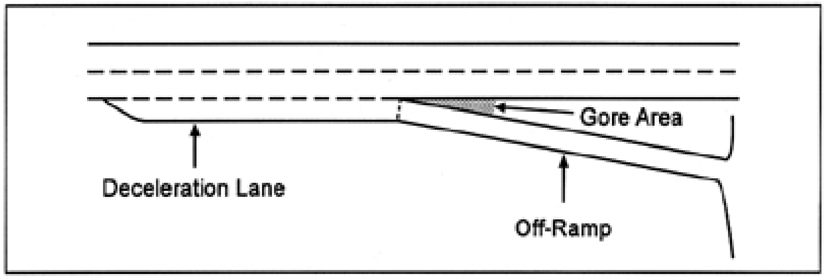
Area Type: Rural				Area Type: Urban			
Ramp Type	**Ramp Configuration**	**Crash Rate [1] crashes/mv**	**Standard Deviation crashes/mv**	**Ramp Type**	**Ramp Configuration**	**Crash Rate [1] crashes/mv**	**Standard Deviation crashes/mv**
Exit	Button hook	1.670	0.774	Exit	Button hook	0.568	0.055
	Scissor	0.553	0.143		Scissor	0.468	0.069
	Slip	not available	--		Slip	0.356	0.191
Entrance	Button hook	0.254	0.098	Entrance	Button hook	0.229	0.024
	Scissor	0.077	0.061		Scissor	0.212	0.035
	Slip	not available	--		Slip	0.225	0.140

Note:
1 - Crash rate is in units of severe crashes per million ramp vehicles.

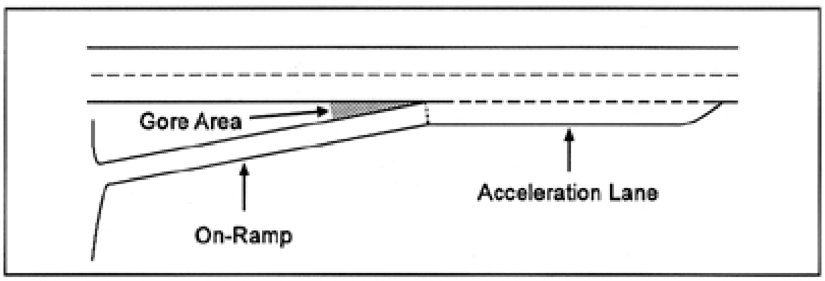
Ramp Speed-Change Lanes

A model for estimating the severe crash frequency associated with the ramp speed-change lane is the subject of this section. The crash rates provided in Table 5-3 do not include crashes that occur in speed-change lanes. Speed-change lanes are the location of a majority of conflicts between ramp traffic and mainlane traffic. Thus, the design of the speed-change lane is likely to have a significant influence on ramp safety. Speed-change lanes associated with an interchange ramp are the subject of this section. These lanes function as either an acceleration lane or a deceleration lane. Both types of speed-change lane are shown in Figure 5-5.

Bauer and Harwood (*2*) developed several models relating crash frequency to the design elements of ramp speed-change lanes. Separate models were developed for predicting total (i.e., property-damage-only plus severe crashes) crashes for acceleration lanes, total crashes for deceleration lanes, and severe crashes for acceleration lanes on diagonal ramps. There was insufficient data in their database to develop severe crash models for deceleration lanes or for other ramp configurations. As a result, the models described in this section are those developed by Bauer and Harwood (*2*) for predicting total crashes. However, an adjustment factor has been added to each model such that it estimates severe crash frequency.



Deceleration Lane and Off-Ramp



On-Ramp and Acceleration Lane

Figure 5-5. Acceleration and Deceleration Speed-Change Lanes. (2)

Bauer and Harwood's model for total crashes in an acceleration lane is:

$$C_{accel} = 0.00702 \left(\frac{ADT_R}{1000} \right)^{0.98} \left(\frac{ADT_M}{1000} \right)^{0.32} e^{(6.88 L_a - 0.59 I_{rural})} (1 - 0.01\, PDO_a) \qquad \textbf{(5-5)}$$

where:

C_{accel} = frequency of severe crashes on acceleration lanes, crashes/yr;
ADT_M = average daily traffic in adjacent freeway lanes (one-way), veh/d
L_a = length of acceleration lane, mi;
I_{rural} = indicator for area type (1 if rural, 0 if urban); and
PDO_a = percent property-damage-only crashes in acceleration lanes (= 52.2).

Bauer and Harwood's model for all crashes in a deceleration lane is:

$$C_{decel} = 0.0261 \left(\frac{ADT_R}{1000} \right)^{1.04} e^{(0.09 W_s - 1.21 I_{rural})} (1 - 0.01\, PDO_d) \qquad \textbf{(5-6)}$$

where:

C_{decel} = frequency of severe crashes on deceleration lanes, crashes/yr;
PDO_d = percent property-damage-only crashes in deceleration lanes (= 52.6); and
W_s = right shoulder width, ft.

Equations 5-5 and 5-6 were derived to predict total crashes, as opposed to severe crashes. An adjustment multiplier of "$1 - 0.01\ PDO$" was added to each equation to convert the prediction into severe crash frequency. This term requires an estimate of the percentage of PDO crashes. This percentage is estimated at 52.2 and 52.6 percent for acceleration lanes and deceleration lanes, respectively, based on the database description provided by Bauer and Harwood (*2*).

The factor for acceleration length in Equation 5-5 and for shoulder width in Equation 5-6 are counter-intuitive in terms of their influence on the corresponding model prediction. The positive sign associated with the acceleration length coefficient implies that longer acceleration lane lengths are associated with more crashes. Logically, a longer acceleration length would be associated with a lower speed differential between mainlane and merging ramp vehicles, which would suggest a safer operation. Similarly, the positive sign of the shoulder width coefficient implies an increase in crashes with wider shoulders. Accident modification factors developed for highways and streets have consistently shown that crashes are reduced with an increase in shoulder width. As a result, accident modification factors for acceleration lane length and ramp shoulder width cannot be derived from these model terms.

Figure 5-6 illustrates the safety prediction models for ramp speed-change lanes. It is based on an average acceleration lane length of 0.23 mi (1200 ft) and a right shoulder width of 8 ft. These dimensions represent average values found in the database used by Bauer and Harwood (*2*).

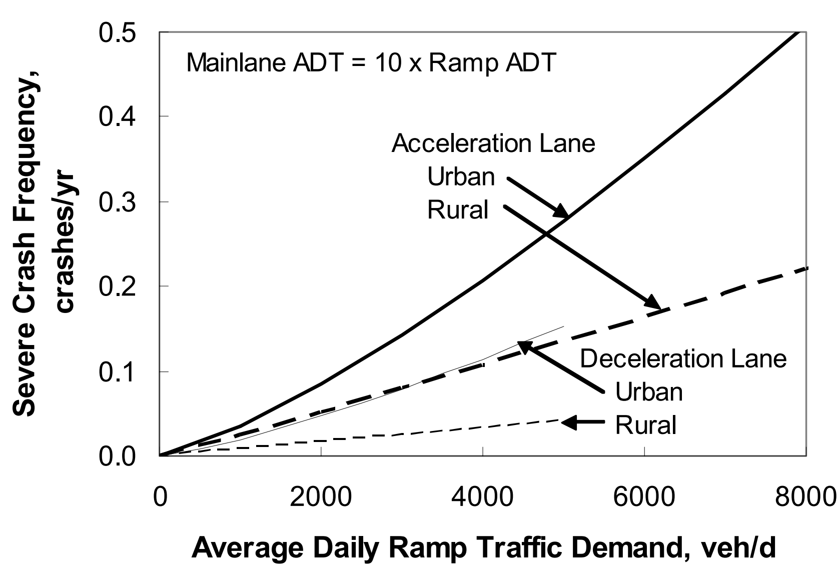


Figure 5-6. Crash Frequencies for Ramp Speed-Change Lanes.

The trends in Figure 5-6 indicate that acceleration lanes have about twice the severe crash frequency as deceleration lanes. They also indicate that speed-change lanes in urban areas have about twice the number of crashes as found at speed-change lanes in rural areas.

ACCIDENT MODIFICATION FACTORS

At this time, there are no AMFs available from the literature that relate crash frequency to interchange ramp geometry, access control, or related factors. It could be rationalized that the AMFs described in other chapters (e.g., those for lane width, shoulder width, grade, etc.) would also be applicable to ramps. However, interchange ramps have unique operational characteristics (e.g., one-way operation, significant speed change along the ramp) that would likely preclude the use of AMFs developed for highway segments.

REFERENCES

1. *Roadway Safety Design Workbook.* Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2005.

2. Bauer K.M., and D.W. Harwood. *Statistical Models of Accidents on Interchange Ramps and Speed-Change Lanes*. Report No. FHWA-RD-97-106. Federal Highway Administration, Washington, D.C., 1997.

3. Khorashadi, A. *Effect of Ramp Type and Geometry on Accidents.* FHWA/CA/TE-98/13. California Department of Transportation, Sacramento, California, November 1998.

Chapter 6
Rural Intersections

Page

INTRODUCTION

At intersections, drivers face a multitude of choices related to path, speed, and route that, in combination with numerous conflicting movements, complicate the driving task and significantly increase the potential for a crash. In Texas, about one-third of all crashes on rural highways occur at intersections. These crashes are consistently more severe than those experienced at urban intersections--primarily because of the high-speed nature of the rural highway. Safety improvements at rural intersections are often focused on design elements that provide separation for turning movements, increased driver visibility and sight lines, and traffic control devices that heighten driver awareness of the intersection's presence.

The development of a safe, efficient, and economical rural intersection design may reflect consideration of a wide variety of design alternatives. A variety of techniques exist for estimating the operational benefits of alternatives; many are automated through software tools. Techniques for estimating construction and right-of-way costs are also available to the designer. Unfortunately, techniques for estimating the safety benefits of alternative designs are not as readily available. This chapter summarizes information in the literature that can be used to estimate the crash frequency associated with various rural intersection design alternatives.

Objective

The objective of this chapter is to synthesize information in the literature that quantitatively describes the relationship between various rural intersection design components and safety. This information is intended to provide a basis for the development of a procedure for estimating the safety benefit of alternative designs. This procedure is documented in Chapter 6 of the *Roadway Safety Design Workbook* (*1*).

The presentation consists of an examination of safety prediction models and accident modification factors (AMFs). Safety prediction models provide an estimate of the expected crash frequency for a typical rural intersection. They include variables for the volume of the conflicting traffic streams. They also include variables for other factors considered to be correlated with crash frequency (e.g., median type, functional class, etc.). One or more AMFs can be multiplied by the expected crash frequency obtained from the prediction model to produce an estimate of the expected crash frequency for a specific intersection.

Scope

This chapter addresses the safety of intersections in a rural area. As such, the crash types used to describe the level of safety are identified as "intersection related." The intersection relationship of a crash is indicated on the crash report or, in some instances, is determined by the researchers. If the researchers determine the intersection-relationship of a crash, it is oftentimes based on crash location relative to the intersection (*2*). Specifically, all crashes that occur within a specified distance back from the intersection are labeled as "intersection related." The combination of crash location and type (e.g., turn related or multi-vehicle) has also been used by some researchers to identify intersection relationship. Crashes that are not intersection related are referred to herein as "mid-block" crashes and are the subject of Chapter 3.

When available, safety relationships that estimate the frequency of *severe* (i.e., injury or fatal) crashes are given preference for inclusion in this document. This preference is due to a wide variation in reporting threshold among counties and states. This variation complicates the extrapolation of crash trends found in one location to another location. Moreover, it can confound the development of safety prediction models using data from multiple agencies. Reporting threshold is strongly correlated with the number of property-damage-only (PDO) crashes found in a crash database. Agencies with a high reporting threshold include relatively few PDO crashes in their database and vice versa. As a consequence, the total crash frequency for a given road will be high if it is located in a state with a low reporting threshold. This problem is minimized when crash data analyses, comparisons, and models are based on data pertaining only to severe crashes.

Overview

This chapter documents a review of the literature related to rural intersection safety. The focus is on quantitative information that relates severe crash frequency to various geometric design components of the rural intersection. The review is not intended to be comprehensive in the context of referencing all relevant works that discuss rural intersection safety. Rather, the information presented herein is judged to be the most current information that is relevant to rural highway design in Texas. It is also judged to be the most reliable based on a review of the statistical analysis techniques used and the explanation of trends.

Where appropriate, the safety relationships reported in the literature are compared herein, with some interpretation offered as an explanation for any differences noted. The relationships are typically presented as reported in the literature; however, the names or the units of some variables have been changed to facilitate their uniform presentation in this document.

This chapter is envisioned to be useful to design engineers who desire a more complete understanding of the relationship between various intersection design components and severe crash frequency. As noted previously, it is also intended to serve as the basis for the development of the safety evaluation procedure described in Chapter 6 of the *Roadway Safety Design Workbook* (*1*).

This chapter consists of two main parts. In the first part to follow, several safety prediction models reported in the literature are described. In the second part, accident modification factors are described for various geometric design and traffic control components. In each of these parts, there is a separate discussion of signalized and unsignalized intersections.

SAFETY PREDICTION MODELS

Described in this part of the chapter are several safety prediction models that were developed to estimate the expected frequency of intersection-related crashes. The discussion is separated into two sections with one section describing models that apply to signalized intersections and a second section describing models that apply to unsignalized intersections. At the end of each section, the models are compared in terms of the relationship between severe crash frequency and traffic volume.

All of the models presented in this part of the chapter make reference to the "major" and "minor" roadways that form the intersection. These models are based on the assumption that the "major road" is the road with the higher volume of the two roads. It most instances, this assumption is in agreement with the number of lanes provided and the functional class of the two roads. However, in some instances, this assumption may mean that a road with more lanes or a higher functional classification but with lower volume will need to be specified as the "minor" road for the purpose of using a safety prediction model.

Rural Signalized Intersections

This section addresses safety prediction models for rural signalized intersections. Two sets of models are described and are identified by the names of their developers. They include:

- Vogt Model
- Harwood Model

Vogt Model

Vogt (*2*) investigated the relationships between crash frequency and various rural intersection geometry and operational attributes. He developed a series of safety prediction models for a variety of intersection configurations and control conditions. The model he developed for four-leg rural signalized intersections is described in this subsection. The models he developed for three-leg and four-leg rural unsignalized intersections are discussed in a later section.

Vogt's model for predicting severe crash frequency at four-leg rural signalized intersections is:

$$C = 0.03854\ Q_{major}^{0.2358}\ Q_{minor}^{0.2358}\ e^{B_1} \qquad \textbf{(6-1)}$$

with,

$$B_1 = -0.2943\, I_{mp} - 0.0113\, P_{lt} + 0.0822\, G_r + 0.0323\, P_t \qquad \textbf{(6-2)}$$

and

$$G_r = 100 \sum \frac{|g_2 - g_1|}{L_{vc}} \qquad \textbf{(6-3)}$$

where:

C = frequency of severe intersection-related crashes, crashes/yr;
Q_{major} = total daily volume on major-road (both directions), veh/d;
Q_{minor} = total daily volume on minor-road (both directions), veh/d;
I_{mp} = signal phasing indicator variable (1 if more than 2 phases; 0 otherwise);
P_{lt} = minor-road left-turn percentage during the peak hour, %;
G_r = average grade rate for all vertical curves within 800 ft of the intersection, %/ft;
g_i = grade of vertical curve tangent i (i = 1 for entry tangent and 2 for exit tangent), %;
L_{vc} = length of vertical curve, ft; and
P_t = percent trucks during the peak hour (average for all intersection movements), %.

It should be noted that the model predicts fewer crashes when the minor road has more left turns. This trend is illogical and is likely correlated with other factors that are present at intersections with numerous left turns. No justification or rationale was offered by the researchers for this counter-intuitive trend.

The characteristics of the data used by Vogt (*2*) are summarized in Table 6-1. As this table indicates, their models are based on three years of crash data for each of 49 rural intersections. As suggested by a comparison of the two databases described, the database used by Vogt is the same as that used by Harwood et al. (*3*) (and described in the next subsection). However, the difference between the two models lies in the method used to define an intersection-related crash. Vogt defined intersection-related crashes to be all crashes that occurred at, or within 250 ft of, the intersection. Harwood et al. used a more stringent criterion in that the crashes had to occur within 250 ft *and* be of a type that is generally related to intersection operations.

Table 6-1. Database Characteristics for Signalized Intersection Safety Prediction Models.

Model Developers	Database Characteristics				
	Data Source	Years of Crash data	Intersection Relationship [1]	Intersection Legs	Number of Intersections
Vogt (*2*)	California DOT & Michigan DOT	3	250 ft	4	49
Harwood et al. (*3*)	California DOT & Michigan DOT	3	250 ft and turn-related, sideswipe, rear end, or angle crashes.	4	49

Note:
1 - Intersection relationship indicates the definition used to identify crash relationship to the intersection. Distances listed denote the distance back from the intersection within which a crash is denoted as "intersection related."

Harwood Model

Harwood et al. (*3*) investigated the relationships between crash frequency and various rural intersection geometry and operational attributes. They developed safety prediction models for four-leg rural signalized intersections using the same database as used by Vogt (*2*). However, they used a different definition for identifying "intersection-related" crashes; they also considered different factors in their model. Their model was developed using total crash frequency (i.e., property-damage-only, injury, and fatal crashes); however, they provided an adjustment factor that can be used to use their model to estimate severe crash frequency.

The model developed by Harwood et al. for predicting severe crash frequency at four-leg rural signalized intersections is:

$$C = 0.00425\ Q_{major}^{0.60}\ Q_{minor}^{0.20}\ e^{B_1}\ (1.0 - 0.01\ PDO) \quad \textbf{(6-4)}$$

with,

$$B_1 = -0.40\ I_{mp} - 0.018\ P_{lt} + 0.11\ G_r + 0.026\ P_t + 0.041\ d_n \quad \textbf{(6-5)}$$

where:

C = frequency of severe intersection-related crashes, crashes/yr; and
d_n = number of driveways on the major road within 250 ft of the intersection; and
PDO = property-damage-only crashes as a percentage of total crashes (= 62.3 percent), %.

It should be noted that the model predicts fewer crashes when the minor road has more left turns. This trend is illogical and is likely correlated with other factors that are present at intersections with numerous left turns. No justification or rationale was offered by the researchers for this counter-intuitive trend. The characteristics of the data used by Harwood et al. (*3*) are summarized in Table 6-1.

Comparison of Signalized Intersection Crash Models

The two models described in the previous subsections are compared in this subsection. The objective of this comparison is to determine which model or models are reasonable in their prediction of severe crash frequency. To facilitate this comparison, the models are examined over a range of traffic volume levels. For both models, the values of the other model variables were set at typical values for rural signalized intersections. These values are listed in Table 6-2.

Table 6-2. Typical Values Used for Rural Signalized Intersection Model Comparison.

Model Variable	Typical Value	Safety Prediction Model Developer	
		Vogt (*2*)	Harwood et al. (*3*)
Signal phasing	more than 2 phases	✔	✔
Grade rate, %/ft	0.0 (i.e., flat)	✔	✔
Percentage left turns on minor road	27	✔	✔
Number of driveways on major road	3	--	✔
Percentage of trucks, %	9	✔	✔

Figure 6-1 illustrates the crash frequency predictions obtained from the two models. The trends shown indicate a general agreement that the annual severe crash frequency tends to equal between 3.0 and 3.5 crashes/yr when the major-road volume is 25,000 veh/d.

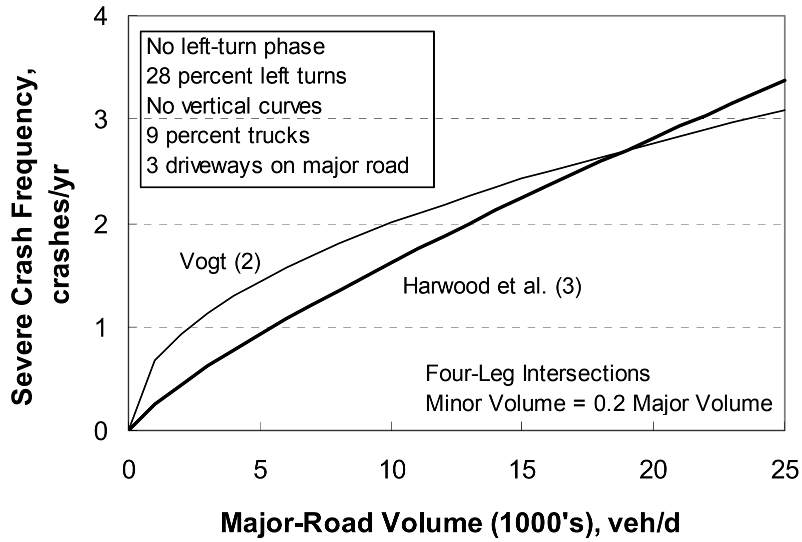


Figure 6-1. Comparison of Safety Prediction Models for Rural Signalized Intersections.

Rural Unsignalized Intersections

This section addresses safety prediction models for rural unsignalized intersections. Specifically, these are two-way stop-controlled (TWSC) intersections. Two sets of models are described and are identified by the names of their developers. They include:

- Bauer and Harwood Models
- Vogt Models

Bauer and Harwood Models

Bauer and Harwood (*4*) investigated the relationships between crash frequency and various intersection geometry and operational attributes. They developed a series of safety prediction models for a variety of intersection configurations and control conditions. The models they developed for three-leg and four-leg rural TWSC intersections are discussed in this subsection. The urban intersection models are described in Chapter 7.

Bauer and Harwood's model for predicting severe crash frequency at rural *four-leg* TWSC intersections is:

$$C = 0.0000113 \; Q_{major}^{0.680} \; Q_{minor}^{0.546} \; e^{B_1 + B_2} \qquad \textbf{(6-6)}$$

with,

$$B_1 = 0.385\, I_{maj3} + 0.013\, V_d + 0.183\, I_{flat} \tag{6-7}$$

and

$$B_2 = -0.234\, I_{mnt} + 0.261\, I_{ma} + 0.170\, I_{cl} + 0.219\, I_{nolite} \tag{6-8}$$

where:

C = frequency of severe intersection-related crashes, crashes/yr;
I_{maj3} = major-road through lanes (1 if 3 or fewer lanes; 0 otherwise);
V_d = major-road design speed, mph;
I_{flat} = terrain indicator variable (1 if flat; 0 if rolling);
I_{mnt} = terrain indicator variable (1 if mountainous; 0 if rolling);
I_{ma} = major-road functional class indicator variable (1 if minor arterial; 0 if principal arterial);
I_{cl} = major-road functional class indicator variable (1 if collector; 0 if principal arterial); and
I_{nolite} = intersection lighting indicator variable (1 if no lighting; 0 otherwise).

It should be noted that the model predicts more crashes when the terrain is flat than when it is rolling and more when it is rolling than mountainous. The researchers note this anomaly and suggest that this trend is likely due to a correlation between this variable and other factors.

Bauer and Harwood's model for predicting severe crash frequency at rural *three-leg* TWSC intersections is:

$$C = 0.0000357\ Q_{major}^{0.781}\ Q_{minor}^{0.384}\ e^{B_1 + B_2} \tag{6-9}$$

with,

$$B_1 = -0.030\, W_s + 0.169\, I_{nolite} + 0.180\, I_{noltln} + 0.062\, I_{cbltln} \tag{6-10}$$

and

$$B_2 = -0.219\, I_{frt} + 0.164\, I_{ma} + 0.192\, I_{cl} \tag{6-11}$$

where:

C = frequency of severe intersection-related crashes, crashes/yr;
I_{noltln} = major-road left-turn channelization indicator variable (1 if no left-turn lane; 0 if painted left-turn lane);
I_{cbltln} = major-road left-turn channelization indicator variable (1 if curbed left-turn lane; 0 if painted left-turn lane);
W_s = major-road outside shoulder width, ft; and
I_{frt} = minor-road channelization (1 if no free right-turn lane; 0 if free right-turn lane).

The regression coefficient in Equation 6-11 for right-turn channelization on the minor road indicates that the addition of a free right-turn lane will increase crashes. This trend is likely attributable to the yield or stop control associated with such lanes and the fact that the driver stopped in the free right-turn lane is typically in a poor position to evaluate safe gaps in the crossing traffic stream. This poor position stems from the large-radius travel path associated with the free right-turn lane. The stop (or yield) line on this path is typically placed near the point of entry to the major road and requires drivers to look back, over their left shoulder, to search for gaps in the conflicting traffic stream. This maneuver is difficult for most drivers and may compromise the diligence with which they search for safe gaps.

Although not apparent by inspection of the coefficients listed in Equations 6-6 through 6-11, an examination of model predictions indicates that three-leg intersections have between 50 and 60 percent of the crashes experienced by four-leg intersections, for similar volume levels.

The characteristics of the data used by Bauer and Harwood (*4*) are summarized in Table 6-3. They used three years of crash data for each of 4126 rural intersections. They defined intersection-related crashes to be all crashes that occurred at, or within 250 ft of, the intersection.

Table 6-3. Database Characteristics for Unsignalized Intersection Safety Prediction Models.

Model Developers	Database Characteristics				
	Data Source	Years of Crash data	Intersection Relationship [1]	Intersection Legs	Number of Intersections
Bauer & Harwood (*4*)	California DOT	3	250 ft	3	2692
				4	1434
Vogt (*2*)	California DOT & Michigan DOT	3	250 ft	3	84
				4	72

Note:

1 - Intersection relationship indicates the definition used to identify crash relationship to the intersection. Distances listed denote the distance back from the intersection within which a crash is denoted as “intersection related.”

Vogt Models

Vogt (*2*) developed a series of safety prediction models for rural TWSC intersections. One model was calibrated for four-leg intersections and a second was calibrated for three-leg intersections. The model for predicting severe crash frequency at rural *four-leg* intersections is:

$$C = 0.0000530\ Q_{major}^{0.7224}\ Q_{minor}^{0.4778} \tag{6-12}$$

His model for predicting severe crash frequency at rural *three-leg* intersections is:

$$C = 0.0000019\ Q_{major}^{1.2028}\ Q_{minor}^{0.1925} \tag{6-13}$$

Although not apparent by inspection of the coefficients listed in Equations 6-12 and 6-13, an examination of model predictions indicates that three-leg intersections have between 35 and 45 percent of the crashes experienced by four-leg intersections, for similar volume levels.

The characteristics of the data used by Vogt (*2*) are summarized in Table 6-3. He used three years of crash data for each of 156 rural intersections. He defined all crashes occurring at, or within 250 ft of, the intersection as "intersection related."

Comparison of Unsignalized Intersection Crash Models

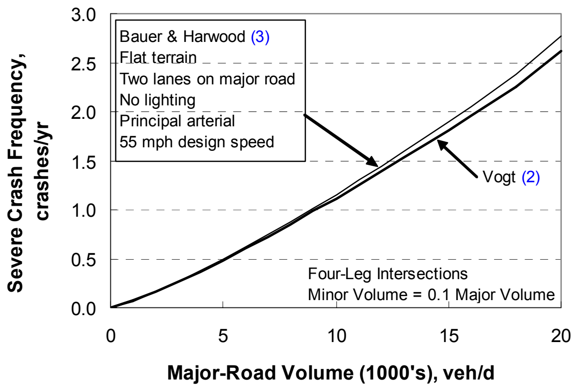
The models described in the previous subsections are compared in this subsection. The objective of this comparison is to determine which model or models are reasonable in their prediction of severe crash frequency. To facilitate this comparison, the models are examined over a range of traffic volume levels. The models were grouped into three-leg and four-leg categories. For the Bauer and Harwood (*4*) model, the values of the model variables were set at typical values for rural unsignalized intersections. These values are listed in Table 6-4.

Table 6-4. Typical Values Used for Rural Unsignalized Intersection Model Comparison.

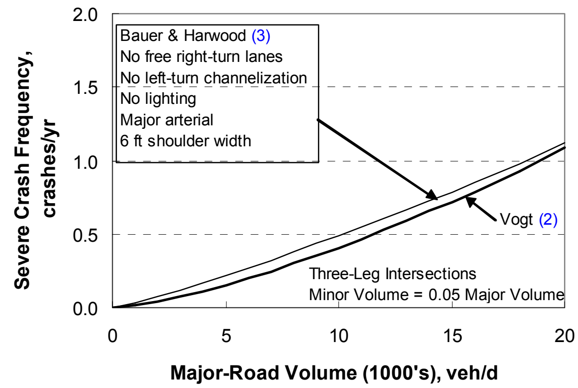
Model Variable	Typical Value	Safety Prediction Model Developer	
		Vogt (*2*)	Bauer & Harwood (*4*)
Shoulder width, ft	6.0	--	✔
Terrain	flat	--	✔
Number of lanes on major road	2	--	✔
Free right-turn lanes on minor road	no	--	✔
Left-turn channelization on major road	no	--	✔
Free right-turn lanes on minor road	no	--	✔
Functional class of major road	principal arterial	--	✔
Design speed of major road, mph	55	--	✔
Intersection lighting	no	--	✔

Typical values listed in Table 6-4 are based on the median values for each variable in the database used by Bauer and Harwood (*4*, Tables 2 and 6).

Figure 6-2 illustrates the crash frequency predictions obtained from the various models. The trends shown indicate a general agreement that the annual severe crash frequency for four-leg intersections tends to equal about 2.7 crashes/yr when the major-road volume is 20,000 veh/d. The three-leg intersection trends are also in general agreement. The trends shown suggest that the annual severe crash frequency for three-leg intersections tends to equal about 1.1 crashes/yr when the major-road volume is 20,000 veh/d.



a. Four-Leg Intersections.



b. Three-Leg Intersections.

Figure 6-2. Comparison of Safety Prediction Models for Rural Unsignalized Intersections.

Development of Representative Intersection Crash Rates

The safety prediction models previously presented are examined more closely in this section for the purpose of developing representative intersection crash rates. A generalized equation was developed for estimating the average crash rate. The equation is:

$$CR = \left(\frac{10^6}{365}\right) \frac{\alpha\, Q_{major}^{\beta_1}\, Q_{minor}^{\beta_2}}{Q_{major} + Q_{minor}} \qquad \textbf{(6-14)}$$

with,

$$\alpha = \alpha_b + e^{(other\ terms)} \qquad \textbf{(6-15)}$$

where:

CR = severe crash rate for intersection-related crashes, crashes/million-vehicle-miles (mvm);
α = regression constant that combines α_b with all exponential terms at their average value; and
α_b = regression constant obtained from multivariate regression model.

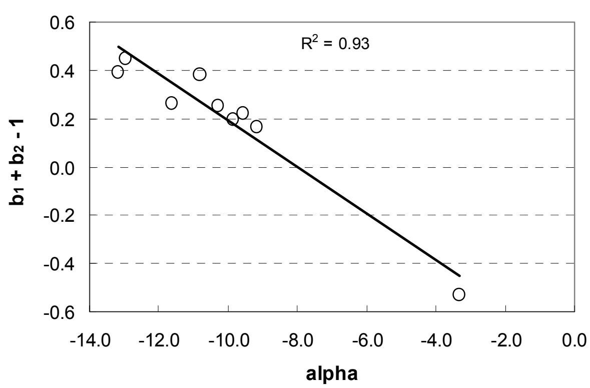
The numerator of this equation includes the basic structure of the safety prediction models described previously. The constant of "$10^6/365$" in Equation 6-14 is included to convert the rate into traditional crash rate units–crashes per million entering vehicles. Equation 6-14 was simplified to the following form by substituting the ratio of the minor-road to major-road entering flows r:

$$CR = 2740\, \frac{\alpha\, Q_{major}^{\beta_1 + \beta_2 - 1}\, r^{\beta_2}}{1 + r} \qquad \textbf{(6-16)}$$

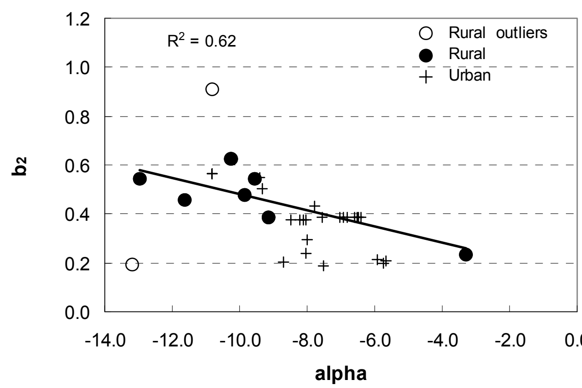
where:

r = ratio of minor-road to major-road daily volumes (= Q_{minor} / Q_{major}).

As a next step in the examination, the relationship between α, β_1, β_2, and various variables describing intersection control and number of approach legs was investigated. The values of α, β_1, β_2 were obtained from the models previously described. To supplement this database, some additional values of α, β_1, β_2 were obtained from some models developed by Vogt and Bared (5, Tables 36 and 37) for predicting severe crash frequency. For this analysis, a variable *alpha* was computed as the natural log of α (i.e., *alpha* = ln[α]). Figure 6-3a illustrates the relationship found between alpha and the sum "$\beta_1 + \beta_2 - 1$" (where β_1 and β_2 are labeled b_1 and b_2, respectively).



a. Correlation between Alpha and "$\beta_1 + \beta_2 - 1$."



b. Correlation between Alpha and β_2.

Figure 6-3. Correlation between Model Coefficients.

Figure 6-3b illustrates the relationship found between α and β_2. The circular data points in this figure represent the database of α and β_2 values applicable to rural intersections. Two of the data points (shown as open circles) did not follow the pattern in the remaining database and were excluded from further analysis. The remaining database included only one pair of values for signalized intersections; all others were for unsignalized intersections. As a result, the rural intersection database was not adequate for evaluating the effect of intersection control mode on β_2. To overcome this limitation, the database was expanded to include α and β_2 values for urban signalized and unsignalized intersections (see Chapter 7). These values are shown in Figure 6-3b using a "+" symbol. In general, the trends in Figures 6-3a and 6-3b illustrate that the variables *alpha*, β_1, and β_2 are correlated with each other. Although not shown, they were also found to be correlated with control mode. A subsequent analysis indicated a similar correlation between *alpha* and the number of intersection legs ($R^2 = 0.31$).

The following relationships were derived based on the analysis of model coefficients:

$$\alpha = e^{-16.52 + 1.60 N_{leg}} \tag{6-17}$$

with,

$$\beta_1 + \beta_2 - 1 = -0.768 - 0.096 \ln(\alpha) \tag{6-18}$$

and

$$\beta_2 = -0.638 + 0.122\, N_{leg} + 0.080\, I_{sig} - 0.066 \ln(\alpha) \qquad \textbf{(6-19)}$$

where:

N_{leg} = number of intersection legs (3 or 4); and
I_{sig} = indicator variable for intersection control mode (1 for signalized; 0 otherwise).

Equations 6-17, 6-18, and 6-19 were used with Equation 6-16 to compute equivalent crash rates for various combinations of intersection control and number of approach legs. The findings from this analysis are presented in Tables 6-5 and 6-6 for three-leg and four-leg rural intersections, respectively.

Table 6-5. Severe Crash Rates for Three-Leg Rural Intersections.

Control Mode	Major-Road Volume, veh/d	Crash Rate, severe crashes per million-vehicle-miles				
		Ratio of Minor-Road to Major-Road Volume				
		0.05	**0.10**	**0.15**	**0.20**	**0.25**
Unsignalized	5000	0.10	0.14	0.16	0.18	0.19
	10,000	0.13	0.18	0.21	0.23	0.25
	15,000	0.15	0.20	0.24	0.26	0.28
	20,000	0.17	0.23	Intersection very likely to meet signal warrants		
	25,000	0.18				
Signalized	5000	0.08	0.11	0.14	0.16	0.17
	10,000	0.10	0.15	0.18	0.20	0.22
	15,000	0.12	0.17	0.21	0.23	0.25
	20,000	0.13	0.19	0.23	0.26	0.28
	25,000	0.14	0.20	0.25	0.28	0.30
	30,000	0.15	0.22	0.26	0.30	0.33
	40,000	0.17	0.24	0.29	0.33	0.36
	≥50,000	0.18	0.26	0.32	0.36	0.39

The following equation should be used to estimate severe crash frequency in conjunction with the crash rates listed in Tables 6-5 or 6-6:

$$C = CR \left(\frac{365}{10^6} \right) (Q_{major} + Q_{minor}) \qquad \textbf{(6-20)}$$

Table 6-6. Severe Crash Rates for Four-Leg Rural Intersections.

Control Mode	Major-Road Volume, veh/d	Crash Rate, severe crashes per million-vehicle-miles				
		Ratio of Minor-Road to Major-Road Volume				
		0.10	**0.30**	**0.50**	**0.70**	**0.90**
Unsignalized	5000	0.18	0.26	0.30	0.31	0.32
	10,000	0.20	0.30	0.34	0.36	0.36
	15,000	0.22	0.33	0.37	0.39	0.40
	20,000	0.23	Intersection very likely to meet signal warrants			
	25,000	0.25				
Signalized	5000	0.15	0.24	0.28	0.30	0.31
	10,000	0.17	0.28	0.32	0.35	0.36
	15,000	0.18	0.30	0.35	0.38	0.39
	20,000	0.20	0.32	0.37	0.40	0.42
	25,000	0.20	0.33	0.39	0.42	0.44
	30,000	0.21	0.35	0.41	0.44	0.45
	40,000	0.23	0.37	0.43	0.46	0.48
	≥50,000	0.24	0.38	0.45	0.49	0.50

ACCIDENT MODIFICATION FACTORS

This part of the chapter describes various accident modification factors that are related to the design of a rural intersection. The discussion is separated into AMFs that apply to signalized intersections and those that apply to unsignalized intersections.

Rural Signalized Intersections

This section identifies the AMFs that are applicable to signalized intersections in a rural environment. These factors were either derived from the models described in the previous section or extracted from other safety prediction models described in the literature. The focus of the discussion is on AMFs related to geometric design; however, AMFs related to access control and other intersection features are also described.

Geometric Design

This subsection describes AMFs related to the geometric design of a rural signalized intersection. Topics specifically addressed are listed in Table 6-7. Many geometric design components or elements are not listed in this table (e.g., approach grade) that are also likely to have some correlation with severe crash frequency. However, a review of the literature did not reveal useful quantitative information describing these effects. The list of available AMFs for intersection geometric design is likely to increase as new research in this area is undertaken.

Table 6-7. AMFs Related to Geometric Design of Rural Signalized Intersections.

Section	Accident Modification Factor		
Cross section	Left-turn lane Alignment skew angle	Right-turn lane Intersection sight distance	Number of lanes

In some instances, an AMF is derived from a safety prediction model as the ratio of "intersection crash frequency with a changed condition" to "intersection crash frequency without the change." In other instances, the AMF is obtained directly from a before-after study. Occasionally, crash rates reported in the literature were used to derive an AMF.

Left-Turn Lane. Harwood et al. *(6)* investigated the relationship between the presence of left- and right-turn lanes and crash frequency. They examined the change in severe crash frequency at intersections that had a left-turn lane installed using a before-after study design. The results of their investigation are summarized in Table 6-8. The values shown in this table can be used to estimate the change in crashes at a signalized intersection at which a left-turn lane is added to one or both of the major-road approaches.

Table 6-8. Effect of Adding a Left-Turn Lane at a Rural Signalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches with Left-Turn Lanes Installed			
	One Approach		Both Approaches [3]	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	0.85	0.86 [1]	not applicable	
4	0.82	0.83 [2]	0.67	0.69

Notes:
1 - Value estimated as: 0.83 = 0.91/ 0.90 × 0.82 using urban intersection data reported by Harwood et al. *(6)*.
2 - Value estimated as: 0.86 = 0.91/ 0.90 × 0.85 using urban intersection data reported by Harwood et al. *(6)*.
3 - AMFs for "Both Approaches" estimated as the square of the "One Approach" AMFs.

The crash rates presented in a previous part of this chapter reflect typical rural signalized intersection design. Data provided by Bauer and Harwood *(4)* indicate that the typical (i.e., base) condition for signalized intersections is "left-turn lane provided." The values presented in Table 6-8 have been converted to equivalent AMFs to reflect this base condition. The resulting left-turn lane AMFs are listed in Table 6-9.

Table 6-9. AMFs for Excluding a Left-Turn Lane at a Rural Signalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches without Left-Turn Lanes Installed			
	One Approach		Both Approaches	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	1.18	1.16	not applicable	
4	1.22	1.21	1.49	1.45

Right-Turn Lane. Harwood et al. (*6*) also investigated the relationship between right-turn lanes and signalized intersection crash frequency. Their recommended AMFs for the addition of a right-turn lane on the major-road approach to a signalized intersection are shown in Table 6-10. Harwood et al. recommend using these AMFs for any signalized intersection, regardless whether it has three or four legs.

Table 6-10. AMFs for Adding a Right-Turn Lane at a Rural Signalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches with Right-Turn Lanes Installed			
	One Approach		Both Approaches [2]	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	0.96 [1]	0.91 [1]	not applicable	
4	0.96	0.91	0.92	0.83

Notes:
1 - Harwood et al. (*6*) did not quantify AMFs for signalized intersections with three legs. They recommend the application of the "four-leg" AMFs to intersections with three legs.
2 - AMFs for "Both Approaches" estimated as the square of the "One Approach" AMFs.

Number of Lanes on the Major and Minor Roads. A series of safety prediction models was developed by Bauer and Harwood (*4*) for rural and urban, signalized and unsignalized intersections. A variable in these models relates the number of through lanes on the major and minor roads to the reported severe crash frequency. The regression coefficients in these models are shown in column 5 of Table 6-11. The AMF for each coefficient b can be computed as $AMF = e^b$. Thus, the coefficient of -0.163 in the first row corresponds to an AMF of 0.85. It indicates that urban signalized intersection approaches where the major-street has "3 or fewer" lanes will have 85 percent of the crashes experienced by approaches with "6 or more" lanes. Similar comparisons of the effect of number-of-lanes can be made within the other combinations of area type, control mode, and road listed.

The coefficients in column 5 were normalized for a common base number-of-lanes to facilitate comparison across the various combinations of area type, control mode, and road. The base number of lanes used for this analysis is two through lanes. The normalized coefficients are listed in column 6. These values have similar interpretation as to those in column 5, the only difference being that the coefficients in column 6 have a "two-lane approach" as the base condition. Examination of the normalized coefficients indicates that the effect of number-of-lanes varies by control mode. Specifically, an increase in lanes at a signalized intersection is associated with an increase in severe crash frequency, all other factors unchanged. In contrast, an increase in lanes at an unsignalized intersection is associated with a *decrease* in severe crash frequency.

Table 6-11. AMFs for Number of Through Lanes at Rural and Urban Intersections.

Area Type	Control Mode	Road	Number of Through Lanes	Regression Coefficient	Normalized Coefficient [1]	Estimated Coefficient [2]	Adjusted AMF_{lane} [3]
Urban	Signalized	Major	3 or fewer	-0.163	0.000	0.000	0.99
			4 or 5	-0.151	0.012	0.014	1.00
			6 or more	0.000	0.163	0.029	1.01
		Minor	3 or fewer	-0.155	0.000	0.000	1.00
			4 or more	0.000	0.155	0.014	1.01
	Unsignalized	Major	3 or fewer	0.282	0.000	0.000	1.20
			4 or 5	0.049	-0.233	-0.185	1.00
			6 or more	0.000	-0.282	-0.371	0.83
		Minor	3 or fewer	n.a.	n.a.	0.000	1.00
			4 or more	n.a.	n.a.	-0.185	0.83
Rural	Signalized	Major	3 or fewer	n.a.	n.a.	0.000	**1.00**
			4 or 5	n.a.	n.a.	0.014	**1.01**
			6 or more	n.a.	n.a.	0.029	**1.03**
		Minor	3 or fewer	n.a.	n.a.	0.000	**1.00**
			4 or more	n.a.	n.a.	0.014	**1.01**
	Unsignalized	Major	3 or fewer	0.385	0.000	0.000	1.00
			4 or 5	0.000	-0.385	-0.185	0.83
			6 or more	n.a.	n.a.	-0.371	0.69
		Minor	3 or fewer	n.a.	n.a.	0.000	1.00
			4 or more	n.a.	n.a.	-0.185	0.83

Notes:
n.a. - data not available.
1 - Normalized Coefficient: regression coefficient adjusted to yield a coefficient of 0.0 for two through lanes for all combinations of area type, control mode, and road. Computed as $b_i - b_2$; where, b_i is the coefficient for combination i and b_2 is the coefficient for the two-lane case.
2 - Estimated Coefficient: normalized coefficient estimated using the calibrated regression model.
3 - Adjusted AMF: estimated coefficient adjusted for number of lanes for the base condition (i.e., rural: 2 lanes major, 2 lanes minor; urban: 4 lanes major, 2 lanes minor) and converted to an AMF using $AMF = e^{(estimated\ coefficient)}$.

Regression analysis was used to identify the factors that influence the normalized coefficients. After separate evaluation of all factors, the regression model having the best fit included only an indicator variable to account for the effect of control mode. The form of this model is:

$$b = c_0 + (c_1 + c_2 I_{sg})(N_{ln} - 2) \quad \textbf{(6-21)}$$

where:
I_{sg} = indicator variable for control mode (1 if signalized; 0 if unsignalized); and
N_{ln} = number of through lanes on the road.

Weighted regression was used for the analysis. The weight w assigned to each variable is equal to the reciprocal of the coefficient squared (i.e., $w_i = 1/b_i^2$). The calibrated model form is:

$$b = (-0.093 + 0.100\, I_{sg})\, (N_{ln} - 2) \qquad \textbf{(6-22)}$$

Both constants in Equation 6-22 are statistically significant at a 99 percent confidence level. The weighted coefficient of determination R^2 of the regression model is 0.99.

As a last step, the estimated coefficients obtained from Equation 6-22 were adjusted to a base condition number-of-lanes reflective of a rural intersection. The base condition for rural signalized intersections is two lanes on both the major and minor roads. The AMF values appropriate for these intersections are listed in rows 12 through 16 of column 8 in Table 6-11 (highlighted in bold font).

Alignment Skew Angle. Harwood et al. (*3*) found that intersection skew angle was correlated with intersection crash frequency at unsignalized intersections. However, intersection skew apparently was *not* found to be related to crash frequency at signalized intersections. Harwood et al. recommended an AMF_{skew} of 1.0 (no effect) for intersection skew angle at signalized intersections.

Intersection Sight Distance. Harwood et al. (*3*) convened an expert panel to determine the relationship between crash frequency and intersection sight distance at signalized intersections. The panel concluded that sight distance restrictions at a signalized intersection would not significantly influence crash frequency. They recommended an AMF_{SD} equal to 1.0 for signalized intersections.

Access Control

Several safety prediction models have been developed for rural intersections that include a variable that relates driveway frequency to crash frequency. Driveway frequency was defined to be the count of driveways within 250 ft of the intersection on the major road. This count would include driveways on both sides of the road and on both major-road intersection legs. The regression coefficients in these models are shown in column 6 of Table 6-12. The AMF for each coefficient b can be computed as:

$$AMF_{nd} = e^{b\,(d_n - 3)} \qquad \textbf{(6-23)}$$

where:

AMF_{nd} = driveway frequency accident modification factor; and

d_n = number of driveways on the major road within 250 ft of the intersection.

This equation reflects a base condition of three driveways. The coefficients listed in Table 6-12 consistently indicate an increase in crash frequency with an increase in driveway frequency, regardless of the control mode, driveway type, or number of intersection legs.

Regression analysis was used to identify the factors that influence the coefficient values. After separate evaluation of all factors, the regression model having the most logical fit included an indicator variable to account for the effect of control mode. The form of this model is:

$$b = c_0 + c_1\, I_{sg} \qquad \textbf{(6-24)}$$

Table 6-12. Coefficient Analysis for Driveway Frequency at Rural Intersections.

Control Mode	Driveway Type [1]	Intersection Legs	Model Source	Crash Severity	Regression Coefficient	Estimated Coefficient [2]
Signalized	Any	4	Harwood et al. (*3*)	All	0.041	0.046
	Comm.	4	Washington et al. (*7*)	All	0.0539	0.046
Unsignalized	Any	4	Harwood et al. (*3*)	All	0.13	0.056
	Any	3	Vogt (*2*)	All	0.0391	0.056
	Any	4	Washington et al. (*7*)	All	0.1219	0.056
	Comm.	3	Washington et al. (*7*)	All	0.0681	0.056

Notes:
1 - Driveway Type: Any - all driveway types (e.g., commercial, residential, industrial, etc.); Comm.- only commercial.
2 - Estimated Coefficient: normalized coefficient estimated using the calibrated regression model.

Trends in the coefficients in column 6 of Table 6-12 suggest that there is a possible influence of "number of legs" on the value of the coefficient *b*. However, the effect represented in these coefficients suggests that the addition of a fourth intersection leg increases the AMF value by a factor of 2.0 or more. This amount of increase is rationalized to be excessive and is likely due to colinearity in the coefficients derived from the models for four-leg unsignalized intersections.

Weighted regression was used for the analysis. The weight *w* assigned to each variable is equal to the reciprocal of the coefficient squared (i.e., $w_i = 1/b_i^2$). The calibrated model form is:

$$b = 0.056 - 0.010\, I_{sg} \qquad \textbf{(6-25)}$$

The constant "0.011" in Equation 6-25 is statistically significant at an 80 percent confidence level. The weighted coefficient of determination R^2 of the regression model is 0.76. The coefficient for signalized intersections is obtained from Table 6-12 (or Equation 6-25) as 0.046. This value can be combined with Equation 6-23 to obtain the following AMF for driveway frequency:

$$AMF_{nd} = e^{0.046\,(d_n - 3)} \qquad \textbf{(6-26)}$$

Figure 6-4 illustrates the driveway frequency AMF for rural, four-leg signalized intersections. The two AMFs obtained from the coefficients reported by Harwood et al. (*3*) and by Washington et al. (*7*) are shown with thin trend lines. The AMF obtained from Equation 6-26 is shown with a thick bold line (and labeled "derived"). The trends shown in the figure are in general agreement and suggest that an intersection with no driveways within 250 ft will have an AMF of about 0.87 and implies that it will be associated with 13 percent fewer crashes than an intersection with three driveways (say, two driveways on one major-road approach and one on the other approach).

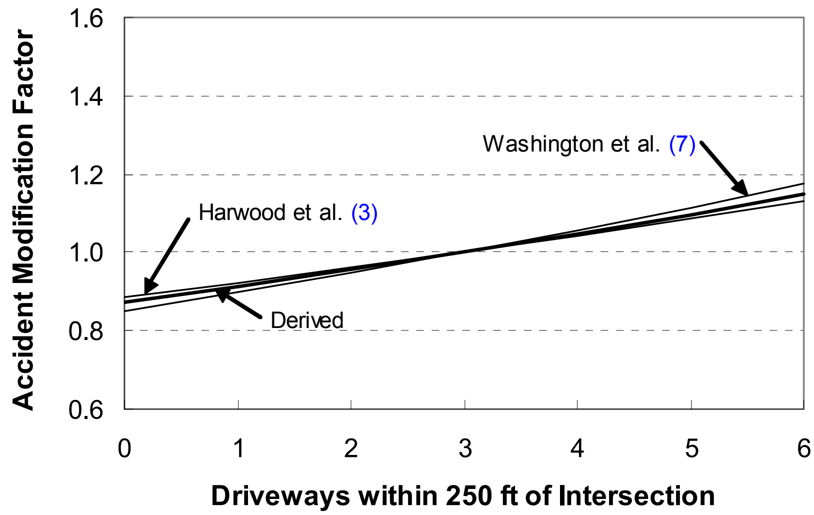


Figure 6-4. Driveway Frequency AMF for Rural Signalized Intersections.

Other Adjustment Factors

This section describes AMFs related to features of the signalized intersection that are not categorized as related to geometric design or access control. Specifically, the AMFs described in this section address truck presence and speed.

Truck Presence. Vogt (*2*) and Harwood et al. (*3*) both found a relationship between crash frequency and truck presence at signalized intersections on rural highways. Each researcher developed a safety prediction model that includes various factors (including truck percentage). The AMF that is derived from these models is shown below. It reflects a base condition of 9 percent trucks.

$$AMF_{tk} = e^{b(P_t - 9)} \tag{6-27}$$

where:

AMF_{tk} = truck presence accident modification factor, and
P_t = percent trucks during the peak hour (average for all intersection movements), %.

The value of variable *b* is shown in Table 6-13 for the two referenced sources. Also shown is the weighted average of the two coefficients. The weighted average can be used with Equation 6-27 to obtain the following AMF for truck presence:

$$AMF_{tk} = e^{0.028(P_t - 9)} \tag{6-28}$$

Table 6-13. Coefficient Analysis for Truck Presence at Rural Signalized Intersections.

Model Source	Intersection Type	Crash Severity	Coefficient *b*
Vogt (*2*)	Rural 4-leg signalized intersection	Severe	0.0323
Harwood et al. (*3*)	Rural 4-leg signalized intersection	All	0.026
		Weighted Average: [1]	**0.028**

Note:
1 - Weighted average computed as: $0.028 = (1/0.0323 + 1/0.026) / (1/0.0323^2 + 1/0.026^2)$.

Figure 6-5 illustrates the two expressions for the truck presence AMF that are based on the models developed by Vogt (*2*) and by Harwood et al. (*3*). The AMF obtained from Equation 6-28 is shown with a thick bold line (and labeled "derived"). The trends shown suggest that an intersection with no trucks has an AMF of about 0.77. Thus, this intersection will be associated with 23 percent fewer crashes than an intersection with 9 percent trucks. Trucks have a more difficult time stopping in response to the change in signal indication than passenger cars. This fact, coupled with the high-speed nature of most rural signalized intersections, is likely to result in more truck-involved rear-end and right-angle crashes as truck percentage increases.

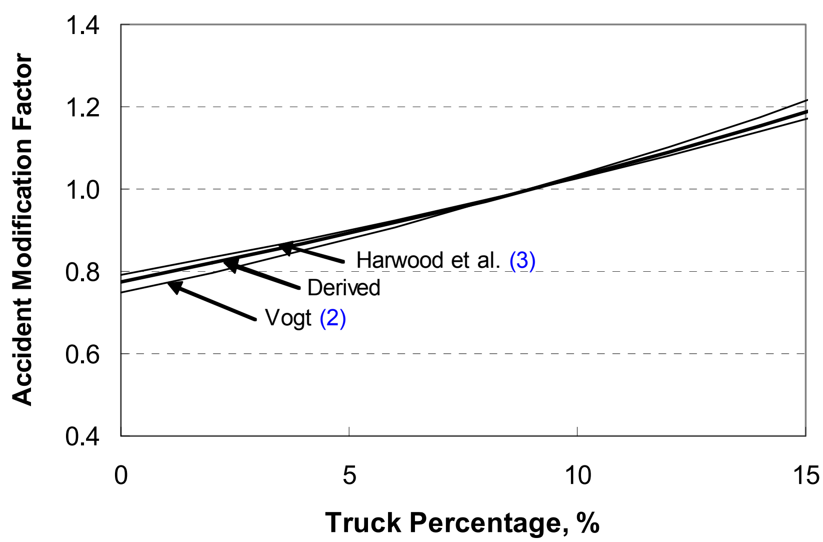


Figure 6-5. Truck Presence AMF for Rural Signalized Intersections.

Speed. Several safety prediction models have been developed that include a variable that relates speed limit or design speed to crash frequency. The regression coefficients in these models are shown in column 7 of Table 6-14. The AMF associated with each coefficient *b* can be derived using Equation 6-29. This equation reflects a base speed of 55 mph.

$$AMF_{sl} = e^{b(V-55)} \qquad \textbf{(6-29)}$$

where:

AMF_{sl} = speed accident modification factor; and
V = major-road speed limit (or design speed), mph.

Table 6-14. Coefficient Analysis for Speed at Rural and Urban Intersections.

Area Type	Control Mode	Road Type	Intersection Legs	Speed Type [1]	Model Source	Regression Coefficient [2]	Estimated Coefficient [3]
Urban	Signalized	Major	4	Design	Bauer & Harwood (4)	0.005	0.005
Rural	Signalized	Major	4	Limit	Washington et al. (7)	0.0397	0.019
	Unsignal-ized	Major	4	Design	Bauer & Harwood (4)	0.013	0.019
		Minor	4	Limit	Vogt (2)	0.0339	0.019
		Minor	4	Limit	Washington et al. (7)	0.0289	0.019

Notes:
1 - Speed type: type of speed used to calibrate the regression coefficient (i.e., design speed, or speed limit).
2 - All coefficients listed are derived from models relating speed to *severe* crash frequency.
3 - Estimated Coefficient: normalized coefficient estimated using the calibrated regression model.

An examination of the regression coefficients listed in Table 6-14 indicated a consistent trend toward an increase in crash frequency with an increase in either the speed limit or the design speed. Regression analysis was used to determine if area type, control mode, road type, number of intersection legs, or speed type was correlated with the coefficient values. Based on this analysis, the following model was found to offer the best fit to the data:

$$b = c_0 + c_1 I_{urb} \qquad \textbf{(6-30)}$$

where:
I_{urb} = indicator variable for area type (1 if urban; 0 if rural).

Weighted regression was used for the analysis. The weight w assigned to each variable is equal to the reciprocal of the coefficient squared (i.e., $w_i = 1/b_i^2$). The calibrated model form is:

$$b = 0.019 - 0.014\, I_{urb} \qquad \textbf{(6-31)}$$

The constants in Equation 6-31 are statistically significant at a 94 percent confidence level. The weighted coefficient of determination R^2 of the regression model is 0.80. The coefficient for rural signalized intersections is obtained from Table 6-14 (or Equation 6-31) as 0.019. It is equally applicable to speed limit or design speed. This value can be combined with Equation 6-29 to obtain the following AMF for speed:

$$AMF_{sl} = e^{0.019\,(V - 55)} \qquad \textbf{(6-32)}$$

Figure 6-6 illustrates the speed AMF for rural signalized intersections. The AMF obtained from the coefficient reported by Washington et al. (7) is shown with the thin trend line. The AMF obtained from Equation 6-32 is shown with a thick bold line (and labeled "derived"). The trends in the two AMFs indicate that lower speeds are associated with fewer severe crashes. These AMFs were derived from severe crash data. Hence, the trends found likely reflect the increase in crash severity with increasing speed.

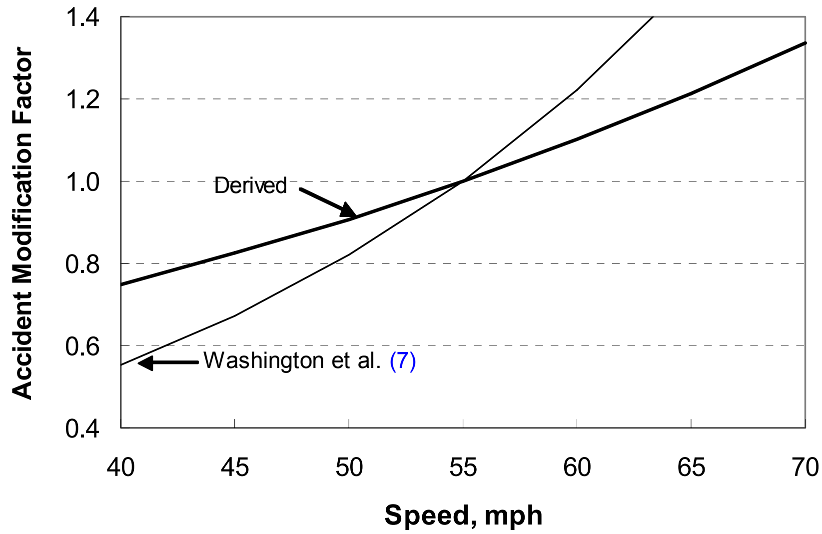


Figure 6-6. Speed AMF for Rural Signalized Intersections.

Rural Unsignalized Intersections

This section identifies the AMFs that are applicable to unsignalized intersections in a rural environment. Specifically, these are two-way stop-controlled intersections. The factors identified were either derived from the models described previously or extracted from other safety prediction models described in the literature. The focus of the discussion is on AMFs related to geometric design; however, AMFs related to access control and other intersection features are also described.

Geometric Design

This subsection describes AMFs related to the geometric design of a rural unsignalized intersection. Topics specifically addressed are listed in Table 6-15. Many geometric design components or elements are not listed in this table (e.g., approach grade) that are also likely to have some correlation with severe crash frequency. However, a review of the literature did not reveal useful quantitative information describing these effects. The list of available AMFs for intersection geometric design is likely to increase as new research in this area is undertaken.

Table 6-15. AMFs Related to Geometric Design of Rural Unsignalized Intersections.

Section	Accident Modification Factor		
Cross section	Left-turn lane Shoulder width Intersection sight distance	Right-turn lane Median presence	Number of lanes Alignment skew angle

In some instances, an AMF is derived from a safety prediction model as the ratio of "intersection crash frequency with a changed condition" to "intersection crash frequency without the change." In other instances, the AMF is obtained directly from a before-after study. Occasionally, crash rates reported in the literature were used to derive an AMF.

Left-Turn Lane. Harwood et al. (*6*) investigated the relationship between the presence of left- and right-turn lanes and crash frequency. They examined the change in severe crash frequency at intersections that had a left-turn lane installed using a before-after study design. The results of their investigation are summarized in Table 6-16. The values shown in this table can be used to estimate the change in crashes at unsignalized intersections at which a left-turn lane is added to one or both of the major-road approaches.

Table 6-16. AMFs for Adding a Left-Turn Lane at a Rural Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches with Left-Turn Lanes Installed			
	One Approach		Both Approaches [1]	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	0.56	0.45	not applicable	
4	0.72	0.65	0.52	0.42

Note:
1 - AMFs for "Both Approaches" estimated as the square of the "One Approach" AMFs.

The crash rates presented in a previous part of this chapter reflect typical rural signalized intersection design. Data provided by Bauer and Harwood (*4*) indicate that the typical (i.e., base) condition for rural unsignalized intersections is "no left-turn lane provided." The values presented in Table 6-16 reflect this base condition.

Right-Turn Lane. Harwood et al. (*6*) also investigated the relationship between right-turn lane presence and unsignalized intersection crash frequency. Their recommended AMFs for the addition of a right-turn lane on the major-road approach to an unsignalized intersection are shown in Table 6-17. Harwood et al. recommend using these AMFs for any unsignalized intersection, regardless whether it has three or four legs.

Table 6-17. AMFs for Adding a Right-Turn Lane at a Rural Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches with Right-Turn Lanes Installed			
	One Approach		Both Approaches [1]	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	0.86	0.77	not applicable	
4	0.86	0.77	0.74	0.59

Note:
1 - AMFs for "Both Approaches" estimated as the square of the "One Approach" AMFs.

Number of Lanes on the Major and Minor Roads. Bauer and Harwood (*4*) developed several safety prediction models that relate traffic and geometric factors to severe crash frequency at rural unsignalized intersections. One of the factors included in several of their models was the number of through lanes on the major road. The coefficients in these models were used in a regression analysis to compute the AMFs listed in column 3 of Table 6-18. Details of this regression analysis are provided in the discussion associated with Table 6-11. The AMFs in Table 6-18 reflect a base condition of two through lanes on the major and minor roads.

Table 6-18. AMFs for Number of Through Lanes at Rural Unsignalized Intersections.

Road	Number of Through Lanes on Major Road	AMF_{lane}
Major	3 or fewer	1.00
	4 or 5	0.83
	6 or more	0.69
Minor	3 or fewer	1.00
	4 or more	0.83

Shoulder Width. Bauer and Harwood (*4*) developed several safety prediction models that relate outside shoulder width and other factors to severe crash frequency at rural, unsignalized intersections. The AMF that is derived from these models is shown below. It reflects a base shoulder width of 8 ft.

$$AMF_{sw} = e^{-0.030\,(W_s - 8)} \tag{6-33}$$

where:
AMF_{sw} = shoulder width accident modification factor; and
W_s = outside shoulder width, ft.

The regression coefficient of "-0.030" in Equation 6-33 was derived from a model for three-leg intersections. However, in the absence of information to the contrary, it is believed to be equally applicable to four-leg intersections.

Figure 6-7 illustrates the shoulder width AMF. The trend line shown suggests that 5 ft shoulders are associated with an AMF of 1.09 and implies that intersections with 5 ft shoulders experience 9 percent more crashes than those with 8 ft shoulders, all other factors unchanged.

Median Presence and Width. Bauer and Harwood (*4*) developed a safety prediction model for three-leg unsignalized intersections that includes a variable relating the presence of a median on the major road to the reported total crash frequency. The regression coefficients in this model were used to compute the relative effect of median presence on crash frequency. The computed AMFs are shown in column 3 of Table 6-19. These AMFs reflect the base condition of an undivided major road. In the absence of research to the contrary, this AMF is believed to be equally applicable to intersections with four legs.

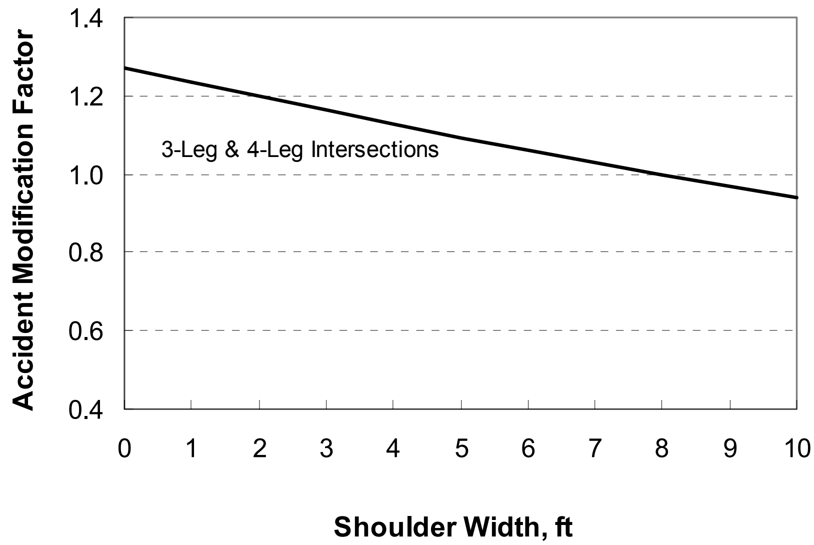


Figure 6-7. Shoulder Width AMF for Rural Unsignalized Intersections.

Table 6-19. AMFs for Median Presence at Rural Unsignalized Intersections.

Model Source	Median Type on Major Road	$AMF_{mp, base}$
Bauer & Harwood (*4*)	Divided	0.73
	Undivided	1.00

Additional research on the relationship between intersection median width and crash frequency has also been conducted by Vogt (*2*), Washington et al. (*7*), and Harwood et al. (*8*). All three research efforts found a relationship between crash frequency and median width at unsignalized intersections on rural highways. Each researcher developed a safety prediction model that includes various factors (including median width). The AMF that is derived from these models is shown below. It reflects a 16 ft median width as the base condition.

$$AMF_{mw} = e^{b(W_m - 16)} \qquad \textbf{(6-34)}$$

where:

AMF_{mw} = median width accident modification factor; and
W_m = median width, ft.

The value of variable b is shown in Table 6-20 for the three referenced sources. Also shown is the weighted average of the coefficients applicable to "all" crash severities. The weighted average can be used with Equation 6-34 to obtain the following AMF for median width:

$$AMF_{mw} = e^{-0.012(W_m - 16)} \qquad \textbf{(6-35)}$$

Table 6-20. Coefficient Analysis for Median Width at Rural Unsignalized Intersections.

Model Source	Road	Intersection Legs	Crash Severity	Coefficient *b*
Vogt (*2*)	Major	3	All	-0.0546
Washington et al. (*7*)	Major	3	All	-0.0106
Harwood et al. (*8*)	Major	4	All	-0.0122
	Major	4	Severe	-0.0135
			Weighted Average: [1]	**-0.012**

Note:
1 - Weighted average computed as: -0.012 = (-1/0.0546 − 1/0.0106 − 1/0.0122) / (1/0.0546^2 + 1/0.0106^2 + 1/0.0122^2).

Figure 6-8 illustrates the three expressions for the median width AMF that are based on the models listed in Table 6-20 for "all" crash severities. The AMF obtained from Equation 6-35 is shown with a thick bold line (and labeled "derived"). It is coincident with the trend obtained from the Harwood et al. (*8*) model. The trends shown suggest that an intersection with a 25 ft median has an AMF of about 0.90, which implies that it will be associated with 10 percent fewer crashes than an intersection with a 16 ft median.

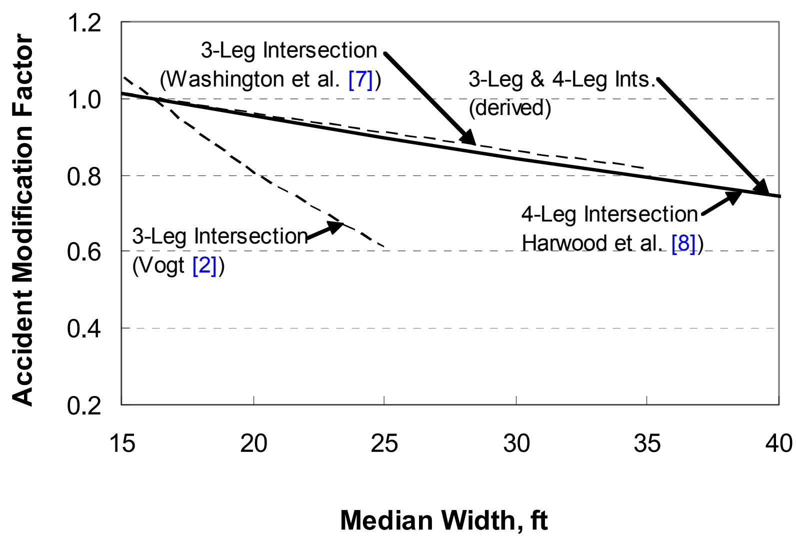


Figure 6-8. Median Width AMF for Rural Unsignalized Intersections.

In application, the base median presence and median width AMFs should be used together to evaluate the likely change in crash frequency due to introduction of a median in the vicinity of an unsignalized intersection. Both of these AMFs were developed using crash data reflecting all crash severities; however, the trends are rationalized to be applicable to severe crashes as well. The following equation demonstrates the manner by which the two AMFs should be combined:

$$AMF_{mp} = AMF_{mp,base} \times AMF_{mw} \qquad \textbf{(6-36)}$$

If a left-turn bay is present, $AMF_{mp,\,base}$ should equal 1.0.

Alignment Skew Angle. Vogt (2), Harwood et al. (3), and Washington et al. (7) developed safety prediction models that relate skew angle and other factors to total crash frequency at rural unsignalized intersections. The AMF that is derived from these models is shown below. It reflects a base condition of no skew (i.e., a 90-degree intersection).

$$AMF_{skew} = e^{b\, I_{sk}} \qquad \textbf{(6-37)}$$

where:

AMF_{skew} = skew angle accident modification factor, and

I_{sk} = skew angle of the intersection, degrees.

Skew angle is defined as the absolute value of the difference between the intersection angle and 90 degrees (i.e., I_{sk} = | intersection angle − 90 |). By this definition, skew angle is always a positive quantity when the two roads intersect at other than a 90 degree angle.

The values of b to be used with Equation 6-37 are listed in Table 6-21. The weighted average for severe crashes at three-leg intersections is computed from the two referenced models and equals 0.019. The weighted average for "all" crash severities is also computed and equals 0.0048. The ratio of these two values suggests that b is about four times larger for severe crashes than it is for "all" crash severities. This ratio was used to estimate the value of b for severe crashes at four-leg intersections as 0.021. The estimated values are used to derive the following skew angle AMF:

$$AMF_{skew} = \begin{bmatrix} e^{0.019\, I_{sk}} & if\ 3\ legs \\ e^{0.021\, I_{sk}} & if\ 4\ legs \end{bmatrix} \qquad \textbf{(6-38)}$$

Table 6-21. Coefficient Analysis for Skew Angle at Rural Unsignalized Intersections.

Model Source	Intersection Legs	Crash Severity	Coefficient *b*
Vogt (2)	3	Severe	0.023
Harwood et al. (3)	3	All	0.0040
Washington et al. (7)	3	Severe	0.0163
		All	0.0101
Weighted Average: [1]		Severe	**0.019**
		All	0.0048
Harwood et al. (3)	4	All	0.0054
		Severe	**0.021** [2]

Note:

1 - Weighted average computed as: 0.019 = (1/0.023 + 1/0.0163) / (1/0.023^2 + 1/0.0163^2); 0.0048 = (1/0.004 + 1/0.0101) / (1/0.004 + 1/0.0101^2).

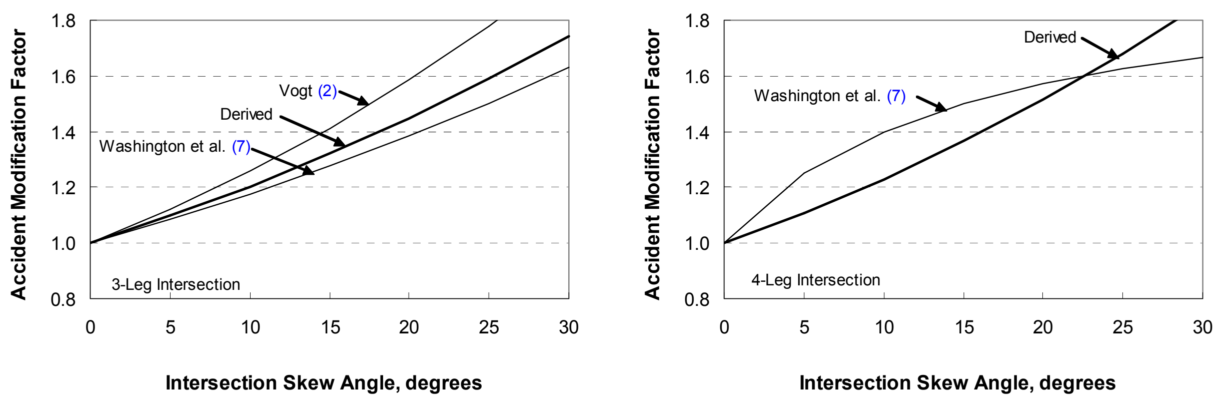
2 - Estimated as: 0.021 = 0.0054 × 0.019/0.0048.

Washington et al. (*7*) developed an AMF for skew angle at four-leg intersections; however, its formulation did not follow that of Equation 6-37. Their AMF is:

$$AMF_{skew,4} = 1 + \frac{0.048\, I_{sk}}{0.72 + 0.048\, I_{sk}} \qquad \textbf{(6-39)}$$

This equation was derived using severe crash data.

Figure 6-9 illustrates the two expressions for the skew angle AMF. The trend lines shown suggest that a three-leg intersection with 30 degrees of skew is associated with an AMF of 1.74 and implies that intersections with no skew experience 57 percent (=1/1.74) fewer crashes than those with 30 degrees of skew, all other factors unchanged. The AMF derived for four-leg intersections is consistent with that developed by Washington et al. (*7*) (i.e., Equation 6-39). Additional research is needed to determine which functional form is most appropriate. In the interim, Equation 6-38 is rationalized to offer the more reasonable estimate of the relationship between skew and crash frequency at four-leg intersections.



a. Three-Leg Intersections. *b. Four-Leg Intersections.*

Figure 6-9. Skew Angle AMF for Rural Unsignalized Intersections.

Intersection Sight Distance. During the development of AMFs for rural two-lane highways, Harwood et al. (*3*) convened an expert panel to determine the relationship between crash frequency and intersection sight distance deficiencies. Sight distance was considered to be "limited" if the available sight distance was less than that specified by AASHTO policy for a speed of 10 mph less than the major-road design speed. The AASHTO policy in reference is published in *A Policy on Geometric Design of Highways and Streets* (*9*).

The expert panel determined that crash frequency would increase by 5 percent for each intersection quadrant that had limited intersection sight distance. Thus, if two quadrants had limited sight distance, AMF_{SD} would be equal to 1.10 (= 1.00 + 0.05 × 2).

Access Control

Several safety prediction models have been developed for rural intersections that include a variable that relates driveway frequency to crash frequency. Driveway frequency was defined to be the count of driveways within 250 ft of the intersection on the major road. This count would include driveways on both sides of the road and on both major-road intersection legs. The regression coefficients in these models are shown in column 6 of Table 6-12. The AMF for each coefficient *b* can be computed as:

$$AMF_{nd} = e^{b\, d_n} \qquad \textbf{(6-40)}$$

where:

AMF_{nd} = driveway frequency accident modification factor; and
d_n = number of driveways on the major road within 250 ft of the intersection.

This equation reflects a base condition of no driveways. The coefficients listed in Table 6-12 consistently indicate an increase in crash frequency with an increase in driveway frequency, regardless of the control mode, driveway type, or number of intersection legs.

An analysis of the various coefficients in Table 6-12 indicates that the coefficient for unsignalized intersections is 0.056. This value can be combined with Equation 6-40 to obtain the following AMF for driveway frequency:

$$AMF_{nd} = e^{0.056\, d_n} \qquad \textbf{(6-41)}$$

Figure 6-10 illustrates the driveway frequency AMF for signalized intersections. The AMFs obtained from the coefficients reported by other researchers are shown with thin trend lines. The AMF obtained from Equation 6-41 is shown with a thick bold line (and labeled "derived"). The AMF trend lines attributed to others suggest that there is a possible influence of "number of legs" on the value of the AMF. However, the trends shown suggest that the addition of a fourth intersection leg increases the AMF value by a factor of 2.0 or more. This amount of increase is rationalized to be excessive and is likely due to colinearity in the coefficients derived from the models for four-leg unsignalized intersections.

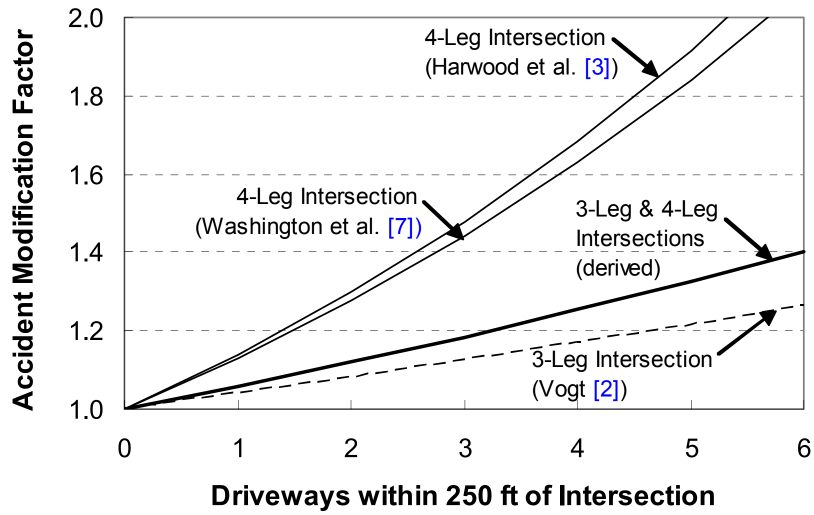


Figure 6-10. Driveway Frequency AMF for Rural Unsignalized Intersections.

Other Adjustment Factors

This section describes AMFs related to features of the unsignalized intersection that are not categorized as related to geometric design or access control. Specifically, the AMFs described in this section addresses truck presence and speed.

Truck Presence. Washington et al. (*7*) found a relationship between crash frequency and truck presence at unsignalized intersections on rural highways. They developed a safety prediction model that includes a factor representing truck percentage. The AMF that is derived from these models is shown below. It reflects a base condition of 9 percent trucks.

$$AMF_{tk} = e^{b(P_t - 9)} \qquad \textbf{(6-42)}$$

where:

AMF_{tk} = truck presence accident modification factor, and
P_t = percent trucks during the peak hour (average for all intersection movements), %.

The value of variable *b* is shown in Table 6-22. Also shown is the weighted average of the two coefficients. The weighted average can be used with Equation 6-42 to obtain the following AMF for truck presence:

$$AMF_{tk} = e^{-0.030(P_t - 9)} \qquad \textbf{(6-43)}$$

Table 6-22. Coefficient Analysis for Truck Presence at Rural Unsignalized Intersections.

Model Source	Intersection Legs	Crash Severity	Coefficient *b*
Washington et al. (7)	3	Severe	-0.0253
	4	Severe	-0.0520
		Weighted Average: [1]	**-0.030**

Note:
1 - Weighted average computed as: $-0.030 = (-1/0.0253 - 1/0.0520) / (1/0.0253^2 + 1/0.0520^2)$.

Figure 6-11 illustrates the two expressions for the truck presence AMF that are based on the models developed by Washington et al. (7). The AMF obtained from Equation 6-43 is shown with a thick bold line (and labeled "derived"). The derived trend suggests that an intersection with no trucks has an AMF of about 1.30, which implies that it will be associated with 30 percent more crashes than an intersection with 9 percent trucks. This relationship between truck presence and AMF value is similar to that found for urban street segments. It is likely that an increase in trucks does not make the intersection safer; rather, it probably indicates that drivers are more cautious when there are many trucks in the traffic stream.

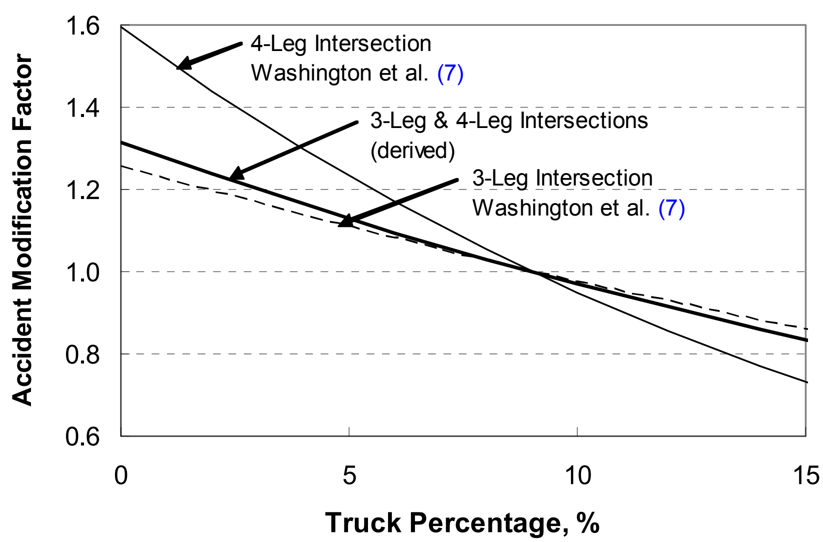


Figure 6-11. Truck Presence AMF for Rural Unsignalized Intersections.

Speed. Several safety prediction models have been developed for rural and urban intersections that include a variable that relates speed limit or design speed to crash frequency. The regression coefficients in these models were shown previously in Table 6-14. The AMF for each coefficient *b* can be computed as:

$$AMF_{sl} = e^{b(V-55)} \quad \textbf{(6-44)}$$

where:

AMF_{sl} = speed accident modification factor; and
V = major-road speed limit (or design speed), mph.

This equation reflects a base speed of 55 mph. The coefficient for rural unsignalized intersections is obtained from Table 6-14 as 0.019. This value can be combined with Equation 6-44 to obtain the following AMF for speed:

$$AMF_{sl} = e^{0.019\,(V-55)} \qquad \textbf{(6-45)}$$

Figure 6-12 illustrates the speed AMF for rural unsignalized intersections. The AMFs obtained from the coefficients reported by other researchers are shown with the thin trend line. The AMF obtained from Equation 6-45 is shown with a thick bold line (and labeled "derived"). The trends in the two AMFs indicate that lower speeds are associated with fewer severe crashes. These AMFs were derived from severe crash data. Hence, the trends found likely reflect the increase in crash severity with increasing speed.

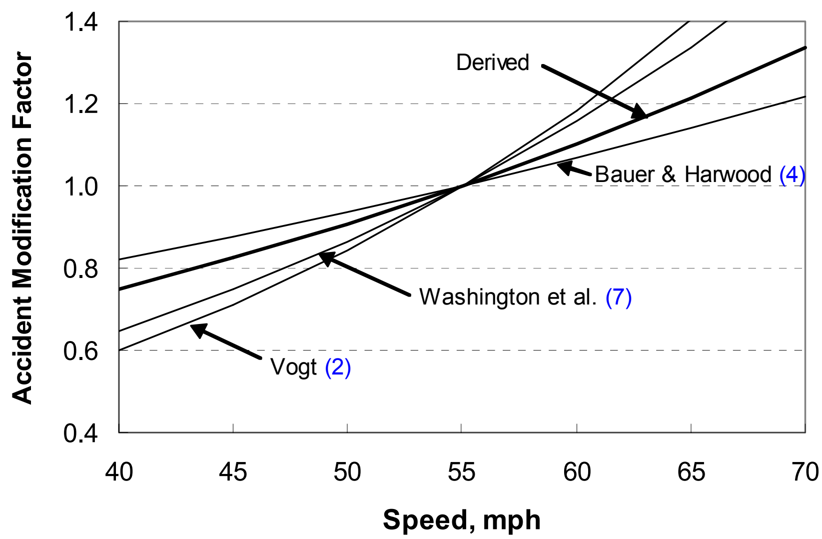


Figure 6-12. Speed AMF for Rural Unsignalized Intersections.

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Chapter 7
Urban Intersections

Page

INTRODUCTION

Intersections are a necessary consequence of a surface street system. They represent the point where two streets cross and thus, are points of significant potential conflict. On a statewide basis, more than one-half of all crashes in urban areas occur at intersections. The safe operation of an intersection requires the use of traffic control devices to separate the conflicting movements in time. These devices typically include yield sign, stop sign, or traffic signal. The geometric design of the intersection and the type of traffic control devices used (and, if signal control is used, the signal phasing and timing) have a significant effect on the safety and operation of the intersection. The accommodation of automobile, truck, pedestrian, and bicycle travel modes presents unique design challenges in the urban environment, and especially at intersections. The high cost of right-of-way in urban areas can present an additional challenge to intersection design.

The development of a safe, efficient, and economical urban intersection design may reflect consideration of a wide variety of design alternatives. A variety of techniques exist for estimating the operational benefits of alternatives; many are automated through software tools. Techniques for estimating construction and right-of-way costs are also available to the designer. Unfortunately, techniques for estimating the safety benefits of alternative designs are not as readily available. This chapter summarizes information in the literature that can be used to estimate the crash frequency associated with various urban intersection design alternatives.

Objective

The objective of this chapter is to synthesize information in the literature that quantitatively describes the relationship between various urban intersection design components and safety. This information is intended to provide a basis for the development of a procedure for estimating the safety benefit of alternative designs. This procedure is documented in Chapter 7 of the *Roadway Safety Design Workbook* (*1*).

The presentation consists of an examination of safety prediction models and accident modification factors (AMFs). Safety prediction models provide an estimate of the expected crash frequency for a typical urban intersection. They include variables for the volume of the conflicting traffic streams. They also include variables for other factors considered to be correlated with crash frequency (e.g., median type, functional class, etc.). One or more AMFs can be multiplied by the expected crash frequency obtained from the prediction model to produce an estimate of the expected crash frequency for a specific intersection.

Scope

This chapter addresses the safety of intersections in an urban area. As such, the crash types used to describe the level of safety are identified as "intersection related." The intersection relationship of a crash is indicated on the crash report or, in some instances, is determined by the researchers. If the researchers determine the intersection-relationship of a crash, it is oftentimes based on crash location relative to the intersection. Specifically, all crashes that occur within a specified distance back from the intersection are labeled as "intersection related." The combination of crash location and type (e.g., turn related or multi-vehicle) has also been used by some researchers to identify intersection relationship. Crashes that are not intersection related are referred to herein as "mid-block" crashes and are the subject of Chapter 4.

When available, safety relationships that estimate the frequency of *severe* (i.e., injury or fatal) crashes are given preference for inclusion in this document. This preference is due to a wide variation in reporting threshold among cities and states. This variation complicates the extrapolation of crash trends found in one location to another location. Moreover, it can confound the development of safety prediction models using data from multiple agencies. Reporting threshold is strongly correlated with the number of property-damage-only (PDO) crashes found in a crash database. Agencies with a high reporting threshold include relatively few PDO crashes in their database and vice versa. As a consequence, the total crash frequency for a given intersection will be high if it is located in a city with a low reporting threshold. This problem is minimized when crash data analyses, comparisons, and models are based on data pertaining only to severe crashes.

The relationships described in this chapter address the occurrence of vehicle-related crashes at urban intersections. They assume that the distribution of pedestrian and bicycle crashes remains unchanged, regardless of the change in design or traffic volume. Relationships that specifically focus on vehicle-pedestrian and vehicle-bicycle crashes on streets are not addressed.

Overview

This chapter documents a review of the literature related to urban intersection safety. The focus is on quantitative information that relates severe crash frequency to various geometric design components of the urban intersection. The review is not intended to be comprehensive in the context of referencing all relevant works that discuss urban intersection safety. Rather, the information presented herein is judged to be the most current information that is relevant to urban street design in Texas. It is also judged to be the most reliable based on a review of the statistical analysis techniques used and the explanation of trends.

Where appropriate, the safety relationships reported in the literature are compared herein, with some interpretation offered as an explanation for any differences noted. The relationships are typically presented as reported in the literature; however, the names or the units of some variables have been changed to facilitate their uniform presentation in this document.

This chapter is envisioned to be useful to design engineers who desire a more complete understanding of the relationship between various intersection design components and severe crash frequency. As noted previously, it is also intended to serve as the basis for the development of the safety evaluation procedure described in Chapter 7 of the *Roadway Safety Design Workbook* (*1*).

This chapter consists of two main parts. In the first part to follow, several safety prediction models reported in the literature are described. In the second part, accident modification factors are described for various geometric design and traffic control components. In each of these parts, there is a separate discussion of signalized and unsignalized intersections.

SAFETY PREDICTION MODELS

Described in this part of the chapter are several safety prediction models that were developed to estimate the expected frequency of intersection-related crashes. The discussion is separated into two sections with one section describing models that apply to signalized intersections and a second section describing models that apply to unsignalized intersections. At the end of each section, the models are compared in terms of the relationship between severe crash frequency and traffic volume.

All of the models presented in this part of the chapter make reference to the "major" and "minor" streets that form the intersection. These models are based on the assumption that the "major street" is the street with the higher volume of the two streets. It most instances, this assumption is in agreement with the number of lanes provided and the functional class of the two streets. However, in some instances, this assumption may mean that a street with more lanes or a higher functional classification but with lower volume will need to be specified as the "minor" street for the purpose of using a safety prediction model.

Urban Signalized Intersections

This section addresses safety prediction models for urban signalized intersections. Three sets of models are described and are identified by the names of their developers. They include:

- Lyon Models
- Bauer and Harwood Models
- McGee Models

Lyon Models

Lyon et al. (*2*) created a family of safety prediction models specifically for urban signalized intersections. The intersections in their database are located in the city of Toronto in Ontario, Canada. Separate models were developed for various combinations of intersection legs and functional classification of the intersecting streets. The following generalized model form was used for all combinations:

$$C_i = \alpha\ Q_{major}^{\beta_1}\ Q_{minor}^{\beta_2}\ e^{(\beta_3\ Q_{minor})} \qquad \textbf{(7-1)}$$

where:

C_i = frequency of severe intersection-related crashes at intersections with leg and classification combination i, crashes/yr;
Q_{major} = total daily volume on major street (both directions), veh/d;
Q_{minor} = total daily volume on minor street (both directions), veh/d; and
α, β_j = regression coefficients (j = 1, 2, 3).

Table 7-1 lists the values of α and β_j for each combination of intersection legs and functional class. Also listed are the coefficient values for an "overall" model for the three-leg and the four-leg intersection categories. This overall model is based on the full database, independent of the functional class of the intersecting streets.

Table 7-1. Calibration Coefficients for Lyon Models.

Number of Intersection Legs	Functional Classification		Model Calibration Coefficients			
	Major Street	**Minor Street**	**α**	**β_1**	**β_2**	**β_3**
4	Local	Local	0.000531	0.434	0.382	0.00000947
	Minor arterial	Local	0.000877			
		Collector	0.000992			
		Minor arterial	0.001458			
	Major arterial	Local	0.001087			
		Collector	0.001347			
		Minor arterial	0.001549			
		Major arterial	0.001632			
	Any	Any	0.000082	0.570	0.545	0.00000604
3 [a]	Minor arterial, collector, local	Minor arterial, collector, local	0.000204	0.588	0.375	0.0
	Major arterial	Local	0.000160			
		Collector	0.000232			
		Minor or major arterial	0.000251			
	Any	Any	0.000063	0.598	0.503	0.0

Note:
a - Values of α were multiplied by $0.5^{\beta 2}$ to convert minor-street volume from "entering volume" to "leg volume."

As Table 7-1 shows, the β_1, β_2, and β_3 values are the same for all functional class combinations of four-leg intersections (except when using all combinations) and for all functional class combinations of three-leg intersections. Only the value for α varies among the various functional classes.

Figures 7-1 and 7-2 illustrate the predicted severe crash frequency for the family of models developed by Lyon et al. (*2*). As the legend in each figure indicates, a ratio of major to minor volume has been specified to facilitate the comparison of the various models. The ratio used for four-leg intersections is 0.2 and that for three-leg intersections is 0.1. These ratios are typical of most intersections between a major arterial and a minor arterial or collector street.

The trends shown in Figures 7-1 and 7-2 indicate that the functional class of the intersecting streets is correlated with severe crash frequency. In general, intersections where the major street is classified as a major arterial and the minor street is classified as a major arterial, minor arterial, or collector experience more crashes than intersections where the major street is classified as a minor arterial, collector, or local. This finding suggests that intersections on streets that are functionally more "important" tend to experience more crashes than intersections on less important streets. It is likely a reflection of the more complex design and signalization environment (e.g., channelized turn lanes, left-turn phases, etc.) associated with "important" intersections. It may also reflect the likelihood that "important" intersections often operate more nearly at their capacity during many hours of the day. Lengthy delays and, possibly, congestion result in more dense traffic flows and increased driver anxiety that could lead to a reduction in the overall level of intersection safety.

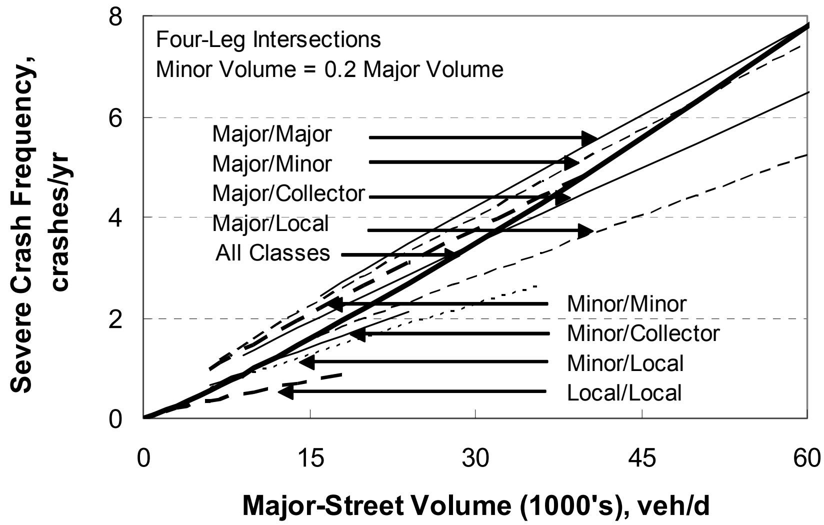


Figure 7-1. Crash Models Developed by Lyon et al. for Four-Leg Intersections.

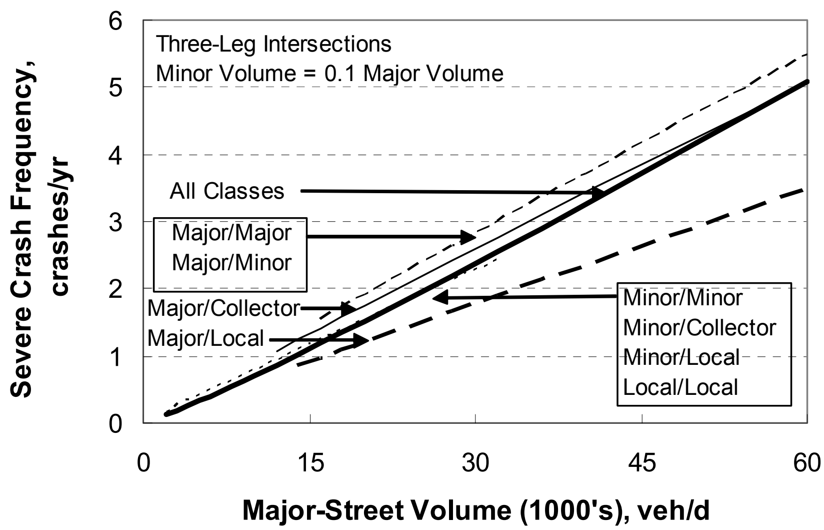


Figure 7-2. Crash Models Developed by Lyon et al. for Three-Leg Intersections.

A supplemental examination was conducted that explored the relationship between crash frequency and the number of intersection legs. To facilitate this examination, the volume ratio was set to a common value of 0.2. On streets where the major street is classified as a major arterial, the trends shown indicate that three-leg intersections tend to experience about 70 to 90 percent of the crashes that four-leg intersections experience, all other factors being the same. In contrast, on streets where the major street is classified as a minor arterial, collector, or local street; three-leg intersections experience about the same crash frequency as four-leg streets.

The characteristics of the data used by Lyon et al. (*2*) are summarized in Table 7-2. They used five years of crash data for each of 1716 urban intersections. The researchers defined all crashes occurring at, or within 65 ft of, the intersection as "intersection related."

Table 7-2. Database Characteristics for Signalized Intersection Safety Prediction Models.

Model Developers	Database Characteristics				
	Data Source	Years of Crash data	Intersection Relationship [1]	Intersection Legs	Number of Intersections
Lyon et al. (*2*)	City of Toronto	5	65 ft	3	306
				4	1410
Bauer & Harwood (*3*)	California Dept. of Transportation	3	250 ft	4	1342
McGee et al. (*4*)	California Dept. of Transportation	8	not available	3	170
				4	629

Note:
1 - Intersection relationship indicates the definition used to identify crash relationship to the intersection. Distances listed denote the distance back from the intersection within which a crash is denoted as "intersection related."

Bauer and Harwood Models

Bauer and Harwood (*3*) investigated the relationships between crash frequency and various intersection geometry and operational attributes. They developed a series of safety prediction models for a variety of intersection configurations and control conditions. The model they developed for four-leg urban signalized intersections is described in this subsection. The models they developed for three-leg and four-leg urban unsignalized intersections are discussed in a later section. The rural intersection models are described in Chapter 6.

Bauer and Harwood's model for predicting severe crash frequency at four-leg urban signalized intersections is:

$$C = 0.001066\ Q_{major}^{0.574}\ Q_{minor}^{0.215}\ e^{B_1 + B_2} \qquad \textbf{(7-2)}$$

with,

$$B_1 = -0.051\, I_{pt} + 0.400\, I_{fa} - 0.240\, I_{mp} - 0.290\, I_{ac} \qquad \textbf{(7-3)}$$

and

$$B_2 = -0.155\, I_{\min} - 0.163\, I_{maj3} - 0.151\, I_{maj4} + 0.005\, V_d \qquad \textbf{(7-4)}$$

where:

C = frequency of severe intersection-related crashes, crashes/yr;
I_{pt} = control type indicator variable (1 if intersection is pretimed; 0 if semi-actuated);
I_{fa} = control type indicator variable (1 if intersection is fully actuated; 0 if semi-actuated);
I_{mp} = signal phasing indicator variable (1 if more than 2 phases; 0 otherwise);
I_{min} = minor-street through lanes (1 if 3 or fewer lanes; 0 otherwise);

I_{ac} = major-street access control indicator variable (1 if no access control; 0 if partial control);
I_{maj3} = major-street through lanes (1 if 3 or fewer lanes; 0 otherwise);
I_{maj4} = major-street through lanes (1 if 4 or 5 lanes; 0 otherwise); and
V_d = major-street design speed, mph.

It should be noted that the model predicts fewer crashes when the major street does not have access control, relative to one with partial access control. No justification or rationale was offered by the researchers for this counter-intuitive trend.

The characteristics of the data used by Bauer and Harwood (*3*) are summarized in Table 7-2. As this table indicates, their models are based on three years of crash data for each of 1342 urban intersections. In contrast to Lyon et al. (*2*), Bauer and Harwood defined intersection-related crashes to be all crashes that occurred at, or within 250 ft of, the intersection.

McGee Models

McGee et al. (*4*) developed a series of safety prediction models for urban intersections. Collectively, the intersections represented five states and Canada. The vast majority of the data were obtained from the California Department of Transportation. They used the generalized equation shown previously in Equation 7-1 to develop models based solely on the California data. The calibration coefficients for the signalized intersection models are listed in Table 7-3. The calibrated model predicts the frequency of severe crashes.

Table 7-3. Calibration Coefficients for McGee Signalized Intersection Models.

Number of Intersection Legs	Model Calibration Coefficients			
	α	β_1	β_2	β_3
4	0.003180	0.4911	0.1975	0.0
3 [a]	0.000480	0.6370	0.1901	0.0

Note:
a - Value of α was multiplied by $0.5^{\beta 2}$ to convert minor-street volume from “entering volume” to “leg volume.”

As indicated by the data in Table 7-3, separate models were calibrated to the three-leg and four-leg intersections in the database. Although not apparent by inspection of the coefficients listed in the table, examination of model predictions indicates that three-leg intersections have between 60 and 80 percent of the crashes experienced by four-leg intersections, for similar volume levels. This finding is generally consistent with the trend noted previously for the Lyon et al. (*2*) model.

A second model was developed by McGee et al. (*4*) using the crash data for the other states (and Canada) combined; however, this model is not described herein because it does not distinguish between three-leg and four-leg intersections. The findings noted in the previous paragraph indicate that such a distinction should have been found in the combined database.

The characteristics of the data used by McGee et al. (*4*) are listed in Table 7-2. As indicated in the table, their models are based on eight years of crash data for each of 799 urban intersections. The researchers did not indicate the method they used to identify intersection-related crashes.

Comparison of Signalized Intersection Crash Models

The models described in the previous subsections are compared in this subsection. The objective of this comparison is to determine which model or models are reasonable in their prediction of severe crash frequency. To facilitate this comparison, the models are examined over a range of traffic volume levels. The models were grouped into three-leg and four-leg categories. For the Bauer and Harwood (*3*) model, the values of the other model variables were set at typical values for urban signalized intersections. These values are listed in Table 7-4.

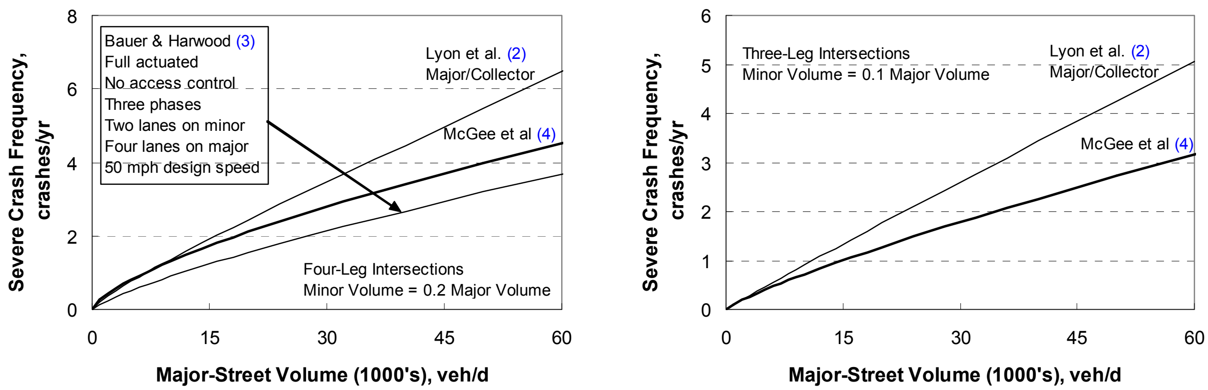
As the information in Table 7-4 indicates, only the Bauer and Harwood model included variables related to the design or operation of the intersection. The typical values listed are based on the median values for each variable in the California database, as described by Bauer and Harwood (*3*, Table 18).

Table 7-4. Typical Values Used for Urban Signalized Intersection Model Comparison.

Model Variable	Typical Value	Safety Prediction Model Developer		
		Lyon et al. (*2*)	Bauer & Harwood (*3*)	McGee et al. (*4*)
Signal control	fully actuated	--	✔	--
Access control	none	--	✔	--
Signal phasing	more than 2 phases	--	✔	--
Number of lanes on major street	4	--	✔	--
Number of lanes on minor street	2	--	✔	--
Design speed, mph	50	--	✔	--

Figure 7-3 illustrates the crash frequency predictions obtained from the various models. Bauer and Harwood did not develop a model for three-leg signalized intersections. The trends shown indicate a general agreement that the annual severe crash frequency for the four-leg intersections tends to equal between 4 and 6 crashes/yr when the major-street volume is 60,000 veh/d. The three-leg intersection trend lines exhibit a little more variability and suggest that severe crashes number between 3 and 5 crashes/yr for a similar volume level.

The trend lines shown in Figure 7-3 for Lyon et al. are consistently above those for McGee et al. and for Bauer and Harwood. This trend suggests that drivers in the city of Toronto (i.e., from which the Lyon data were obtained) are more inclined to have severe crashes than those in the state of California (from which the McGee and the Bauer and Harwood data were obtained). However, it could also be explained by possible differences in the definition of an injury crash. A review of crash data from five states (including California) and Toronto by McGee et al. (*4*) confirms that severe crash rates at four-leg intersections in Toronto are 20 percent larger than in California (40 percent larger for three-leg intersections). This finding is consistent with the trends in Figure 7-3. It should also be noted that crash rates in the other four states are even larger than that found in Toronto.



a. Four-Leg Intersections. *b. Three-Leg Intersections.*

Figure 7-3. Comparison of Safety Prediction Models for Urban Signalized Intersections.

Urban Unsignalized Intersections

This section addresses safety prediction models for urban unsignalized intersections. Specifically, these are two-way stop-controlled (TWSC) intersections. Two sets of models are described and are identified by the names of their developers. They include:

- Bauer and Harwood Models
- McGee Models

Bauer and Harwood Models

Bauer and Harwood (*3*) investigated the relationships between crash frequency and various intersection geometry and operational attributes. They developed a series of safety prediction models for a variety of intersection configurations and control conditions. The models they developed for three-leg and four-leg urban TWSC intersections are discussed in this subsection. The rural intersection models are described in Chapter 6.

Bauer and Harwood's model for predicting severe crash frequency at urban *four-leg* TWSC intersections is:

$$C = 0.003053\ Q_{major}^{0.584}\ Q_{minor}^{0.206}\ e^{B_1 + B_2} \qquad \textbf{(7-5)}$$

with,

$$B_1 = -0.081\ W_l - 0.747\ I_{lt} - 0.382\ I_{ac} - 0.079\ I_{ma} - 0.401\ I_{cl} \qquad \textbf{(7-6)}$$

and

$$B_2 = 0.282\, I_{maj3} + 0.049\, I_{maj4} - 0.020\, W_s - 0.300\, I_{frt} \tag{7-7}$$

where:

C = frequency of severe intersection-related crashes, crashes/yr;
W_l = major-street lane width, ft;
I_{lt} = major-street left-turn control indicator variable (1 if left turns are prohibited; 0 otherwise);
I_{ac} = major-street access control indicator variable (1 if no access control; 0 partial control);
I_{ma} = major-street functional class indicator variable (1 if minor arterial; 0 if principal arterial);
I_{cl} = major-street functional class indicator variable (1 if collector; 0 if principal arterial);
I_{maj3} = major-street through lanes (1 if 3 or fewer lanes; 0 otherwise);
I_{maj4} = major-street through lanes (1 if 4 or 5 lanes; 0 otherwise);
W_s = major-street outside shoulder width, ft; and
I_{frt} = minor-street channelization (1 if no free right-turn lane; 0 if free right-turn lane).

It should be noted that the model predicts fewer crashes when the major street does not have access control, relative to one with partial access control. No justification or rationale was offered by the researchers for this counter-intuitive trend.

Bauer and Harwood's model for predicting severe crash frequency at urban *three-leg* TWSC intersections is:

$$C = 0.000445\ Q_{major}^{0.696}\ Q_{minor}^{0.238}\ e^{B_1 + B_2} \tag{7-8}$$

with,

$$B_1 = -0.048\, W_l - 0.393\, I_{lt} - 0.581\, I_{frt} - 0.057\, I_{noltln} + 0.209\, I_{cbltln} \tag{7-9}$$

and

$$B_2 = -\ 0.182\, I_{div} + 0.094\, I_{nolite} \tag{7-10}$$

where:

C = frequency of severe intersection-related crashes, crashes/yr;
I_{noltln} = major-street left-turn channelization indicator variable (1 if no left-turn lane; 0 if painted left-turn lane);
I_{cbltln} = major-street left-turn channelization indicator variable (1 if curbed left-turn lane; 0 if painted left-turn lane);
I_{div} = major-street median indicator variable (1 if divided; 0 if undivided); and
I_{nolite} = intersection lighting indicator variable (1 if no lighting; 0 otherwise).

It should be noted that the model predicts fewer crashes on a major street that does not have left-turn channelization and more crashes on one that has curbed channelization. This trend would seem counter-intuitive. No justification or rationale was offered by the researchers for this counter-intuitive trend.

Although not apparent by inspection of the coefficients listed in Equations 7-5 through 7-10, an examination of model predictions indicates that three-leg intersections have between 50 and

65 percent of the crashes experienced by four-leg intersections, for similar volume levels. This finding is generally consistent with the trend noted previously for the signalized intersection models.

The characteristics of the data used by Bauer and Harwood (*3*) are summarized in Table 7-5. They used three years of crash data for each of 4399 urban intersections. The researchers defined all crashes occurring at, or within 250 ft of, the intersection as "intersection related."

Table 7-5. Database Characteristics for Unsignalized Intersection Safety Prediction Models.

Model Developers	Database Characteristics				
	Data Source	Years of Crash data	Intersection Relationship [1]	Intersection Legs	Number of Intersections
Bauer & Harwood (*3*)	California Dept. of Transportation	3	250 ft	3	3057
				4	1342
McGee et al. (*4*)	California Dept. of Transportation	8	not available	3	939
				4	479
	California, Florida, Maryland, Virginia, Wisconsin, Toronto	varies, 4 to 10	not available	3	99
				4	199

Note:
1 - Intersection relationship indicates the definition used to identify crash relationship to the intersection. Distances listed denote the distance back from the intersection within which a crash is denoted as "intersection related."

McGee Models

McGee et al. (*4*) developed a series of safety prediction models for urban intersections. Collectively, the intersections represented five states and Canada. The vast majority of the data were obtained from the California Department of Transportation (DOT). They used the generalized equation shown previously in Equation 7-1 to develop their models. They developed one set of models using the California DOT database and a separate set using the mix of states and Canada. The calibration coefficients for the unsignalized intersection models they developed are listed in Table 7-6. The calibrated model predicts the frequency of severe crashes.

Table 7-6. Calibration Coefficients for McGee Unsignalized Intersection Models.

Data Source	Number of Intersection Legs	Model Calibration Coefficients			
		α	β_1	β_2	β_3
California DOT	3 [a]	0.000146	0.7032	0.2011	0.0
	4	0.000340	0.6188	0.2946	0.0
California, Florida, Maryland, Virginia, Wisconsin, Toronto	3 [a]	0.0000011	0.968	0.558	0.0
	4	0.000426	0.499	0.430	0.0

Note:
a - Value of α was multiplied by $0.5^{\beta 2}$ to convert minor-street volume from "entering volume" to "leg volume."

As indicated by the data in Table 7-6, separate models were calibrated to the three-leg and four-leg intersections in the database. Although not apparent by inspection of the coefficients listed in the table, examination of model predictions based on the "California DOT" data indicates that three-leg intersections have 55 percent of the crashes experienced by four-leg intersections. This finding is generally consistent with the trend noted previously for the Bauer and Harwood unsignalized models and for the signalized intersection models. In contrast, this trend is not as stable in the model predictions based on the five states plus Toronto. A comparison of these two models indicates that three-leg intersections have between 35 and 105 percent of the crashes of the four-leg intersections. This wide variation is not explained by McGee et al. nor is it apparent in the crash rates quoted in their research report (*4*, Table 4). A detailed examination of these crash rates indicates that three-leg intersections have about 70 percent of the crashes experienced by four-leg intersections–a trend that is consistent with that found in previous models.

The characteristics of the data used by McGee et al. (*4*) are summarized in Table 7-5. As this table indicates, their "California DOT" models are based on eight years of crash data for each of 1418 unsignalized intersections. Their other two models are based on data from several states (and Toronto) and collectively represent 298 intersections. The researchers did not indicate the method they used to identify intersection-related crashes.

Comparison of Unsignalized Intersection Crash Models

The models described in the previous subsections are compared in this subsection. The objective of this comparison is to determine which model or models are reasonable in their prediction of severe crash frequency. To facilitate this comparison, the models are examined over a range of traffic volume levels. The models were grouped into three-leg and four-leg categories. For the Bauer and Harwood (*3*) model, the values of the model variables were set at typical values for urban unsignalized intersections. These values are listed in Table 7-7.

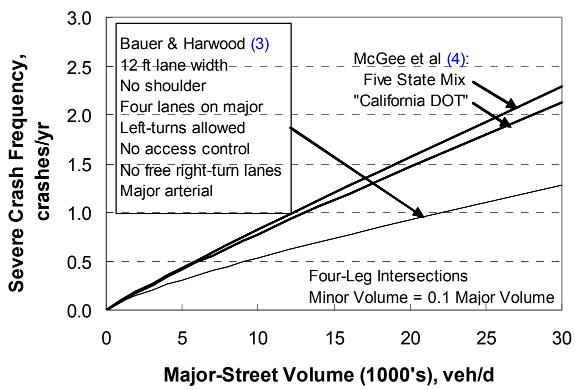
Table 7-7. Typical Values Used for Urban Unsignalized Intersection Model Comparison.

Model Variable	Typical Value	Safety Prediction Model Developer	
		McGee et al. (*4*)	Bauer & Harwood (*3*)
Lane width, ft	12	--	✔
Shoulder width, [1] ft	1.5	--	✔
Access control	none	--	✔
Left-turn operation on major street	allowed	--	✔
Number of lanes on major street	4	--	✔
Free right-turn lanes on minor street	no	--	✔
Median type (divided, undivided)	undivided street	--	✔
Left-turn channelization on major street	painted bay	--	✔
Intersection lighting	yes	--	✔

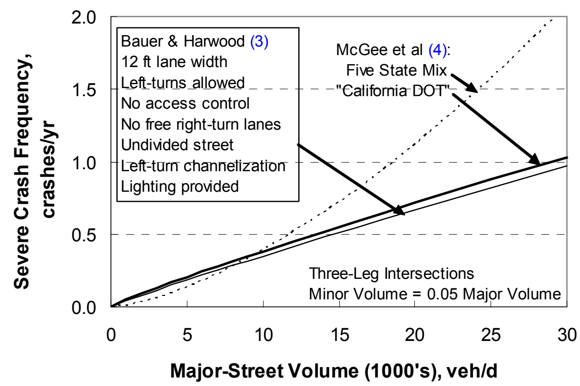
Note:
1 - It is assumed that curb-and-gutter is provided on the typical urban street instead of a shoulder. This cross section is assumed to have an equivalent "shoulder" width of 1.5 ft.

As the information in Table 7-7 indicates, only the Bauer and Harwood model included variables related to the design or operation of the intersection. The typical values listed are based on the median values for each variable in the California database, as described by Bauer and Harwood (*3*, Tables 10 and 14).

Figure 7-4 illustrates the crash frequency predictions obtained from the various models. The trends shown indicate a general agreement that the annual severe crash frequency for four-leg intersections tends to equal between 1.3 and 2.3 crashes/yr when the major-street volume is 30,000 veh/d.



a. Four-Leg Intersections.



b. Three-Leg Intersections.

Figure 7-4. Comparison of Safety Prediction Models for Urban Unsignalized Intersections.

The three-leg intersection trend lines collectively exhibit more variability than the four-leg models. As noted previously, the model developed by McGee et al. for three-leg intersections using the "five state" database yielded an illogical trend relative to four-leg intersection crash frequencies. This model is shown using a dashed trend line in Figure 7-4b. There appears to be some unexplained artifact in this model that cannot be explained by other models or by examination of the crash rates reported by the models' developers.

Development of Representative Intersection Crash Rates

The safety prediction models previously presented are examined more closely in this section for the purpose of developing representative intersection crash rates. To obtain the desired crash rate equation, Equation 7-1 was divided by the sum of entering flow and then multiplied by a conversion constant to obtain the traditional crash rate units–crashes per million entering vehicles. The resulting crash rate formula is:

$$CR = \left(\frac{10^6}{365}\right) \frac{\alpha \, Q_{major}^{\beta_1} \, Q_{minor}^{\beta_2}}{Q_{major} + Q_{minor}} \qquad \textbf{(7-11)}$$

with,

$$\alpha = \alpha_b + e^{(\beta_3 \, Q_{minor} \; + \; other \;\; terms)} \qquad \textbf{(7-12)}$$

where:

CR = severe crash rate for intersection-related crashes, crashes/million-vehicle-miles (mvm);
α = regression constant that combines α_b with all exponential terms at their average value; and
α_b = regression constant obtained from multivariate regression model.

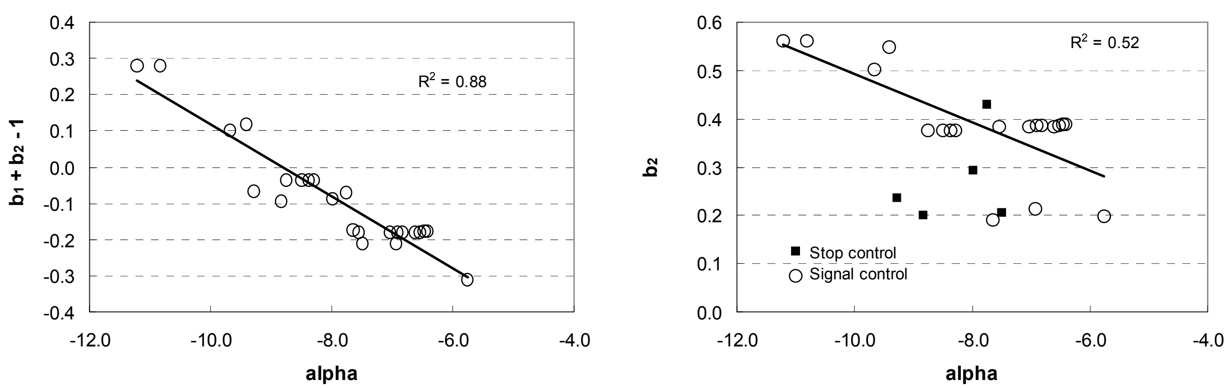
Equation 7-11 was simplified to the following form by substituting the ratio of the minor-street to major-street entering flows r:

$$CR = 2740 \, \frac{\alpha \, Q_{major}^{\beta_1 + \beta_2 - 1} \, r^{\beta_2}}{1 + r} \qquad \textbf{(7-13)}$$

where:

r = ratio of minor-street to major-street daily volumes (= Q_{minor} / Q_{major}).

As a next step in the examination, the relationship between α, β_1, β_2, and various variables describing intersection control and number of approach legs was investigated. For this analysis, a variable *alpha* was computed as the natural log of α (i.e., *alpha* = ln[α]). Figure 7-5a illustrates the relationship found between alpha and the sum "$\beta_1 + \beta_2 - 1$" (where β_1 and β_2 are labeled b_1 and b_2, respectively). Figure 7-5b illustrates the relationship found between α and β_2. In general, the trends illustrate that the variables *alpha*, β_1, and β_2 are correlated with each other and with the type of signal control. A subsequent analysis indicated a similar correlation between *alpha* and the number of intersection legs (R^2 = 0.31).



a. *Correlation between Alpha and "$\beta_1 + \beta_2 - 1$."* b. *Correlation between Alpha and β_2.*

Figure 7-5. Correlation between Model Coefficients.

The following relationships were derived based on the analysis of model coefficients:

$$\alpha = e^{-13.7 \; + \; 1.58 \, N_{leg}} \qquad \textbf{(7-14)}$$

with,

$$\beta_1 + \beta_2 - 1 = -0.880 - 0.100 \ln(\alpha) \qquad \textbf{(7-15)}$$

and

$$\beta_2 = -0.729 + 0.120 N_{leg} + 0.137 I_{sig} - 0.069 \ln(\alpha) \qquad \textbf{(7-16)}$$

where:

N_{leg} = number of intersection legs (3 or 4); and
I_{sig} = indicator variable for intersection control mode (1 for signalized; 0 otherwise).

Equations 7-14, 7-15, and 7-16 were used with Equation 7-13 to compute equivalent crash rates for various combinations of intersection control and number of approach legs. The findings from this analysis are presented in Tables 7-8 and 7-9. The rates listed in Table 7-8 are not sensitive to major-street volume. This outcome was a consequence of Equations 7-14 and 7-15. The value of "$\beta_1 + \beta_2 - 1$" obtained from Equation 7-15 was equal to 0.014 for three-leg intersections. This effectively eliminated the major-street flow variable in Equation 7-13 and produced the noted lack of sensitivity to major-street volume.

Table 7-8. Severe Crash Rates for Three-Leg Urban Intersections.

Control Mode	**Crash Rate, severe crashes per million-vehicle-miles**				
	Ratio of Minor-Street to Major-Street Volume				
	0.05	**0.10**	**0.15**	**0.20**	**0.25**
Unsignalized	0.18	0.21	0.22	0.22	0.23
Signalized	0.12	0.15	0.17	0.18	0.19

Table 7-9. Severe Crash Rates for Four-Leg Urban Intersections.

<table>
<tr><th rowspan="3">Control Mode</th><th rowspan="3">Major-Street Volume, veh/d</th><th colspan="5">Crash Rate, severe crashes per million-vehicle-miles</th></tr>
<tr><th colspan="5">Ratio of Minor-Street to Major-Street Volume</th></tr>
<tr><th>0.10</th><th>0.30</th><th>0.50</th><th>0.70</th><th>0.90</th></tr>
<tr><td rowspan="5">Unsignalized</td><td>5000</td><td>0.25</td><td>0.29</td><td>0.28</td><td>0.27</td><td>0.26</td></tr>
<tr><td>10,000</td><td>0.23</td><td>0.26</td><td>0.26</td><td>0.25</td><td>0.24</td></tr>
<tr><td>15,000</td><td>0.22</td><td>0.24</td><td>0.24</td><td>0.23</td><td>0.22</td></tr>
<tr><td>20,000</td><td>0.21</td><td colspan="4" rowspan="2">Intersection very likely to meet signal warrants</td></tr>
<tr><td>25,000</td><td>0.20</td></tr>
<tr><td rowspan="8">Signalized</td><td>5000</td><td>0.19</td><td>0.24</td><td>0.26</td><td>0.26</td><td>0.26</td></tr>
<tr><td>10,000</td><td>0.17</td><td>0.22</td><td>0.23</td><td>0.23</td><td>0.23</td></tr>
<tr><td>15,000</td><td>0.16</td><td>0.21</td><td>0.22</td><td>0.22</td><td>0.22</td></tr>
<tr><td>20,000</td><td>0.15</td><td>0.20</td><td>0.21</td><td>0.21</td><td>0.21</td></tr>
<tr><td>25,000</td><td>0.15</td><td>0.19</td><td>0.20</td><td>0.21</td><td>0.20</td></tr>
<tr><td>30,000</td><td>0.14</td><td>0.19</td><td>0.20</td><td>0.20</td><td>0.20</td></tr>
<tr><td>40,000</td><td>0.14</td><td>0.18</td><td>0.19</td><td>0.19</td><td>0.19</td></tr>
<tr><td>≥50,000</td><td>0.13</td><td>0.17</td><td>0.18</td><td>0.19</td><td>0.18</td></tr>
</table>

The following equation should be used to estimate severe crash frequency in conjunction with the crash rates listed in Tables 7-8 or 7-9:

$$C = CR \left(\frac{365}{10^6} \right) (Q_{major} + Q_{minor}) \tag{7-17}$$

ACCIDENT MODIFICATION FACTORS

This part of the chapter describes various accident modification factors that are related to the design of an urban intersection. The discussion is separated into AMFs that apply to signalized intersections and those that apply to unsignalized intersections.

Urban Signalized Intersections

This section identifies the AMFs that are applicable to signalized intersections in an urban environment. These factors were either derived from the models described in the previous section or extracted from other safety prediction models described in the literature. The focus of the discussion is on AMFs related to geometric design; however, AMFs related to other intersection features are also described.

Geometric Design

This subsection describes AMFs related to the geometric design of an urban signalized intersection. Topics specifically addressed are listed in Table 7-10. Many geometric design components or elements are not listed in this table (e.g., approach grade) that are also likely to have some correlation with severe crash frequency. However, a review of the literature did not reveal useful quantitative information describing these effects. The list of available AMFs for intersection geometric design is likely to increase as new research in this area is undertaken.

Table 7-10. AMFs Related to Geometric Design of Urban Signalized Intersections.

Section	Accident Modification Factor		
Cross section	Left-turn lane Lane width	Right-turn lane	Number of lanes

In some instances, an AMF is derived from a safety prediction model as the ratio of “intersection crash frequency with a changed condition” to “intersection crash frequency without the change.” In other instances, the AMF is obtained directly from a before-after study. Occasionally, crash rates reported in the literature were used to derive an AMF.

Left-Turn Lane. Harwood et al. (*5*) investigated the relationship between the presence of left- and right-turn lanes and crash frequency. They examined the change in severe crash frequency at intersections that had a left-turn lane installed using a before-after study design. The results of their investigation are summarized in Table 7-11. The values shown in this table can be used to estimate the change in crashes at a signalized intersection at which a left-turn lane is added to one or both of the major-street approaches.

Table 7-11. Effect of Adding a Left-Turn Lane at an Urban Signalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches with Left-Turn Lanes Installed			
	One Approach		Both Approaches [2]	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	0.93	0.94 [1]	not applicable	
4	0.90	0.91	0.81	0.83

Notes:
1 - Data not available from Harwood et al. (*5*). Value estimated using "All Crash" data as: 0.94 = 0.91/ 0.90 × 0.93.
2 - AMFs for "Both Approaches" estimated as the square of the "One Approach" AMFs.

The crash rates presented in a previous part of this chapter reflect typical urban signalized intersection design. Data provided by Bauer and Harwood (*3*) indicate that the typical (i.e., base) condition for signalized intersections is "left-turn lane provided." The values presented in Table 7-11 have been converted to equivalent AMFs to reflect this base condition. The resulting left-turn lane AMFs are listed in Table 7-12.

Table 7-12. AMFs for Excluding a Left-Turn Lane at an Urban Signalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches without Left-Turn Lanes			
	One Approach		Both Approaches	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	1.08	1.06	not applicable	
4	1.11	1.10	1.23	1.21

Right-Turn Lane. Harwood et al. (*5*) also investigated the relationship between right-turn lanes and signalized intersection crash frequency. Their recommended AMFs for the addition of a right-turn lane on the major-street approach to a signalized intersection are shown in Table 7-13. Harwood et al. recommend using these AMFs for any signalized intersection, regardless whether it has three or four legs. It should be noted that the addition of a right-turn lane may reduce the safety afforded to pedestrians, especially if the turn radius is large and turn speeds are high.

Number of Lanes on the Major and Minor Streets. Bauer and Harwood (*3*) developed several safety prediction models that relate the number of through lanes on the major and minor streets to crash frequency. The coefficients in these models were used in a regression analysis to compute the AMFs listed in column 3 of Table 7-14. Details of this regression analysis are provided in the discussion in Chapter 6 that is associated with Table 6-11. The AMFs in Table 7-14 reflect a base condition of four through lanes on the major street and two through lanes on the minor street.

Table 7-13. AMFs for Adding a Right-Turn Lane at an Urban Signalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches with Right-Turn Lanes Installed			
	One Approach		Both Approaches [2]	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	0.96 [1]	0.91 [1]	not applicable	
4	0.96	0.91	0.92	0.83

Notes:
1 - Harwood et al. (5) did not quantify AMFs for signalized intersections with three legs. They recommend the application of the "four-leg" AMFs to intersections with three legs.
2 - AMFs for "Both Approaches" estimated as the square of the "One Approach" AMFs.

Table 7-14. AMFs for Number of Through Lanes at Urban Signalized Intersections.

Street	Number of Through Lanes on Major Street	AMF_{lane}
Major	3 or fewer	0.99
	4 or 5	1.00
	6 or more	1.01
Minor	3 or fewer	1.00
	4 or more	1.01

Lane Width. Bauer and Harwood (3) developed a safety prediction model that relates several factors (including lane width) to total crash frequency (i.e., property-damage-only and severe crashes) at urban signalized intersections. The coefficient relating lane width to crash frequency is shown in the equation below. The base condition for this AMF is a 12 ft lane width. In the absence of research to the contrary, this AMF is believed to be equally applicable to severe crashes.

$$AMF_{lw} = e^{-0.053\,(W_l - 12)} \tag{7-18}$$

where:
AMF_{lw} = lane width accident modification factor; and
W_l = lane width, ft.

Figure 7-6 illustrates the AMF for lane width. The trend line shown suggests that 9 ft traffic lanes are associated with an AMF of 1.17 and implies that intersections with 9 ft lanes experience 17 percent more crashes than those with 12 ft lanes, all other factors unchanged.

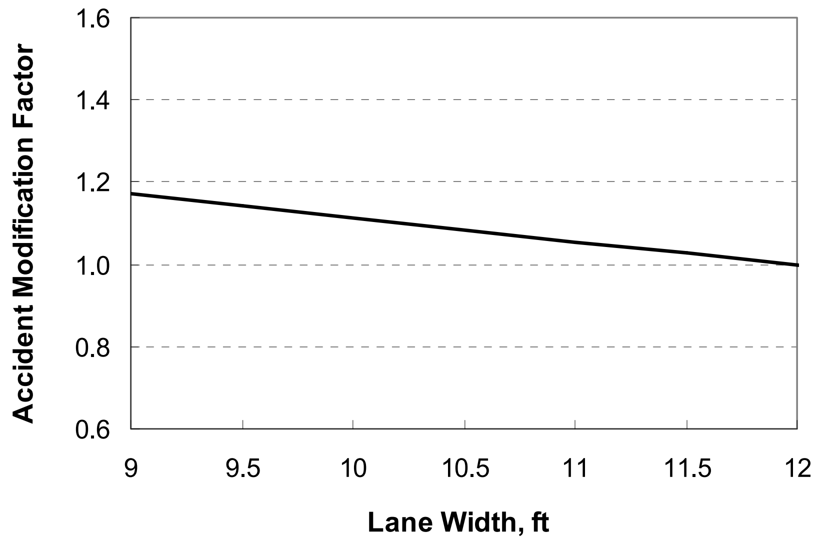


Figure 7-6. Lane Width AMF for Urban Signalized Intersections.

Other Adjustment Factors

This subsection describes AMFs related to features of the street that are not categorized as related to geometric design. The only AMF identified in the research that does not fit into these two categories relates to speed. Several safety prediction models have been developed for rural and urban intersections that include a variable that relates speed limit or design speed to crash frequency. The regression coefficients in these models that provide this relationship are listed in Table 6-14 in Chapter 6. They can be used with the following generalized equation to estimate the speed AMF:

$$AMF_{sl} = e^{b\,(V-40)} \qquad \textbf{(7-19)}$$

where:

AMF_{sl} = speed accident modification factor; and
V = major-street speed limit (or design speed), mph.

This equation reflects a base speed of 40 mph.

Regression analysis was used to identify the factors that influence the coefficient values. After separate evaluation of all factors, the regression model having the best fit included an indicator variable to account for the effect of area type. The findings from this analysis are described in Chapter 6. The coefficient for urban signalized intersections is obtained from Table 6-14 as 0.005. This value can be combined with Equation 7-19 to obtain the following AMF for speed:

$$AMF_{sl} = e^{0.005\,(V-40)} \qquad \textbf{(7-20)}$$

Figure 7-7 illustrates the speed AMF for urban signalized intersections. The AMF obtained from Equation 7-20 is shown with a thick bold line (and labeled "derived"). The AMF obtained from the coefficient reported by Bauer and Harwood (*3*) is also shown but it is coincident with the thick trend line. The trends in the two AMFs indicate that lower speeds are associated with fewer severe crashes. These AMFs were derived from severe crash data. Hence, the trends found likely reflect the increase in crash severity with increasing speed.

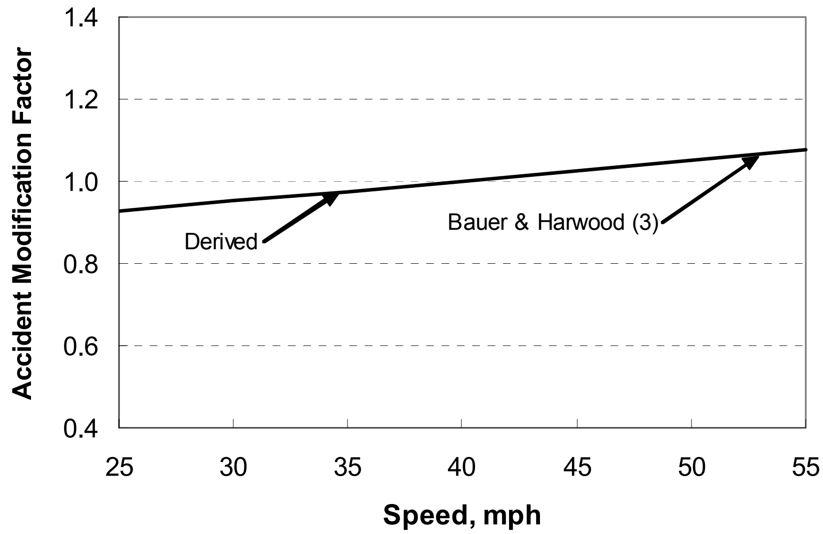


Figure 7-7. Speed AMF for Urban Signalized Intersections.

Urban Unsignalized Intersections

This section identifies the AMFs that are applicable to unsignalized intersections in an urban environment. Specifically, these are two-way stop-controlled intersections. The factors identified were either derived from the models described in a previous section or extracted from other safety prediction models described in the literature. The focus of the discussion is on AMFs related to geometric design; however, AMFs related to other intersection features are also described.

Geometric Design

This subsection describes AMFs related to the geometric design of an urban unsignalized intersection. Topics specifically addressed are listed in Table 7-15. Many geometric design components or elements are not listed in this table (e.g., approach grade) that are also likely to have some correlation with severe crash frequency. However, a review of the literature did not reveal useful quantitative information describing these effects. The list of available AMFs for intersection geometric design is likely to increase as new research in this area is undertaken.

Table 7-15. AMFs Related to Geometric Design of Urban Unsignalized Intersections.

Section	Accident Modification Factor		
Cross section	Left-turn lane Lane width	Right-turn lane Shoulder width	Number of lanes Median presence

In some instances, an AMF is derived from a safety prediction model as the ratio of "intersection crash frequency with a changed condition" to "intersection crash frequency without the change." In other instances, the AMF is obtained directly from a before-after study. Occasionally, crash rates reported in the literature were used to derive an AMF.

Left-Turn Lane. Harwood et al. (*5*) investigated the relationship between the presence of left- and right-turn lanes and crash frequency. They examined the change in severe crash frequency at intersections that had a left-turn lane installed using a before-after study design. The results of their investigation are summarized in Table 7-16. The values shown in this table can be used to estimate the change in crashes at unsignalized intersections at which a left-turn lane is added to one or both of the major-street approaches.

Table 7-16. Effect of Adding a Left-Turn Lane at an Urban Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches with Left-Turn Lanes Installed			
	One Approach		Both Approaches [2]	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	0.67	0.65 [1]	not applicable	
4	0.73	0.71	0.53	0.50

Notes:
1 - Data not available from Harwood et al. (*5*). Value estimated using "All Crash" data as: 0.65 = 0.71/ 0.73 × 0.67.
2 - AMFs for "Both Approaches" estimated as the square of the "One Approach" AMFs.

The crash rates presented in a previous part of this chapter reflect typical urban signalized intersection design. Data provided by Bauer and Harwood (*3*) indicate that the typical (i.e., base) condition for urban unsignalized intersections is "left-turn lane provided." The values presented in Table 7-16 have been converted to equivalent AMFs to reflect this base condition. The resulting left-turn lane AMFs are listed in Table 7-17.

Table 7-17. AMFs for Excluding a Left-Turn Lane at an Urban Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches without Left-Turn Lanes Installed			
	One Approach		Both Approaches	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	1.49	1.53	not applicable	
4	1.37	1.41	1.88	1.98

Right-Turn Lane. Harwood et al. (*5*) also investigated the relationship between right-turn lane presence and unsignalized intersection crash frequency. Their recommended AMFs for the addition of a right-turn lane on the major-street approach to an unsignalized intersection are shown in Table 7-18. Harwood et al. recommend using these AMFs for any unsignalized intersection, regardless whether it has three or four legs.

Table 7-18. AMFs for Adding a Right-Turn Lane at an Urban Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches with Right-Turn Lanes Installed			
	One Approach [1]		Both Approaches [1, 2]	
	All Crashes	Severe Crashes	All Crashes	Severe Crashes
3	0.86	0.77	not applicable	
4	0.86	0.77	0.74	0.59

Notes:

1 - Harwood et al. (*5*) did not quantify AMFs for urban unsignalized intersections. They recommend that the AMFs developed for rural four-leg unsignalized intersections can also be used for urban unsignalized intersections.

2 - AMFs for "Both Approaches" estimated as the square of the "One Approach" AMFs.

Number of Lanes on the Major and Minor Streets. Bauer and Harwood (*3*) developed several safety prediction models that relate the number of through lanes on the major and minor streets to the reported severe crash frequency. The coefficients in these models were used in a regression analysis to compute the AMFs listed in column 3 of Table 7-19. Details of this regression analysis are provided in the discussion in Chapter 6 that is associated with Table 6-11. The AMFs in Table 7-19 reflect a base condition of four through lanes on the major street and two through lanes on the minor street.

Table 7-19. AMFs for Number of Through Lanes at Urban Unsignalized Intersections.

Street	Number of Through Lanes on Major Street	AMF_{lane}
Major	3 or fewer	1.20
	4 or 5	1.00
	6 or more	0.83
Minor	3 or fewer	1.00
	4 or more	0.83

Lane Width. Bauer and Harwood (*3*) developed several safety prediction models that relate lane width and other factors to severe crash frequency at unsignalized intersections. The AMF that is derived from these models is shown below. It reflects a base condition of 12 ft lanes.

$$AMF_{lw} = e^{b\,(W_l - 12)} \qquad \textbf{(7-21)}$$

where:

AMF_{lw} = lane width accident modification factor; and
W_l = lane width, ft.

The value of variable *b* is shown in Table 7-20 for three-leg and four-leg intersections. Also shown is the weighted average of the two coefficients. The weighted average can be used with Equation 7-21 to obtain the following AMF for lane width:

$$AMF_{lw} = e^{-0.057(W_l - 12)} \qquad \textbf{(7-22)}$$

Table 7-20. Coefficient Analysis for Lane Width at Urban Unsignalized Intersections.

Number of Intersection Legs	Intersection Type	Crash Severity	Coefficient *b*
3	Urban unsignalized intersection	Severe	-0.048
4	Urban unsignalized intersection	Severe	-0.081
		Weighted Average: [1]	**-0.057**

Note:
1 - Weighted average computed as: -0.057 = (-1/0.048 − 1/0.081) / ($1/0.048^2 + 1/0.081^2$).

Figure 7-8 compares Equation 7-22 with the relationships derived from the Bauer and Harwood (*3*) models. The trend line associated with Equation 7-22 is labeled "derived." The trends shown suggest that 9 ft traffic lanes are associated with an AMF of about 1.18 and imply that intersections with 9 ft lanes experience 18 percent more crashes than those with 12 ft lanes, all other factors unchanged.

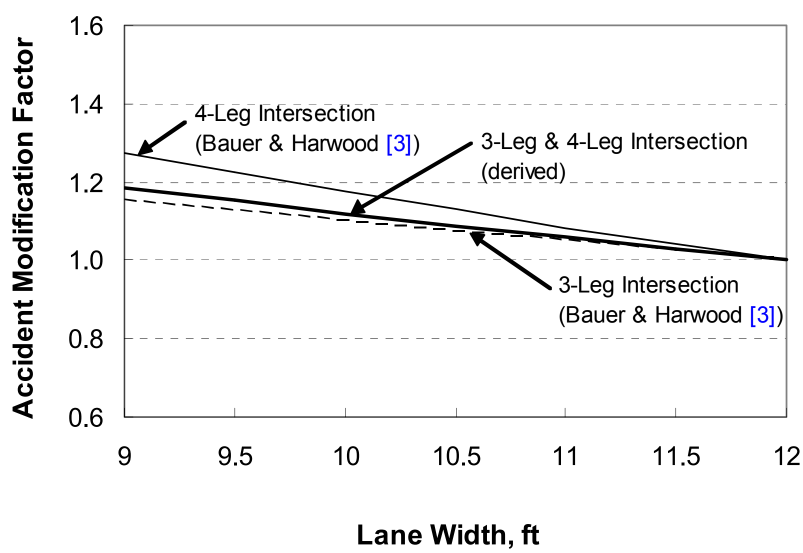


Figure 7-8. Lane Width AMF for Urban Unsignalized Intersections.

Shoulder Width. Bauer and Harwood (*3*) developed several safety prediction models that relate outside shoulder width and other factors to severe crash frequency at urban, unsignalized intersections. The AMF that is derived from these models is shown below. It reflects a base condition of curb-and-gutter, which is estimated to have an equivalent "shoulder" width of 1.5 ft.

$$AMF_{sw} = e^{-0.020\,(W_s - 1.5)} \qquad \textbf{(7-23)}$$

where:
AMF_{sw} = shoulder width accident modification factor; and
W_s = outside shoulder width, ft.

The regression coefficient of "-0.020" in Equation 7-23 was derived from a model for four-leg intersections. However, in the absence of information to the contrary, it is believed to be equally applicable to three-leg intersections.

Figure 7-9 illustrates the shoulder width AMF. The trend lines shown suggest that 5 ft shoulders are associated with an AMF of 0.93 and imply that intersections with 5 ft shoulders experience 7 percent fewer crashes than those with curb-and-gutter (assumed to equal a nominal 1.5 ft shoulder width), all other factors unchanged.

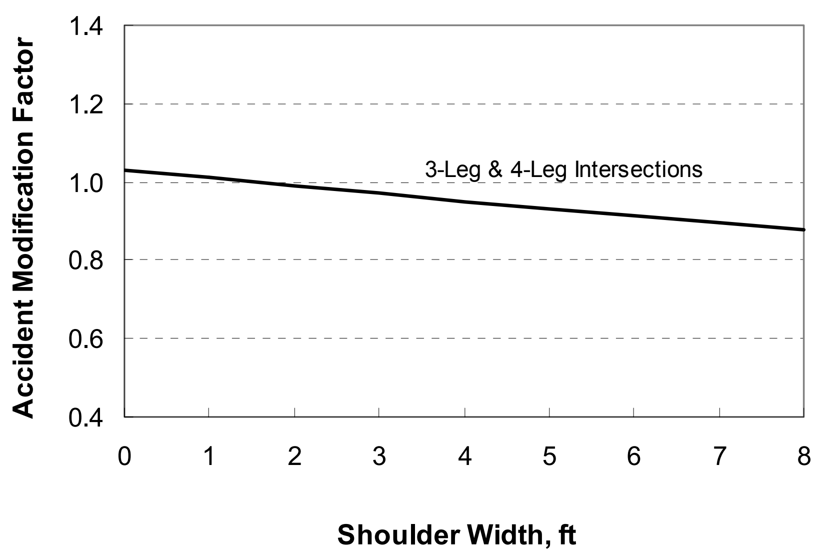


Figure 7-9. Shoulder Width AMF for Urban Unsignalized Intersections.

Median Presence and Width. Equation 7-8 presents the safety prediction model developed by Bauer and Harwood (*3*) for three-leg unsignalized intersections. It includes an AMF that relates the presence of a median on the major street to the reported severe crash frequency. The regression coefficient in this model was used to compute the relative effect of median presence on crash frequency. The computed AMFs are shown in column 3 of Table 7-21. These values were converted into the AMFs listed in column 3. These AMFs reflect the base condition of an undivided major street. In the absence of research to the contrary, this AMF is believed to be equally applicable to intersections with four legs.

Table 7-21. AMFs for Median Presence at Urban Unsignalized Intersections.

Model Source	Median Type on Major Street	$AMF_{mp,\ base}$
Bauer & Harwood (*3*)	Divided	0.83
	Undivided	1.00

Additional research on the relationship between intersection median width and crash frequency has also been conducted by Harwood et al. (*6*). They found a relationship between crash frequency and median width at signalized intersections on urban/suburban streets. The AMF that is derived to represent the effect of median width found by Harwood et al. is shown below. It reflects a 16 ft median width as the base condition.

$$AMF_{mw} = \begin{cases} e^{b(W_m - 16)} & : if\ W_m > 16 \\ 1.0 & : if\ W_m \leq 16 \end{cases} \qquad \textbf{(7-24)}$$

where:

AMF_{mw} = median width accident modification factor; and
W_m = median width, ft.

The value of variable *b* is shown in Table 7-22 for the referenced source. The coefficients applicable to severe crashes are recommended for application with Equation 7-24 to obtain a median width AMF for urban intersections. This equation is limited to median widths of more than 16 ft in recognition of the range of median widths represented in the crash data analyzed by Harwood et al. (*6*). It indicates that crash frequency increases with increasing median width. This trend was confirmed by Harwood et al. (*6*) in a follow-up study of conflicts and other undesirable maneuvers on the median roadway of wider intersections. Wide median roadways tend to be improperly used by drivers and, when complicated by the high median roadway volume found in urban areas, result in an increased propensity for multiple vehicle collision.

Table 7-22. Coefficient Analysis for Median Width at Urban Unsignalized Intersections.

Model Source	Road	Intersection Legs	Crash Severity	Coefficient *b*
Harwood et al. (*6*)	Major	3	All	0.0082
			Severe	**0.0076** [1]
		4	All	0.0173
			Severe	**0.0160**

Note:
1 - Estimated as: 0.0076 = 0.016 × 0.0082/0.0173.

Figure 7-10 illustrates the relationship between median width and the median width AMF based on severe crashes. The trends shown indicate that crash frequency increases with increasing median width, as discussed in the preceding paragraph. The trend lines suggest that a four-leg intersection with a 25 ft median has an AMF of about 1.15, which implies that it will be associated with 15 percent more crashes than a four-leg intersection with a 16 ft median.

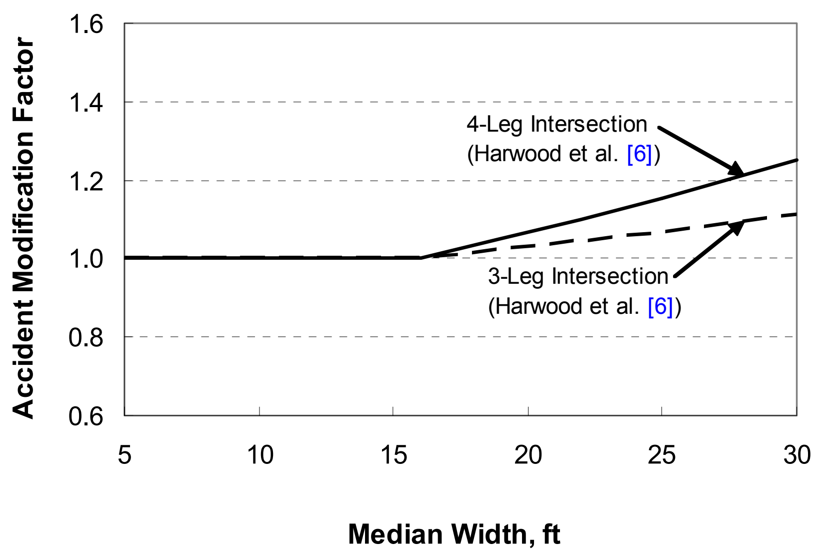


Figure 7-10. Median Width AMF for Urban Unsignalized Intersections.

In application, the base median presence and median width AMFs should be used together to evaluate the likely change in severe crash frequency due to introduction of a median in the vicinity of an unsignalized intersection. The following equation demonstrates the manner by which the two AMFs should be combined:

$$AMF_{mp} = AMF_{mp,base} \times AMF_{mw} \qquad \textbf{(7-25)}$$

If a left-turn bay is present, $AMF_{mp,\,base}$ should equal 1.0.

Other Adjustment Factors

This section describes AMFs related to features of the unsignalized intersection that are not categorized as related to geometric design. The only AMF found that does not fit into one of these two categories is that related to speed. Several safety prediction models have been developed for urban intersections that include a variable that relates speed limit or design speed to crash frequency. The regression coefficients in these models were shown previously in Table 6-14 of Chapter 6. The AMF for each coefficient b can be computed as:

$$AMF_{sl} = e^{b\,(V - 40)} \qquad \textbf{(7-26)}$$

where:

AMF_{sl} = speed accident modification factor; and
V = major-street speed limit (or design speed), mph.

This equation reflects a base speed of 40 mph. The coefficient for urban unsignalized intersections is obtained from Table 6-14 as 0.005. This value can be combined with Equation 7-26 to obtain the following AMF for speed:

$$AMF_{sl} = e^{0.005\,(V-40)} \qquad \textbf{(7-27)}$$

Figure 7-11 illustrates the speed AMF for urban unsignalized intersections. The trend line indicates that lower speeds are associated with fewer severe crashes. These AMFs were derived from severe crash data. Hence, the trends found likely reflect the increase in crash severity with increasing speed.

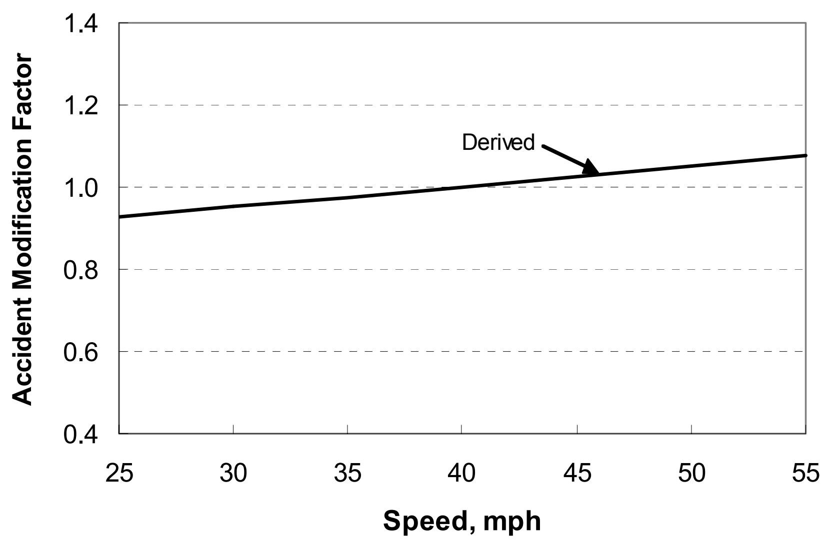


Figure 7-11. Speed AMF for Urban Unsignalized Intersections.

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