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Contractor's Final Report for NCHRP Project 17-68
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Summary

The first edition of the HSM included safety performance functions (SPFs) for roadway segments and intersections. However, not all intersection types are covered in the first edition of the HSM. This research was conducted to develop SPFs for new intersection configurations and traffic control types not covered in the first edition of the HSM, for consideration in the second edition of the HSM. Based on input received through a survey of state and local agencies as well as the research project panel, SPFs were developed for the following general intersection configurations and traffic control types:

- Rural and urban all-way stop-controlled intersections
- Rural three-leg intersections with signal control
- Intersections on high-speed urban and suburban arterials (i.e., roadways with speed limits greater than or equal to 50 mph)
- Urban five-leg intersections with signal control
- Three-leg intersections where the through movements make turning maneuvers at the intersections
- Crossroad ramp terminals at single-point diamond interchanges
- Crossroad ramp terminals at tight diamond interchanges

The research team coordinated with several state agencies to locate candidate intersections for use in developing the SPFs. Site characteristic data were collected for all candidate intersections to select a final list of sites for model development. In addition, crash and traffic volume data were assembled for model development.

The specific intersection configurations and traffic control types and severity levels for which SPFs were developed and recommended for consideration in the second edition of the HSM include:

Intersections with All-Way Stop Control

- Four-leg all-way stop-controlled intersections on rural two-lane highways
  - Total crashes
- Three-leg all-way stop-controlled intersections on urban and suburban arterials
  - Fatal-and-injury (FI) crashes
  - Property-damage-only (PDO) crashes
- Four-leg all-way stop-controlled intersections on urban and suburban arterials
  - FI crashes
  - PDO crashes

Rural Three-Leg Intersections with Signal Control

- Three-leg signalized intersections on rural two-lane highways
  - Total crashes
- Three-leg signalized intersections on rural multilane highways
  - Total crashes
  - FI crashes

Intersections on High-Speed Urban and Suburban Arterials

- Three-leg stop-controlled intersections on high-speed urban and suburban arterials
  - Multiple-vehicle (MV) total crashes
  - MV FI crashes
  - MV PDO crashes
  - Single-vehicle (SV) total crashes
  - SV FI crashes
  - SV PDO crashes
- Three-leg signalized intersections on high-speed urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV FI crashes
  - SV PDO crashes
- Four-leg stop-controlled intersections on high-speed urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV FI crashes
  - SV PDO crashes
- Four-leg signalized intersections on high-speed urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV FI crashes
  - SV PDO crashes

Five-Leg Intersections with Signal Control

- Five-leg signalized intersections on urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV FI crashes
  - SV PDO crashes

Three-Leg Intersections Where the Through Movement Makes a Turning Maneuver at the Intersection

- Three-leg turning intersections on rural two-lane highways
  - Total crashes
- Three-leg turning intersections on urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV PDO crashes

Crossroad Ramp Terminals at Single-Point Diamond Interchanges

- Crossroad ramp terminals at single-point diamond interchanges
  - FI crashes
  - PDO crashes

Crossroad Ramp Terminals at Tight Diamond Interchanges

- Crossroad ramp terminals at tight diamond interchanges
  - FI crashes
  - PDO crashes

In addition to formulating new SPFs, development of SDFs for the new intersection configurations and traffic control types was explored for potential use in combination with the SPFs to estimate crash severity as a function of geometric design elements and traffic control features. However, due to challenges and inconsistencies in developing and interpreting the SDFs, it is recommended for the second edition of the HSM that crash severity for the new intersection configurations and traffic control types be addressed in a manner consistent with existing methods in Chapters 10, 11, and 12 of the first edition of the HSM, without use of SDFs.

Draft text recommended for consideration in the second edition of the HSM is provided as an appendix to the report. In addition, the spreadsheet tools developed as part of NCHRP

Project 17-38 were updated to incorporate the new intersection crash prediction models developed as part of this research including intersections with all-way stop control, three-leg intersections with signal control on rural highways, intersections on high-speed urban and suburban arterials, five-leg intersections, and three-leg intersections where the through movement makes a turning maneuver at the intersection; and an unlocked version of the Enhanced Interchange Safety Analysis Tool (ISATe) that includes a module for ramp terminals and excludes modules for freeways and ramps was modified to incorporate the new crash prediction models for ramp terminals at single-point diamond interchanges and tight diamond interchanges.

Chapter 1. Introduction

1.1 Background

In May 2010, the American Association of State Highway and Transportation Officials (AASHTO) published the first edition of the HSM. This was an important step forward in providing quantitative safety analysis tools to inform decisions made by transportation agencies. HSM Part C includes predictive methods that can be used to anticipate the safety performance of new facilities, assess the safety performance of existing facilities, or estimate the expected effectiveness of proposed improvements to existing facilities. The HSM has become a key safety prediction tool, and state transportation agencies are gaining experience using the HSM in different planning and project contexts. In preparing the first edition of the HSM, decisions that determined which facility types would be addressed by the predictive methods chapters were made based on availability of data, funding limitations, and highway agency priorities. Since the preparation and publication of the first edition of the HSM, several National Cooperative Highway Research Program (NCHRP) projects have been funded to expand the safety knowledge and improve the crash prediction methods provided in Part C of the first edition of the HSM.

The HSM Part C presents predictive methods for estimating the expected average crash frequency for specific intersection configurations and traffic control types. The HSM Part C methods can also be applied to multiple roadway segment and intersection sites to estimate the safety performance at the project, corridor, or network levels. The estimation of expected average crash frequency with the Part C methods uses a combination of SPFs, crash modification factors (CMFs), calibration factors, and (when applicable) observed crash data. Table 1 shows the intersection configurations and traffic control types for which models are available in the HSM Part C chapters (Chapters 10, 11, and 12) and the 2014 Supplement to the HSM (Chapter 19) (AASHTO, 2014), which includes crossroad ramp terminal models. In summary, the first edition of the HSM provides the capability to analyze the safety performance of approximately thirteen intersection configurations and traffic control types. Many intersection and traffic control types such as all-way stop-controlled intersections and rural three-leg signalized intersections are not addressed within the first edition of the HSM.

Table 1. Intersection types addressed by predictive methods in the first edition of the HSM

Intersection Type	HSM Chapter			
	10	11	12	19
Three-leg intersections with stop control on minor approach	X	X	X	
Four-leg intersections with stop control on minor approaches	X	X	X	
Three-leg intersections with signal control			X	
Four-leg intersections with signal control	X	X	X	
Diamond ramp terminals at crossroad				X
Parclo ramp terminals at crossroad				X
Free-flow ramp terminals at crossroad				X

The science of crash frequency prediction and crash severity prediction has evolved as the HSM was developed. The earliest HSM Part C chapter—Chapter 10 on rural two-lane, two-way roads—was developed as a prototype with no new research beyond what was available at the time of its development. It contains SPFs for all crash severities combined, with tabulated severity distributions in the form of proportions available to separate the total crash frequency predictions into crash frequencies for individual crash severity levels. HSM Chapters 11 and 12 contain separate SPFs by crash severity level. HSM Chapter 19 for ramp terminals contains SPFs to predict the frequency of all FI crash severity levels combined and then implements crash SDFs to separate the FI crash frequency into frequencies by individual severity level as a function of ramp terminal characteristics. Table 2 summarizes the treatment of crash severity in the various HSM Part C chapters.

Table 2. Treatment of severity in current HSM models

HSM Chapter	Treatment of Severity
10	Tabulated crash severity distributions
11	Separate SPFs for KAB and KABC crashes
12	Separate SPFs for KABC and PDO crashes
19	SPFs for FI crashes with crash SDFs and SPFs for PDO crashes

Crash severity levels: fatal (K), incapacitating injury (A), non-incapacitating injury (B), possible injury (C), and PDO.

In terms of addressing crash types within the HSM predictive methods for at-grade intersections, Chapters 10, 11, and 19 use tabulated collision type distributions that can be applied to crash predictions of all collision types combined, while HSM Chapter 12 contains separate SPFs for predicting single- and MV crashes at intersections.

CMFs are used in the existing crash prediction models to account for the effects of intersection skew angle, presence of left- and right-turn lanes, signal phasing, right-turn-on-red, red light cameras, and lighting.

This report presents research conducted to develop crash prediction models for additional intersection configurations and traffic control types not currently addressed in the first edition of the HSM.

1.2 Research Objective and Scope

The objective of this research was to develop new intersection crash predictive models for consideration in the second edition of the HSM that are consistent with existing methods in HSM Part C and comprehensive in their ability to address a wide range of intersection configurations and traffic control types in rural and urban areas. The main focus of the research was on:

- Developing SPFs for intersection configurations and traffic control types not currently addressed in the HSM Part C.
- Developing SDFs to be used in combination with SPFs to estimate crash severity as a function of geometric design elements and traffic control features.

The crash prediction methods developed in this research include SPFs, CMFs, and SDFs as applicable in a format consistent with the predictive models in the existing HSM Part C and the 2014 Supplement to the HSM (Chapter 19). Data and methodologies used to develop the predictive models consider traffic volumes on all intersecting roads and streets as well as design elements and traffic control features considered by engineers and planners during the project development process. Roundabouts are not addressed in this research, as new crash prediction models were recently developed for possible inclusion in the second edition of the HSM as part of a separate study (NCHRP Project 17-70, *Development of Roundabout Crash Prediction Models and Methods*).

Crash prediction models were developed for the following intersection configurations and traffic control types for consideration in the second edition of the HSM:

- Intersections with all-way stop control
  - Rural four-leg intersections with all-way stop control
  - Urban and suburban three-leg intersections with all-way stop control
  - Urban and suburban four-leg intersections with all-way stop control
- Three-leg intersections with signal control on rural highways
  - Three-leg intersections with signal control on rural two-lane highways
  - Three-leg intersections with signal control on rural multilane highways
- Intersections on high-speed urban and suburban arterials
  - Three-leg intersections with minor road stop control
  - Three-leg intersections with signal control
  - Four-leg intersections with minor road stop control
  - Four-leg intersections with signal control
- Urban and suburban five-leg intersections with signal control
- Three-leg intersections where the through movements make turning maneuvers at the intersections
  - Three-leg intersections on rural two-lane highways
  - Three-leg intersections on urban and suburban arterials
- Crossroad ramp terminals at single-point diamond interchanges
- Crossroad ramp terminals at tight diamond interchanges

NOTE: A crash prediction model for three-leg all-way stop-controlled intersections on rural two-lane highways was developed as part of this research and is included in this report, but due to limited sample size the model was not recommended for consideration in the second edition of the HSM.

1.3 Overview of Research Methodology

In Phase I of the research, the research team reviewed and summarized literature related to current HSM intersection crash prediction methods; protocols, best practices, and emerging approaches for predictive model development; and current knowledge related to intersection safety. The research team also surveyed transportation agencies to gain knowledge about their experience with the current HSM intersection predictive methods and assess their needs and priorities as they relate to additional (new) intersection models and/or expanded capabilities of existing models. Based on the results of the literature review and survey, the research team identified and prioritized the types of intersection configurations and traffic control types not currently addressed in the HSM for further consideration in this research. The research team then developed work plans for creating crash prediction models for the higher priority intersection configurations and traffic control types, including:

- Intersections with all-way stop control
- Three-leg intersections with signal control on rural two-lane and multilane highways
- Intersections on high-speed expressways
- Three-leg intersections where the through movements make turning maneuvers at the intersections
- Three-leg intersections with a commercial driveway forming a fourth leg
- Five-leg intersections
- Single-point diamond ramp terminals
- Indirect left-turn intersections (i.e., U-turns or J-turns)

The work plans addressed site selection, data collection, database development, and model development.

Other intersection configurations and traffic control types considered for model development but for which work plans for possible execution in Phase II were not developed included: intersections with yield or no control, six-or-more-leg intersections, and diverging-diamond ramp terminals. Work plans were not developed for these intersection configurations and traffic control types due to a combination of priorities from the HSM user survey and the likelihood of successful model development with a sufficient number of sites, exposure, and crash data. For example, because diverging-diamond ramp terminals are relatively new in the United States, limited years of crash data were available for model development at the time of this research. Therefore, the research team did not create a work plan for developing crash prediction models for diverging-diamond ramp terminals.

In Phase II of the research, based on a combination of priorities from the HSM user survey, the likelihood of successful model development, and cost, the research team executed the approved work plans to develop crash prediction models for intersections with all-way stop control, three-leg intersections with signal control on rural two-lane and multilane highways, intersections on high-speed expressways, five-leg intersections, and single-point diamond ramp terminals. The research team updated existing spreadsheet tools to include the new crash prediction models developed as part of this research, conducted sensitivity analyses to check that the results were reasonable, and updated/revised the crash prediction models as necessary. In addition, the research team developed recommended text for consideration in the second edition of the HSM and prepared portions of this report that document Phases I and II of the research.
In Phase III of the research, the research team executed the approved work plan to develop crash prediction models for three-leg intersections where the through movements make turning maneuvers at the intersections and adapted and executed the work plan for crossroad ramp

terminals at single-point diamond interchanges to address crossroad ramp terminals at tight diamond interchanges. Similar to Phase II, the research team updated existing spreadsheet tools to include the new crash prediction models developed in Phase III of this research, conducted sensitivity analyses to check that the results made sense, updated/revised the crash prediction models as necessary, developed recommended text for consideration in the second edition of the HSM, and prepared portions of this report that document Phase III of the research. Throughout the course of the research, the research team kept abreast of other ongoing research related to the HSM that could potentially impact the direction of this research.

1.4 Outline of Report

This report presents an overview of research conducted to develop new intersection crash predictive models for consideration in the second edition of the HSM, for intersection configurations and traffic control types not addressed in the first edition of the HSM. The remainder of this report is organized as follows:

- Chapter 2. Literature Review and Survey of Practice
- Chapter 3. Development of Models for Use in HSM Crash Prediction Methods: Intersections with All-Way Stop Control
- Chapter 4. Development of Models for Use in HSM Crash Prediction Methods: Three-Leg Intersections with Signal Control on Rural Highways
- Chapter 5. Development of Models for Use in HSM Crash Prediction Methods: Intersections on High-Speed Urban and Suburban Arterials
- Chapter 6. Development of Models for Use in HSM Crash Prediction Methods: Five-Leg Intersections
- Chapter 7. Development of Models for Use in HSM Crash Prediction Methods: Three-Leg Intersections where the Through Movements Make Turning Maneuvers at the Intersections
- Chapter 8. Development of Models for Use in HSM Crash Prediction Methods: Crossroad Ramp Terminals at Single-Point Diamond Interchanges
- Chapter 9. Development of Models for Use in HSM Crash Prediction Methods: Crossroad Ramp Terminals at Tight Diamond Interchanges
- Chapter 10. Conclusions and Recommendations
- Chapter 11. References
- Chapter 12. Abbreviations, Acronyms, Initialisms, and Symbols

Appendix A—Draft Text for the Second Edition of the HSM

HSM Chapter 10—Predictive Method for Rural Two-Lane, Two-Way Roads
HSM Chapter 11—Predictive Method for Rural Multilane Highways
HSM Chapter 12—Predictive Method for Urban and Suburban Arterials
HSM Chapter 19—Predictive Method for Ramps

Chapter 2.
Literature Review and Survey of Practice

This section summarizes literature relevant to the objectives of this research and results of a survey of transportation agencies intended to gain knowledge about their experience with the current HSM intersection predictive methods and assess their needs and priorities as they relate to additional (new) intersection models and/or expanded capabilities of existing models. Information is summarized according to the following topics:

- Review of current HSM intersection crash prediction methods
- Protocols, best practices, and emerging approaches for predictive model development
- Current knowledge related to intersection safety
- Survey of current practice and crash prediction needs
- Summary of current knowledge related to intersection crash prediction modeling

2.1 Review of Current HSM Intersection Crash Prediction Methods

The current HSM predictive models for intersections were developed in the following order:

- HSM Chapter 10—intersections on rural two-lane, two-way roads
- HSM Chapter 11—intersections on rural multilane highways
- HSM Chapter 12—intersections on urban and suburban arterials
- HSM Chapter 19—crossroad ramp terminals at interchanges

HSM Chapters 10, 11, and 12 are included in the first edition of the HSM published in 2010. Chapter 19 was published as a 2014 Supplement to the HSM.

2.1.1 Concepts Common Across HSM Intersection Predictive Methods

At the most disaggregate level, the HSM predictive methods provide procedures to estimate the expected average crash frequency for individual sites, which are either homogenous roadway segments or intersections (see Figure 1). At-grade intersections are the specific focus of this research. Applying predictive methods for at-grade intersections results in estimates of the expected average crash frequency due to the presence of intersections. This includes crashes occurring within the limits of the intersection (Region A in Figure 1) as well as intersection-related crashes that occur on the intersection legs (Region B in Figure 1). The latter are crashes classified on the crash report as intersection-related or crashes having characteristics consistent with intersection-related crashes (e.g., rear-end collisions in queues). The HSM Appendix A recommends using the intersection-related designation on the crash report when such a field is available to identify the intersection-related crashes that have occurred on the intersection legs.

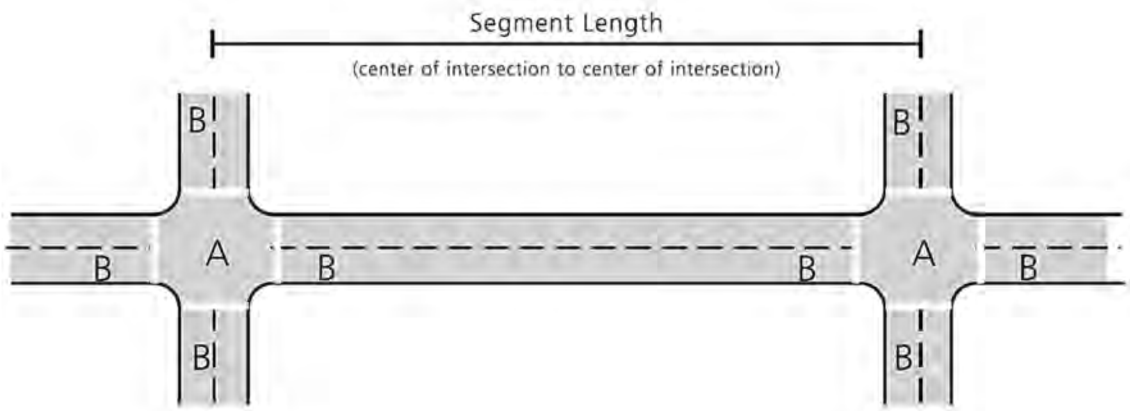


Figure 1. HSM definitions of segments and intersections (AASHTO, 2010)

The basic components of the HSM intersection predictive methods used to estimate the expected average crash frequency are:

- SPFs
- crash severity and collision type distributions
- CMFs
- calibration procedures
- Empirical Bayes (EB) estimation

Combining SPFs, CMFs, and calibration factors without the EB procedure results in a predicted average crash frequency, $N_{predicted}$, based only on the characteristics of the roadway. Estimates of the expected average crash frequency, which incorporate information about the crash history at the site, are available only after applying the EB technique. The basic model structure to predict the average crash frequency at intersections ($N_{predicted\ int}$) is shown in Equation 1. With Equation 1 and additional equations throughout this report, existing notation and definitions in the first edition of the HSM are used as much as possible; however, some modifications to HSM notation and definitions were made for clarity and consistency as necessary.

$$N_{predicted\ int} = N_{spf\ int} \times \left(CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{yi}\right) \times C_i \qquad \text{(Eq. 1)}$$

Where:

$N_{predicted\ int}$ = predicted average crash frequency for an individual intersection for the selected year (crashes/year)
$N_{spf\ int}$ = predicted average crash frequency for an intersection with base conditions (crashes/year)
CMF_{yi} = crash modification factors specific to intersection type *i* and specific geometric design and traffic control features *y*
C_i = calibration factor to adjust the SPF for intersection type *i* to local conditions

Intersection SPFs generally take one of the two forms shown in Equation 2 and Equation 3.

$$N_{spf\ int} = exp\left[a + b \times ln\left(AADT_{maj}\right) + c \times ln(AADT_{min})\right] \quad \text{(Eq. 2)}$$

$$N_{spf\ int} = exp[a + d \times ln(AADT_{total})] \quad \text{(Eq. 3)}$$

Where:

$AADT_{maj}$	=	annual average daily traffic (AADT) on the major road (veh/day)
$AADT_{min}$	=	AADT on the minor road (veh/day)
$AADT_{total}$	=	AADT on the major and minor roads combined (veh/day)
a, b, c, and d	=	estimated regression coefficients

If AADTs on the two major legs of an intersection differ, the larger of the two AADT values is used for $AADT_{maj}$. Similarly, if AADTs on the two minor legs of a four-leg intersection differ, the larger of the two AADT values is used for $AADT_{min}$. As needed, $AADT_{total}$ can be estimated as the sum of $AADT_{maj}$ and $AADT_{min}$. If AADTs are not available for each evaluation year, interpolation and extrapolation are used.

Each SPF also has an associated overdispersion parameter, k. This parameter provides insights into how average crash frequencies at the sites used to estimate the SPF vary compared to the SPF predictions. Larger values indicate larger site-to-site variation around the SPF prediction.

2.1.2 HSM Chapter 10—Predictive Methods for Intersections on Rural Two-Lane, Two-Way Roads

Chapter 10 of the HSM includes SPFs for the following intersection configurations and traffic control types on rural two-lane, two-way roads:

- Three-leg intersections with minor road stop control (3ST)
- Four-leg intersections with minor road stop control (4ST)
- Four-leg intersections with signal control (4SG)

The intersection SPFs for rural two-lane, two-way roads predict an average crash frequency for all crash severities and collision types combined. Default tabular distributions are provided for crash severity and collision type. Crash severities can be disaggregated into five levels: fatal (K), incapacitating injury (A), non-incapacitating injury (B), possible injury (C), and property-damage-only (O or PDO). The collision type distributions are provided for three levels of severity: total—all severities combined (KABCO), FI severities (KABC), and PDO.

2.1.3 HSM Chapter 11—Predictive Methods for Intersections on Rural Multilane Highways

Chapter 11 of the HSM includes SPFs for the following intersection configurations and traffic control types on rural multilane highways:

- Three-leg intersections with minor road stop control on rural, four-lane divided or undivided highways (3ST)
- Four-leg intersections with minor road stop control on rural, four-lane divided or undivided highways (4ST)
- Four-leg intersections with signal control on rural, four-lane divided or undivided highways (4SG)

For all three intersection types, the intersecting minor roads may be two- or four-lane highways. The intersection SPFs for rural multilane highways predict average crash frequency for all crash severities and collision types combined (KABCO), FI crashes (KABC), and FI crashes with possible injuries excluded (KAB). Default tabular distributions are provided for crash severity and collision type for four levels of severity: total – all severities combined (KABCO), FI severities (KABC), FI severities with possible injuries excluded (KAB), and PDO. Alternative SPFs by collision type and severity level are also provided in Appendix 11B of the HSM for stop-controlled intersections on rural multilane highways.

2.1.4 HSM Chapter 12—Predictive Methods for Intersections on Urban and Suburban Arterials

Chapter 12 of the HSM includes SPFs for the following intersection configurations and traffic control types on urban and suburban arterials:

- Three-leg intersections with minor road stop control (3ST)
- Three-leg intersections with signal control (3SG)
- Four-leg intersections with minor road stop control (4ST)
- Four-leg intersections with signal control (4SG)

The HSM intersection predictive methods for urban and suburban arterials separately predict vehicle-pedestrian and vehicle-bicycle crashes. This results in a different model structure for $N_{predicted\ int}$ than shown in Equation 1 to calculate total crashes at an intersection, combining vehicle-only plus vehicle-pedestrian plus vehicle-bicycle crashes. This structure of the SPFs for intersections in urban and suburban areas is illustrated in Equation 4 and Equation 5, again using existing HSM notations and variable definitions as appropriate.

$$N_{predicted\ int} = \left(N_{bi} + N_{pedi} + N_{bikei}\right) \times C_i \quad \text{(Eq. 4)}$$

$$N_{bi} = N_{spf\ int} \times \left(CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{yi}\right) \quad \text{(Eq. 5)}$$

Where:

$N_{predicted\ int}$	=	predicted average crash frequency for an individual intersection for the selected year (crashes/year)
N_{bi}	=	predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle crashes) (crashes/year)
N_{pedi}	=	predicted average crash frequency of vehicle-pedestrian crashes of an intersection (crashes/year)
N_{bikei}	=	predicted average crash frequency of vehicle-bicycle crashes of an intersection (crashes/year)
$N_{spf\ int}$	=	predicted total average crash frequency of intersection-related crashes for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions) (crashes/year)
$CMF_{1i}\ldots CMF_{yi}$	=	crash modification factors specific to intersection type *i* and specific geometric design and traffic control features *y*
C_i	=	calibration factor for intersection type *i* to adjust prediction to local conditions

The SPF portion of N_{bi}, $N_{spf\ int}$, is the sum of two disaggregate predictions by collision type, as shown in Equation 6.

$$N_{spf\ int} = N_{bimv} + N_{bisv} \qquad \text{(Eq. 6)}$$

Where:

N_{bimv}	=	predicted average crash frequency of MV crashes of an intersection for base conditions (crashes/year); and
N_{bisv}	=	predicted average crash frequency of SV crashes of an intersection for base conditions (crashes/year).

SPFs to estimate MV crashes are provided for three severity levels: total (KABCO), FI, and PDO. Because the SPFs for the different severity levels were developed independently, for any given collision type (e.g., MV) and site type (e.g., 3ST), the “total” crash prediction (i.e., all severities) may not equal the sum of the FI and PDO predictions. Preliminary values for FI and PDO crashes obtained directly from the SPFs may be adjusted using an approach illustrated by Equations 7 and 8, provided as an example for MV crashes. The same approach is also used to make these same types of adjustments to the SV crash predictions.

$$N_{bimv(FI)} = N_{bimv(total)} \times \left(\frac{N'_{bimv(FI)}}{N'_{bimv(FI)} + N'_{bimv(PDO)}}\right) \qquad \text{(Eq. 7)}$$

$$N_{bimv(PDO)} = N_{bimv(total)} - N_{bimv(FI)} \qquad \text{(Eq. 8)}$$

Where:

$N_{bimv(FI)}$ = predicted average crash frequency of MV, FI crashes of an intersection for base conditions (crashes/year)

$N_{bimv(total)}$ = predicted average crash frequency of MV crashes (all severities) of an intersection for base conditions (crashes/year)

$N_{bimv(PDO)}$ = predicted average crash frequency of MV, PDO crashes of an intersection for base conditions (crashes/year)

$N'_{bimv(FI)}$ = preliminary value for predicted average crash frequency of MV, FI crashes of an intersection for base conditions (crashes/year)

$N'_{bimv(PDO)}$ = preliminary value for predicted average crash frequency of MV, PDO crashes of an intersection for base conditions (crashes/year)

Separate collision type distributions for MV, FI; MV, PDO; SV, FI; and SV, PDO are provided for each of the four intersection types addressed by the predictive method.

Separate model structures are used to estimate the yearly number of vehicle-pedestrian crashes, N_{pedi}, at stop-controlled and signalized intersections on urban and suburban arterials. The average number of annual vehicle-pedestrian crashes for stop-controlled intersections is estimated with Equation 9.

$$N_{pedi} = N_{bi} \times f_{pedi} \quad \text{(Eq. 9)}$$

Where:

f_{pedi} = pedestrian crash adjustment factor for intersection type *i*

The average number of annual vehicle-pedestrian crashes for signalized intersections is estimated with Equation 10.

$$N_{pedi} = N_{pedbase} \times CMF_{1p} \times CMF_{2p} \times CMF_{3p} \quad \text{(Eq. 10)}$$

Where:

$N_{pedbase}$ = predicted average crash frequency of vehicle-pedestrian crashes for base conditions at signalized intersections (crashes/year)

CMF_{1p} = crash modification factor for number of bus stops within 1,000 ft of the center of the intersection

CMF_{2p} = crash modification factor for presence of one or more schools within 1,000 ft of the center of the intersection

CMF_{3p} = crash modification factor for number of alcohol sales establishments within 1,000 ft of the center of the intersection

The predicted number of vehicle-pedestrian crashes per year for base conditions at signalized intersections, $N_{pedbase}$, is estimated using Equation 11.

$$N_{pedbase} = exp\left(a + b \times ln(AADT_{total}) + c \times ln\left(\frac{AADT_{min}}{AADT_{maj}}\right) + d \times ln(PedVol) + e \times n_{lanesx}\right) \quad \text{(Eq. 11)}$$

Where:

$PedVol$ = sum of daily pedestrian volumes crossing all intersection legs (pedestrians/day), only considering crossing maneuvers immediately adjacent to the intersection (e.g., along a marked crosswalk or the extended path of any approaching sidewalk)

n_{lanesx} = maximum number of traffic lanes crossed by a pedestrian, including through and turning lanes, in any crossing maneuver at the intersection considering the presence of refuge islands (only raised or depressed refuges are considered).

Base conditions associated with $N_{pedbase}$ are absence of bus stops, schools, and alcohol sales establishments near the intersection. All of the vehicle-pedestrian crashes predicted with Equations 10 and 11 are assumed to be FI crashes (none as PDO).

The average number of annual vehicle-bicycle crashes, N_{bikei}, at all intersection types are predicted using the same model structure as that used to predict N_{pedi} at stop-controlled intersections (see Equation 12).

$$N_{bikei} = N_{bi} \times f_{bikei} \quad \text{(Eq. 12)}$$

Where:

f_{bikei} = bicycle crash adjustment factor for intersection type i

All of the vehicle-bicycle crashes predicted with Equation 12 are assumed to be FI crashes (none as PDO).

2.1.5 HSM Chapter 19—Predictive Method for Ramps

Chapter 19 of the HSM includes SPFs for the following types of crossroad ramp terminals at service interchanges:

- Three-leg terminals with diagonal exit ramp (D3ex)
- Three-leg terminals with diagonal entrance ramp (D3en)
- Four-leg terminals with diagonal ramps (D4)
- Four-leg terminals at four-quadrant parclo A (A4)
- Four-leg terminals at four-quadrant parclo B (B4)
- Three-leg terminals at two-quadrant parclo A (A2)
- Three-leg terminals at two-quadrant parclo B (B2)

These ramp terminal types are illustrated in Figure 2. Predictive methods provided in Chapter 19 of the HSM capture both stop control (ST) and signal control (SG) for each of the terminal types and cover crossroad ramp terminals with anywhere from two to six crossroad through lanes (total of both travel directions).

Utilizing new notation introduced as part of the 2014 Supplement to the HSM (Chapter 19), signal-controlled crossroad ramp terminal SPFs generally take the form shown in Equation 13.

$$N_{spf,w,SGn,at,z} = \exp[a + b \times \ln(c \times AADT_{xrd}) + d \times \ln(c \times AADT_{ex} + c \times AADT_{en})] \quad \text{(Eq. 13)}$$

with

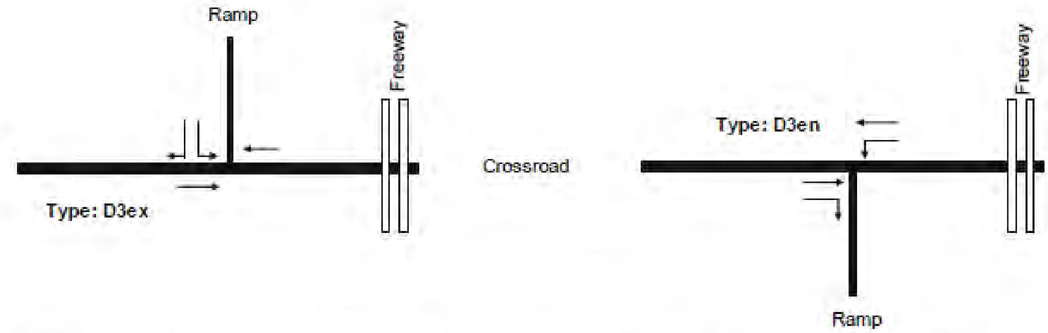
$$AADT_{xrd} = 0.5 \times (AADT_{in} + AADT_{out}) \quad \text{(Eq. 14)}$$

Where:

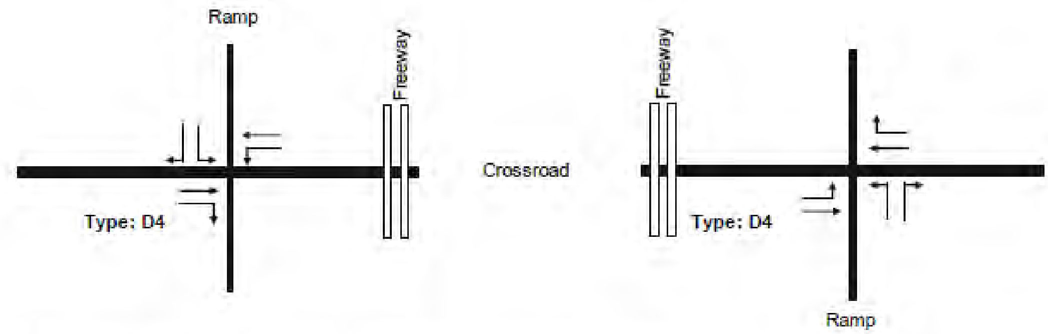
$N_{spf,w,SGn,at,z}$ = predicted average crash frequency of a signal-controlled crossroad ramp terminal of site type *w* (*w* = D3ex, D3en, D4, A4, B4, A2, or B2) with base conditions, *n* crossroad lanes, all collision types (*at*), and severity *z* (*z* = FI, PDO) (crashes/year)
$AADT_{xrd}$ = AADT on the crossroad (veh/day)
$AADT_{in}$ = AADT on the crossroad leg between ramps (veh/day)
$AADT_{out}$ = AADT on the crossroad leg outside of the interchange (veh/day)
$AADT_{ex}$ = AADT on the exit ramp (veh/day)
$AADT_{en}$ = AADT on the entrance ramp (veh/day)

The AADT of the loop exit ramp at a B4 terminal configuration is not included in $AADT_{ex}$, and the AADT of the loop entrance ramp at an A4 configuration is not included in $AADT_{en}$.

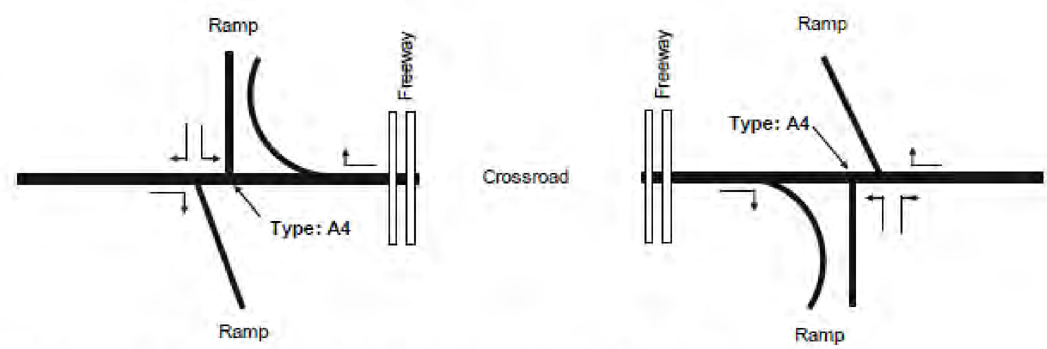
The intersection SPFs predict average crash frequency for all collision types. Default collision type distributions are provided for both FI and PDO crashes. Separate collision type distributions are provided for signalized ramp terminals in rural and urban areas.



a. Three-Leg Ramp Terminal With Diagonal Exit or Entrance Ramp (*D3ex* and *D3en*)

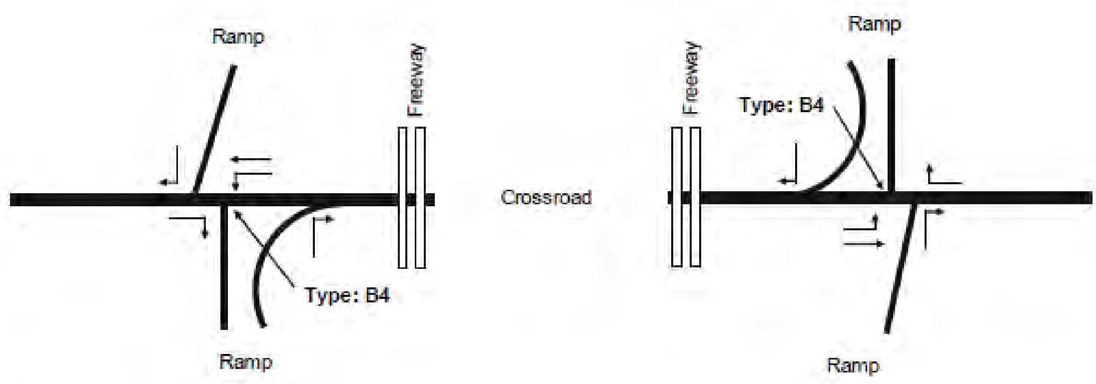


b. Four-Leg Ramp Terminal With Diagonal Ramps (*D4*)

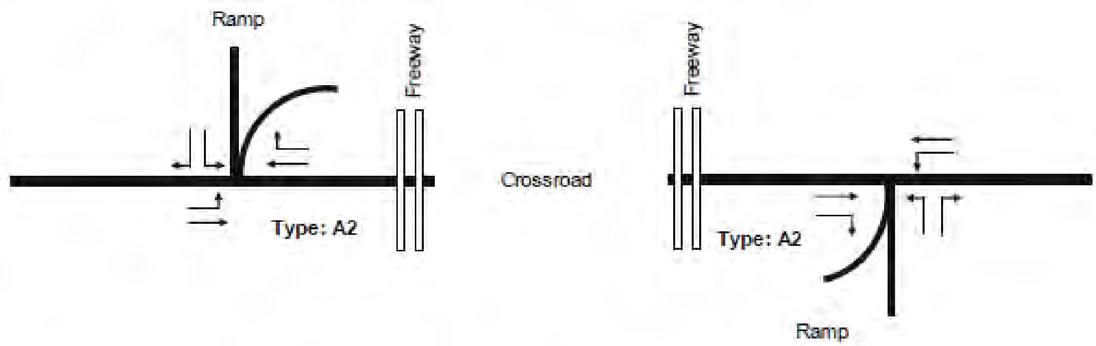


c. Four-Leg Ramp Terminal at Four-Quadrant Parclo A (*A4*)

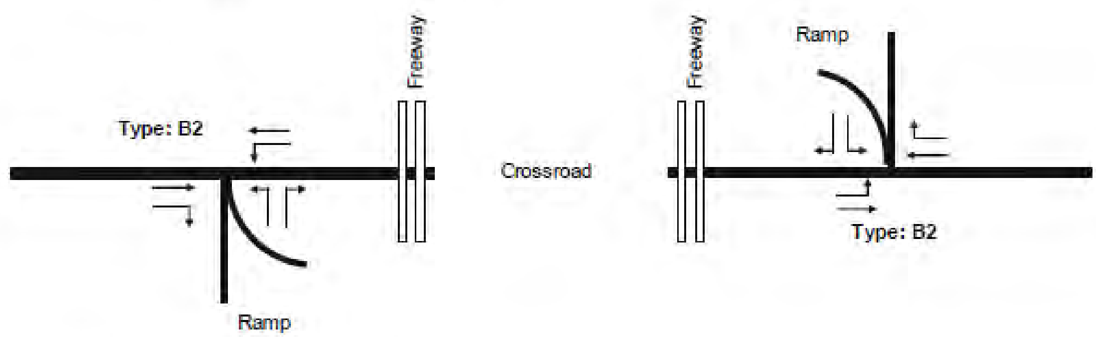
Figure 2. Ramp terminal configurations (AASHTO, 2014)



d. Four-Leg Ramp Terminal at Four-Quadrant Parclo B (*B4*)



e. Three-Leg Ramp Terminal at Two-Quadrant Parclo A (*A2*)



f. Three-Leg Ramp Terminal at Two-Quadrant Parclo B (*B2*)

Figure 2. Ramp terminal configurations (AASHTO, 2014) (continued)

One-way, stop-controlled crossroad ramp terminal SPFs generally take the form shown in Equation 15.

$$N_{spf,w,ST,at,z} = \exp[a + b \times \ln(c \times AADT_{xrd}) + d \times \ln(c \times AADT_{ex} + c \times AADT_{en})] \quad \text{(Eq. 15)}$$

Where:

$N_{spf,w,ST,at,z}$ = predicted average crash frequency of a one-way, stop-controlled crossroad ramp terminal of site type *w* (*w* = D3ex, D3en, D4, A4, B4, A2, B2) with base conditions, all collision types (*at*), and severity *z* (*z* = FI, PDO)

All other terms have been previously defined.

The intersection SPFs predict average crash frequency for all collision types. Default collision type distributions are provided for both FI and PDO crashes. Separate collision type distributions are provided for one-way, stop-controlled ramp terminals in rural and urban areas.

The predictive methods for ramps in Chapter 19 of the 2014 Supplement to the HSM, including crossroad ramp terminals, utilize SDFs to disaggregate the predicted average frequency of FI crashes into an estimate of average crash frequency for the following severity levels: fatal (K), incapacitating injury (A), non-incapacitating injury (B), and possible injury (C). Adopting the SDF notation introduced as part of the 2014 Supplement to the HSM, the SDFs for crossroad ramp terminals are illustrated in Equations 16 through 19.

$$P_{aS,x,at,K} = \frac{\exp(V_{K+A})}{\frac{1.0}{C_{sdf,aS,x}} + \exp(V_{K+A}) + \exp(V_B)} \times P_{K|K+A,aS,x,at} \quad \text{(Eq. 16)}$$

$$P_{aS,x,at,A} = \frac{\exp(V_{K+A})}{\frac{1.0}{C_{sdf,aS,x}} + \exp(V_{K+A}) + \exp(V_B)} \times \left(1 - P_{K|K+A,aS,x,at}\right) \quad \text{(Eq. 17)}$$

$$P_{aS,x,at,B} = \frac{\exp(V_B)}{\frac{1.0}{C_{sdf,aS,x}} + \exp(V_{K+A}) + \exp(V_B)} \quad \text{(Eq. 18)}$$

$$P_{aS,x,at,C} = 1 - (P_K + P_A + P_B) \quad \text{(Eq. 19)}$$

Where:

$P_{aS,x,at,K}$ = probability of a fatal crash (given that a fatal or injury crash occurred) for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)

$P_{aS,x,at,A}$ = probability of an incapacitating injury crash (given that a fatal or injury crash occurred) for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
$P_{aS,x,at,B}$ = probability of a non-incapacitating injury crash (given that a fatal or injury crash occurred) for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
$P_{aS,x,at,C}$ = probability of a possible injury crash (given that a fatal or injury crash occurred) for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
V_j = systematic component of crash severity likelihood for severity *j*
$P_{K|K+A,aS,x,at}$ = probability of a fatal crash given that the crash has a severity of either fatal or incapacitating injury for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
$C_{sdf,aS,x}$ = calibration factor to adjust SDF for local conditions for all ramp terminal sites (*aS*) and control type *x* (*x* = ST: stop control, Sgn: signal control, n-lane crossroad)

The basic model form for the systematic components of crash severity likelihood at crossroad ramp terminals is illustrated by Equation 20.

$$V_j = a + (b \times I_{p,lt}) + (c \times [n_{dw} + n_{ps}]) + (d \times I_{ps}) + (e \times I_{rural}) \qquad \text{(Eq. 20)}$$

Where:

$I_{p,lt}$ = protected left-turn operation indicator variable for crossroad (= 1 if protected operation exists, 0 otherwise)
n_{dw} = number of unsignalized driveways on the crossroad leg outside of the interchange and within 250 ft of the ramp terminal
n_{ps} = number of unsignalized public street approaches to the crossroad leg outside of the interchange and within 250 ft of the ramp terminal
I_{ps} = non-ramp public street leg indicator variable (= 1 if leg is present, 0 otherwise)
a, b, c, d, and *e* = estimated SDF coefficients

Chapter 19 of the HSM does not provide a predictive method for crossroad ramp terminals with all-way stop control, but it does include an interim method that utilizes the one-way stop control models.

2.1.6 HSM Predictive Method Calibration

The intersection predictive methods in the HSM contain calibration factors to adjust predictions of the HSM models, developed with data from selected jurisdictions and for specific time periods, to be applicable to other jurisdictions and time periods. Equation 1, for example, presents the calibration factor in general form as C_i, with *i* representing a specific site type. The notation C_i is generally used to represent predictive model calibration factors when the site type of interest is an intersection.

Calibration factors associated with the intersection predictive methods are calculated as the ratio of total observed crash frequencies for a selected set of sites to total predicted average crash frequencies for the same sites, during the same time period, determined using the applicable predictive method. This ratio is illustrated in Equation 21.

$$C_i = \frac{\sum observed\ crashes}{\sum predicted\ crashes} \quad \text{(Eq. 21)}$$

Calibration is to be performed separately for each specific facility and site type.

The introduction of SDFs in the freeway and interchange chapters of the HSM resulted in the need for an SDF calibration method. The calibration method utilizes comparisons of observed versus predicted severe crashes (i.e., fatal, incapacitating injury, and non-incapacitating injury crashes). First, observed crash data from the calibration sites are used to calculate the observed probability of a severe crash, given that an FI crash has occurred, using Equation 22.

$$P_{o,aS,ac,at,KAB} = \frac{\sum_i^{n_{sites}} \sum_{t=1}^{n_c} (N_{o,w(i),x(i),at,K,t} + N_{o,w(i),x(i),at,A,t} + N_{o,w(i),x(i),at,B,t})}{\sum_i^{n_{sites}} \sum_{t=1}^{n_c} (N_{o,w(i),x(i),at,K,t} + N_{o,w(i),x(i),at,A,t} + N_{o,w(i),x(i),at,B,t} + N_{o,w(i),x(i),at,C,t})} \quad \text{(Eq. 22)}$$

Where:

$P_{o,aS,ac,at,KAB}$ = observed probability of a severe crash (i.e., K, A, or B) for all collision types (*at*), all sites (*aS*), and all control types (*ac*)

$N_{o,w(i),x(i),at,m,t}$ = observed crash frequency for site *i* with site type *w(i)*, year *t*, control type *x(i)*, for all collision types (*at*), and severity *m* (*m* = K, A, B, C)

n_{sites} = number of sites

n_c = number of years in calibration period

Next, the predicted average crash frequency is used to calculate the predicted probability of a severe crash, given that an FI crash has occurred, using Equation 23.

$$P_{p,aS,ac,at,KAB} = \frac{\sum_i^{n_{sites}} \sum_{t=1}^{n_c} (N_{p,w(i),x(i),at,K,t} + N_{p,w(i),x(i),at,A,t} + N_{p,w(i),x(i),at,B,t})}{\sum_i^{n_{sites}} \sum_{t=1}^{n_c} (N_{p,w(i),x(i),at,K,t} + N_{p,w(i),x(i),at,A,t} + N_{p,w(i),x(i),at,B,t} + N_{p,w(i),x(i),at,C,t})} \quad \text{(Eq. 23)}$$

Where:

$P_{p,aS,ac,at,KAB}$ = predicted probability of a severe crash (i.e., K, A, or B) for all collision types (*at*) all sites (*aS*) and all control types (*ac*);

$N_{p,w(i),x(i),at,m,t}$ = predicted crash frequency for site *i* with site type *w(i)*, year *t*, control type *x(i)*, for all collision types (*at*), and severity *m* (*m* = K, A, B, C);

The final step involves computing the calibration factor using Equation 24.

$$C_{sdf,w} = \frac{P_{o,aS,ac,at,KAB}}{1.0 - P_{o,aS,ac,at,KAB}} \times \frac{1.0 - P_{p,aS,ac,at,KAB}}{P_{p,aS,ac,at,KAB}} \quad \text{(Eq. 24)}$$

Calibration is performed separately for each SDF.

2.1.7 Empirical Bayes Estimation

This section describes the EB estimation, as used in Step 15 of the HSM Predictive Method (presented in the Part C Introduction and Applications Guidance) when applicable. EB estimation combines a predicted average crash frequency for a site with an observed crash frequency for that site, resulting in an estimate of the expected average crash frequency. Combining the observed crash frequency for a specific site with the predicted crash frequency for other similar sites increases the accuracy of the expected crash frequency estimate. The EB method reduces regression-to-the-mean (RTM) bias in estimates obtained from observed crash frequency alone, where locations experiencing higher than average crash frequency over some time period will naturally see lower crash frequencies in a following time period and vice versa.

Two or more years of crash data are desirable for applying the EB method. A more straightforward EB estimation can occur when crash data are assigned to each segment or intersection (i.e., a site-specific EB analysis). However, when crash data are not segment- or intersection-specific, the EB method can be applied across multiple, aggregated sites (i.e., project-level EB method). Only the site-specific EB method is discussed here.

Applying the site-specific EB method results in an estimate of expected average crash frequency for a site ($N_{expected}$), produced by combining predicted average crash frequency for the site ($N_{predicted}$) with the observed crash frequency for that same site ($N_{observed}$).

$N_{expected}$ for a site is calculated using Equations 25 and 26.

$$N_{expected} = w \times N_{predicted} + (1.00 - w) \times N_{observed} \quad \text{(Eq. 25)}$$

$$w = \frac{1}{1 + k \times (\sum_{all\ study\ years} N_{predicted})} \quad \text{(Eq. 26)}$$

where:

$N_{expected}$ = expected average crash frequency obtained by combining the predicted average crash frequency ($N_{predicted}$) with the observed crash frequency ($N_{observed}$) using the EB method
$N_{predicted}$ = predicted average crash frequency obtained using the appropriate predictive model
$N_{observed}$ = observed crash frequency
w = weighted adjustment to be placed on the HSM predictive model estimate
k = overdispersion parameter associated with the SPF

2.2 Protocols, Best Practices, and Emerging Approaches for Predictive Model Development

This section discusses formal protocols and guidance for predictive model development, as well as best practices for predictive model development identified from past experience and emerging approaches. This discussion focuses on issues applicable to modeling at-grade intersection safety performance and does not necessarily address issues relevant only to roadway segments or ramps.

2.2.1 Development of SPFs

SPFs for intersections are equations that relate the expected intersection crash frequency (possibly by type and/or severity) for some defined time period to characteristics of the intersection. In the context of the intersection predictive methods in the HSM, SPFs are used to estimate a predicted average intersection crash frequency (in units of crashes/year) for a given combination of major- and minor road traffic volumes. SPFs in the HSM are provided for different facility types, numbers of intersecting legs, and traffic control types. Intersection SPFs can also appear as “fuller models,” which include additional variables beyond only major- and minor road traffic volumes for crash prediction.

A synthesis paper published in *Transportation Research Part A* (Lord and Mannering, 2010) contains descriptions of key issues and methodological approaches applicable to SPF-like models that have been used to understand how various factors influence the frequency of crashes. Data and methodological issues explored in the paper include overdispersion, underdispersion, time-varying explanatory variables, temporal and spatial correlation, low sample mean, small sample size, injury severity and collision type correlation, omitted variable bias, endogenous variables, functional form, and fixed parameters. The wide variety of methods that appear in the published literature to deal with these issues are then identified and described, including Poisson regression, negative binomial regression, Poisson-lognormal models, zero-inflated models, Conway-Maxwell-Poisson model, Gamma model, generalized estimating equation model, generalized additive models, random effects models, negative multinomial models, random-parameters models, bivariate/multivariate models, finite mixture/Markov switching models, duration models, hierarchical/multilevel models, neural and Bayesian network models, and support vector machine models. Finally, maximum likelihood and Bayesian estimation methods are briefly covered. Similar to the synthesis paper by Lord and Mannering

(2010), the Federal Highway Administration (FHWA) *Safety Performance Function Development Guide: Developing Jurisdiction-Specific SPFs* (Srinivasan and Bauer, 2013) provides guidance for states interested in developing their own SPFs and addresses a variety of issues which may be encountered when developing jurisdiction-specific SPFs.

Recent advances in SPF development allow for the estimation of variance and 95% confidence intervals for the estimate of average crash frequency resulting from multiplying SPFs and CMFs. More advanced SPFs deal with temporal correlation, usually through aggregating multiple years of data for a particular set of sites, as well as spatial correlation by using more complex modeling techniques. Model forms other than negative binomial are sometimes used to handle excessive numbers of sites with no crashes (zero-inflated models), to achieve more flexible model forms (Poisson-lognormal), and to deal with both overdispersion and underdispersion (Conway-Maxwell-Poisson models). These model forms are discussed very briefly in the FHWA SPF Guide, but the guide provides additional references for more complex model forms.

The progress made in methodological approaches used to understand how various factors influence the frequency of crashes has been substantial (Lord and Mannering, 2010). The most common method for developing intersection SPFs is to estimate a negative binomial (NB) regression model with number of intersection and intersection-related crashes as the dependent variable and traffic volumes, geometry, traffic control devices, and other factors characterizing an intersection as independent variables. NB regression requires the specification of the "additional dispersion" (additional when compared to a Poisson model) that is common to crash data. It is typical for this "additional dispersion" to be specified as a dispersion parameter multiplied by the expected number of crashes squared (NB-2 model). The dispersion parameter for intersections is usually treated as a fixed parameter.

There are some variations to the standard NB-2 model that can still be integrated with existing HSM predictive methods while at the same time possibly improving the parameter estimates associated with traffic volumes, geometric features, traffic control, and other factors in the models in terms of "accuracy" and "precision." Count regression models with identification variables for counties, districts, regions, or states treated as "fixed effects" (see Le and Porter, 2012) or "random effects" (see Shankar et al., 1998) are modeling alternatives that help to address crash-influencing factors that may be common to groups of intersections, but that are not captured by the models. The methods were successfully employed in NCHRP Project 17-45, *Enhanced Safety Prediction Analysis Tool for Freeways and Interchanges* (Bonneson et al., 2012). Random effects models were also used to estimate multi-state models in developing the HSM Chapter 12 SPFs. The negative multinomial modeling approach can also be used to address correlations in disturbance terms across observations, and is particularly useful when a single intersection appears in a database multiple times due to multiple observation years that are kept disaggregated. NB regression, count regression models with "fixed effects" and "random effects," and negative multinomial modeling were considered most relevant for developing SPFs for this research.

2.2.2 Development of Crash Modification Factors

CMFs are multiplicative factors used to compute the expected number of crashes at a site with a certain set of characteristics. CMFs are multiplied by the expected number of crashes at that site without those characteristics, commonly referred to as "base conditions" in the HSM predictive model context. For any given characteristic or treatment, CMF values greater than one indicate that the characteristic or treatment is expected to increase the number of crashes compared to the base conditions, while values lower than one indicate that the treatment is expected to decrease the number of crashes. FHWA's *A Guide to Developing Quality Crash Modification Factors* (CMF Guide) (Gross et al., 2010) was published with a primary purpose of describing methods behind the development of CMFs and providing guidance for adequate application and reporting of CMFs, depending on the available data and treatments that CMFs need to address. Crash Modification Functions (CMFunctions) may also be used to compute CMFs for a specific site, and while they require more data than computing single CMF values, they allow the CMF values to vary as site characteristics change, which may make them preferable among practitioners.

A variety of study approaches have been used to develop CMFs including EB before-after studies, before-after with comparison group studies, full Bayes studies, surrogate measure studies, cross-sectional studies, case-control studies, cohort studies, meta-analysis studies, and expert-panel studies. For all types of CMF studies, the number of treatment and non-treatment sites and duration of the before and after periods, as well as the size of treatment effect, impact the estimated CMFs and their associated standard errors. Before-after studies are considered the most dependable study design for CMF development, but in both national HSM discussions and the research literature, there is emerging interest in whether quality CMFs can be developed using the estimated parameters of regression models from cross-sectional studies. The current state of the research literature indicates that it is possible to estimate reliable CMFs from cross-sectional studies as long as care is taken in the data collection and modeling approaches used. Desirable characteristics of such studies include:

- Carefully collected databases created using both traditional and non-traditional data sources to confirm and enhance the independent variable measurements
- Logical variable specifications with appropriate boundary conditions
- Repeated, consistent findings across studies and locations
- Implementation of observational study design methods, such as the propensity scores-potential outcomes framework to significantly reduce selection bias

2.2.3 Treatment of Crash Severity in Predictive Methods

The way in which crash severity is addressed within the HSM intersection predictive methods has evolved over time as chapters were developed. In Chapter 10 (Rural Two-Lane, Two-Way Roads), SPFs predict average crash frequency for all crash severities. Using proportional distributions, crash severities can be disaggregated into five levels: fatal (K), incapacitating injury (A), non-incapacitating injury (B), possible injury (C), and property-damage-only (O or PDO). In Chapter 11 (Rural Multilane Highways), the intersection SPFs predict average crash frequency for all crash severities combined (KABCO), FI crashes (KABC), and FI crashes with possible injuries excluded (KAB). No additional levels of disaggregation by severity level are available. In Chapter 12 (Urban and Suburban Arterials), intersection SPFs predict average crash frequency for all crash severities combined (KABCO), FI crashes (KABC), and PDO crashes. No additional levels of disaggregation by severity level are available. The urban and suburban intersection SPFs for different levels of severity were independently estimated. For any given collision type (e.g., MV) and site type (e.g., 3ST), it is likely that a "total" crash prediction (i.e., all severities) will not equal the sum of the FI and PDO predictions. Preliminary values for FI and PDO crashes obtained directly from the SPFs are therefore adjusted to make this summing process work. All vehicle-pedestrian and vehicle-bicycle crashes that are predicted as part of the urban and suburban intersection methods are treated as FI crashes (none as PDO). Finally, in Chapter 19 (Predictive Method for Ramps), intersection SPFs predict average crash frequency for FI crashes (KABC) and PDO crashes at crossroad ramp terminals of freeway interchanges. SDFs are then used to further disaggregate the SPF predicted average frequency of FI crashes into an estimate of average crash frequency for the following severity levels: fatal (K), incapacitating injury (A), non-incapacitating injury (B), and possible injury (C). Two SDFs are provided, one for one-way stop-control terminals and another for signalized terminals. These two SDFs are applicable to other subcategories of terminal types within these broader categories.

The evolution in the way severity is addressed in HSM predictive methods explicitly recognizes that severity distributions may change with traffic volumes, design decisions, traffic control, and other characteristics. A synthesis paper by Savolainen et al. (2011) presents key issues and methodological approaches that have been used to understand how various factors influence the severity outcomes of crashes. While Savolainen et al. state that there has been substantial progress made in modeling crash severities, this progress has been made and tracked primarily by those making more theoretical advancements in methodological areas of crash severity modeling. Only more recently have these methods been incorporated into applied, product-oriented safety research, such as research aimed at developing predictive methods. For example, the research that resulted in the development of HSM Chapter 19 (Predictive Method for Ramps) utilized multinomial logit (MNL) models to develop the SDFs.

Generally speaking, the databases used to estimate severity models and corresponding SDFs consist of the same crashes and intersections as the databases used to estimate the frequency models and corresponding SPFs, but the databases are restructured so that the basic observation unit (i.e., database row) is the crash instead of the intersection. To more seamlessly work together with existing HSM predictive methods, the multinomial logit (e.g., Shankar and Mannering, 1996), nested logit (e.g., Shankar et al., 1996), and the binary logit (e.g., Al-Ghamdi, A., 2002) are the most logical severity modeling alternatives. These three severity modeling alternatives begin with defining a set of linear functions (S_{jr}), shown in general form in Equation 27, that define how injury severity outcome j for crash r is determined.

$$S_{jr} = X_{jr}\beta_j + \epsilon_{jr} \qquad \text{(Eq. 27)}$$

Where:

X_{jr}	=	a row of observed characteristics (e.g., driver, vehicle, roadway, environment) associated with crash *r* that have an impact on injury severity outcome *j*
β_j	=	a vector of parameters to be estimated that quantify how the characteristics in X_{jr} impact injury severity outcome *j*
ε_{jr}	=	a disturbance term that accounts for unobserved and unknown characteristics of crash *r* that impact injury severity outcome *j*

There are as many such linear functions as there are possible injury severity outcomes. The probability of having injury severity outcome *j* for crash *r* is then the probability that the linear function for that severity outcome is larger than the functions for any other severity outcome, i.e.:

$$P_r(j) = P\left(S_{jr} \geq S_{Jr}\right) \forall J \neq J \quad \text{(Eq. 28)}$$

or

$$P_r(j) = P\left(X_{jr}\beta_j + \epsilon_{jr} \geq X_{Jr}\beta_J + \epsilon_{Jr}\right) \forall J \neq J \quad \text{(Eq. 29)}$$

with *J* denoting all possible injury outcomes for crash *r*, and $P_r(j)$ the probability of crash *r* having injury outcome *j*. Obviously, the injury severity outcome of a crash, given a set of observable characteristics, can never be predicted with certainty because the values of the disturbance terms for the different injury outcomes are never known with certainty. The severity models, ultimately leading to the SDFs, are developed by making assumptions about the properties of these disturbance terms.

The multinomial logit model is commonly used to model crash severity in cases with three or more possible injury severity outcomes. In NCHRP Project 17-45, for example, FI outcomes were classified as either fatal and incapacitating injury (KA), non-incapacitating injury (B), or possible injury (C). If the disturbance terms of the linear functions referenced in Equations 27-29 are assumed to be identically and independently distributed as extreme value, the multinomial logit model, shown in Equation 30, results.

$$P_r(j) = \frac{exp(X_{jr}\beta_j)}{\sum_{\forall J} exp(X_{Jr}\beta_J)} \quad \text{(Eq. 30)}$$

For model estimation, one of the injury outcome categories is arbitrarily selected as the “base injury severity outcome,” and all of its corresponding parameters (i.e., β_j*’s*) are represented with zeroes. The remaining β_j*’s* that are estimated represent the values for the β_j*’s* relative to the β_j*’s* for the selected “base injury severity outcome.” This results in the form seen in the HSM Chapter 19 SDFs and provided in more general form in Equation 31.

$$P_r(j) = \frac{exp(X_{jr}\beta_j)}{1+\sum_j exp(X_{jr}\beta_j)} \quad \text{(Eq. 31)}$$

Where *j* in this case represents all possible injury severity outcomes except for the base outcome. In NCHRP Project 17-45, for example, possible injury (C) was selected as the base outcome. $P_r(KA)$ and $P_r(B)$ were predicted using an equation similar to Equation 28; $P_r(C)$ was then predicted as 1 - $P_r(KA)$ - $P_r(B)$.

The MNL model provides a high level of flexibility in terms of model specification, allowing exploration of various possible severity-related relationships such as:

- Certain roadway characteristics and traffic conditions impacting the probability of some severity outcomes relative to the base outcome, but not impacting others
- Certain roadway characteristics and traffic conditions increasing (or decreasing) the probability of the higher severity outcomes relative to the base outcome, while also increasing (or decreasing) the probability of the lower severity outcomes

The MNL assumption that the disturbance terms of the linear functions referenced in Equations 27-29 are identically and independently distributed as extreme value leads to a practical model form for interpretation and prediction. However, the main disadvantage of the MNL model is that it is characterized by the independence of irrelevant alternatives (IIA) assumption, meaning that it does not appropriately handle scenarios with two or more severity outcomes that are close substitutes with shared unobserved effects captured by their disturbance terms (i.e., violations of the IIA assumption).

The nested logit model is a common modeling alternative for discrete outcome data when the IIA assumption is violated. In the case of crash injury severity outcomes, it might be expected that the linear functions, S_{jr}, for possible injury (C) and no injury (O) have some shared unobserved effects captured in the disturbance terms. This results in a nesting structure, similar to the example illustrated in Figure 3.

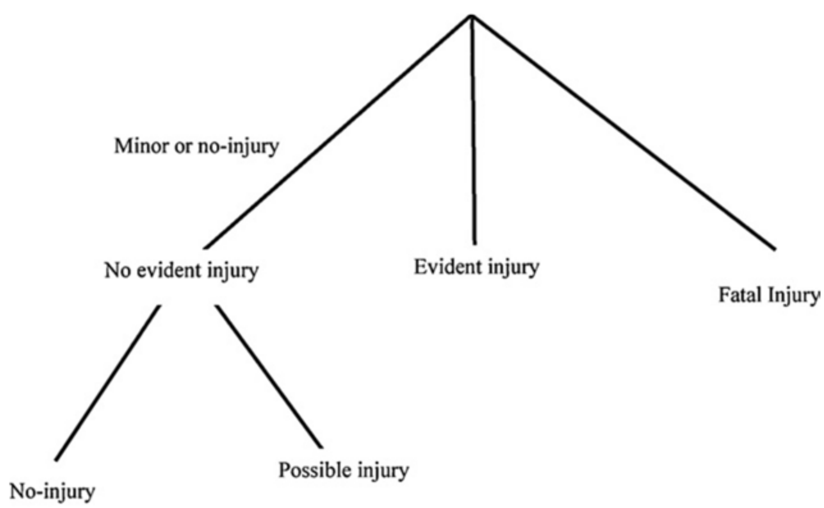


Figure 3. Example of nested structure for crash injury severities (Savolainen et al., 2011)

The nested logit model handles this nesting structure through making the assumption that the disturbance terms of the linear functions referenced in Equations 27-29 are generalized extreme value distributed, resulting in the model structure shown in Equations 32-34 (as outlined in Washington et al., 2010 and Savolainen et al., 2011).

$$P_r(j) = \frac{exp(X_{jr}\beta_j + \phi_j LS_{jr})}{\sum_{\forall J} exp(X_{Jr}\beta_J + \phi_J LS_{Jr})} \quad \text{(Eq. 32)}$$

$$P_r(q|j) = \frac{exp(X_{qr}\beta_{q|j})}{\sum_{\forall Q} exp(X_{Qr}\beta_{Q|j})} \quad \text{(Eq. 33)}$$

$$LS_{jr} = ln\left[\sum_{\forall Q} exp(X_{Qr}\beta_{Q|j})\right] \quad \text{(Eq. 34)}$$

Where:

$P_r(j)$ = probability of crash *r* having injury outcome *j*, where *j* is an outcome in the "top level" of the nest (e.g., fatal injury, evident injury, or no evident injury)
X_{jr}, X_{qr} = rows of observed characteristics (e.g., driver, vehicle, roadway, environment) associated with crash *r* that have an impact on injury severity outcome *j* (across the top level of the nests) and *q* (within the nests), respectively
β_j, $\beta_{q|j}$ = vectors of parameters to be estimated that quantify how the characteristics in X_{jr} and X_{qr} impact injury severity outcome *j* (across the top level of the nests) and *q* (within the nests), respectively
$P_r(q|j)$ = probability of crash *r* having injury outcome *q*, conditioned on the outcome being in category *j* (e.g., probability of either possible injury or no injury conditioned on a no evident injury crash)
LS_{jr} = "log-sum" or "inclusive value" for the nest (i.e., the expected value of the linear functions, such as those in Equation 27, for the outcomes within the nest)
ϕ_j = "log-sum coefficient" to be estimated

Within this framework, the probability of having one of the severity outcomes (*q*) within the nest, $P_r(q)$, is given in Equation 35 (with all terms previously defined).

$$P_r(q) = P_r(j) \times P_r(q|j) \quad \text{(Eq. 35)}$$

For the nested logit to be considered an appropriate modeling alternative for a given context, the "log-sum coefficient" should fall between zero and one. If the parameter equals one, the model reduces to the MNL. A hypothesis test can be set up to determine whether the "log-sum coefficient" is different than one at some level of statistical significance. This approach was used in NCHRP Project 17-45 before selecting the MNL as an appropriate model form for estimating the HSM Chapter 19 SDFs.

In cases where crash severity is assigned one of two possible outcomes [e.g., fatal and incapacitating injury (KA) or not (BCO)], there are only two such linear functions. With the disturbance terms of these two functions identically and independently distributed as extreme value, the binary logit model results, shown in Equation 36.

$$P_r(j) = \frac{exp(X_{jr}\beta_j)}{1+exp(X_{jr}\beta_j)} \quad \text{(Eq. 36)}$$

The outcome *j* in this case represents one of two possible injury severity outcomes. Continuing the example from the previous paragraph, an injury severity outcome can be classified as either fatal and incapacitating injury (KA) or not (BCO). If the BCO outcomes are set to the "base injury severity outcome," the binary logit parameters could be estimated and the severity probabilities predicted using Equations 37 and 38.

$$P_r(KA) = \frac{exp(X_{KAr}\beta_{KA})}{1+exp(X_{KAr}\beta_{KA})} \quad \text{(Eq. 37)}$$

$$P_r(BCO) = 1 - P_r(KA) \quad \text{(Eq. 38)}$$

The binary logit is a key modeling alternative for multiple reasons, including:

- Sample size challenges, particularly those associated with more severe injury outcomes, may limit the SDFs to being only "two-category SDFs" (an example of which is provided in Equations 37 and 38);
- When injury severity outcomes are classified into more than two categories, estimating a series of binary logit models instead of an MNL model can "temper" estimation issues when IIA is violated.

Anytime that more than one severity outcome has to be combined into one severity outcome category for SDF estimation (e.g., combining both "K" and "A" crashes into a "KA" category), the severity outcome proportions can later be disaggregated with a combination of the SDF probability prediction and default severity distributions for those categories.

2.2.4 Treatment of Collision Type in Predictive Methods

The way in which collision type is addressed within the HSM intersection predictive methods is also different from chapter to chapter, but not to the extent of severity differences. In Chapter 10 (Rural Two-Lane, Two-Way Roads), Chapter 11 (Rural Multilane Highways), and Chapter 19 (Ramps), intersection SPFs predict average crash frequency for all collision types combined. Default collision type distributions are then used to disaggregate the crash predictions by collision type. Different default collision type distributions are provided for different crash severities. An appendix to Chapter 11 also provides alternative SPFs by collision type for intersections along rural multilane highways. The HSM notes the advantage of this is that more accurate safety prediction for a specific collision type can be obtained using a model developed specifically for that collision type than from using a model for all collision types multiplied by a collision type proportion. The disadvantages include model availability for only selected collision types and the sum of predictions from a series of collision type models will not necessarily equal the prediction for a model with all those collision types combined. There were no base conditions or CMFs specific to the alternative SPFs. In Chapter 12 (Urban and Suburban Arterials), intersection SPFs and other predictive method components predict average crash frequency for the following collision types: MV (excluding crashes involving a pedestrian or bicyclist); SV (excluding crashes involving a pedestrian or bicyclist); vehicle-pedestrian; and vehicle-bicycle. Handling collision types within HSM predictive methods remains a topic of ongoing research.

2.2.5 Calibration Factors for Predictive Models

The HSM calibration procedures for both predictive methods and SDFs are described in Section 2.1.6 of this report. In 2014, Bahar and Hauer (2014) prepared A *User's Guide to Develop Highway Safety Manual Safety Performance Function Calibration Factors* to expand on calibration-related information provided to an HSM user (currently contained in the HSM's Appendix A of Part C), covering four key aspects of predictive model calibration:

- Why calibration is needed
- How to implement the calibration process
- How to assess the results of calibration
- How to prepare for future calibration updates

2.3 Current Knowledge Related to Intersection Safety

This section summarizes recent literature related to intersection safety, focusing on:

- Predictive models developed for intersections configurations and traffic control types not currently addressed in the HSM;
- Recently developed CMFs for intersections that could potentially be incorporated, or used in conjunction, with intersection predictive models.

For this section, the research team reviewed literature published since 2008.

2.3.1 Predictive Models for Intersection Configurations and Traffic Control Types Not Currently Addressed in the HSM

Several predictive models have been developed for intersection configurations and traffic control types not currently addressed in the HSM, including three-leg intersections with signal control on rural highways and three- and four-leg intersections with all-way stop control on rural highways. Virginia Department of Transportation (DOT) developed SPFs using NB regression for several different intersection types, including three-leg intersections with signal control on rural highways (Garber and Rivera, 2010). SPFs were created for both total and FI severity levels. A total of 183 rural signalized three-leg intersections were used for model development. The models consisted of major- and minor road AADT.

Safety Analyst contains SPFs developed for use in predicting the safety performance of specific site subtypes (Harwood et al., 2009). The SPFs developed for Safety Analyst predict crash frequency as a function of AADT only. NB regression was used to create the SPFs. Data from Minnesota were used to develop the intersection SPFs for Safety Analyst. Within Safety Analyst, SPFs to predict total and FI crash frequencies are available for the following intersection configurations and traffic control types not currently addressed in the HSM:

- Three-leg intersections with all-way stop control on rural highways
  - Software makes use of SPF for four-leg intersections with all-way stop control on rural highways for three-leg intersections with all-way stop control on rural highways
- Three-leg intersections with signal control on rural highways
  - Software makes use of SPF for four-leg intersections with signal control on rural highways for three-leg intersections with signal control on rural highways
- Four-leg intersections with all-way stop control on rural highways
- Three-leg intersections with all-way stop control on urban streets
  - Software makes use of SPF for four-leg intersections with all-way stop control on rural highways for three-leg intersections with all-way stop control on urban streets
- Four-leg intersections with all-way stop control on urban streets
  - Software makes use of SPF for four-leg intersections with all-way stop control on rural highways for four-leg intersections with all-way stop control on urban streets

The Pennsylvania DOT developed SPFs (Donnell et al., 2014) for rural two-lane highway segments and intersections. Of the five types of intersections examined, two intersection types currently are not addressed in the HSM: three-leg intersections with signal control, and four-leg intersections with all-way stop control. The SPFs were created based on a NB distribution. Forty-five three-leg signalized intersections and thirty-three four-leg all-way stop-controlled intersections in Pennsylvania were used for model development. Several independent variables were used in the regression modeling for both three-leg intersections with signal control and four-leg intersections with all-way stop control. The independent variables used in the SPF for three-leg intersections with signal control on rural two-lane highways included major- and minor road AADTs, posted speed limit on the major road, and the presence/absence of marked crosswalks. For the four-leg intersection with all-way stop control SPF, significant predictor variables included major- and minor road AADTs and posted speed limit on the major road. Vehicle speed or posted speed limit is a variable that is not addressed in any fashion in the SPFs in the first edition of the HSM for any roadway segments or intersections. It is accounted for in some manner for ramps.

2.3.2 Recently Developed CMFs for Intersection Predictive Models Not Currently Addressed in the HSM

The following paragraphs summarize recently developed CMFs that are not addressed in the current HSM intersection predictive methodology and are not presented in HSM Part D, but could potentially be considered for use in a future edition of the HSM. The CMFs presented in this section address the following topics and countermeasures:

- Intersection sight distance
- Increase signal change interval
- Change left-turn phasing from permissive to protected-permissive
- Implement protected-permissive phasing with flashing yellow arrow for permissive phase

- Install dynamic signal warning flashers
- Discontinue late night flash operations at signalized intersections
- Construct bypass lanes

In NCHRP Project 17-59, researchers developed CMFs for intersection sight distance at unsignalized intersections (i.e., intersections with minor road stop control). CMFs are available for total target crashes (i.e., crashes involving a vehicle on the mainline and a vehicle on the minor road) and several subsets of target crashes including: injury crashes, right-angle crashes, angle crashes, left-turn crashes, and daytime crashes. CMFs vary by major route volume, minor route volume, and available intersection sight distance.

Srinivasan et al. (2011) developed several CMFs based on increasing the yellow interval, all red interval, and both, as well as comparing the change interval to Institute of Transportation Engineers (ITE) recommendations. CMFs were developed for total and FI severity levels and angle and rear-end crashes at urban signalized intersections. The CMFs were developed based on a before-after EB analysis. The CMFs that were found statistically significant at the 0.05 confidence level were for: increase the all red interval (total crashes); increase the change interval while still below ITE recommendation (total crashes and FI crashes); and increase the change interval while greater than ITE recommendation (rear-end crashes).

Srinivasan et al. (2011) conducted a before-after EB analysis that produced CMFs for changing the left-turn phasing from permissive to protected-permissive at urban signalized intersections. There are separate CMFs for treating a single approach and treating more than one approach, as well as CMFs for total crashes, FI crashes, left-turn-opposing-through crashes, and rear-end crashes. Srinivasan et al. (2011) conducted a before-after EB and comparison group analysis to create CMFs for using a flashing yellow arrow for the permissive phase on a protected-permissive left turn at urban signalized intersections. CMFs were broken down into several categories based on permissive and protected phasing, and each category has a CMF for total crashes and left-turn crashes. Simpson and Troy (2015) also studied the safety effectiveness of replacing protected only, protected-permissive, and permissive-only left turns with flashing yellow arrows using an EB before-after methodology to develop several CMFs.

Srinivasan et al. (2011) developed CMFs for installing dynamic signal warning flashers using data from 30 signalized intersections and cross-sectional models. CMFs were developed for total crashes, rear-end crashes, angle crashes, FI crashes, and heavy-vehicle crashes. The CMFs are applicable to urban and rural sites as well as three-leg and four-leg intersections.

Lan and Srinivasan (2013) used naïve before-after, EB before-after, and Full Bayesian (FB) before-after (univariate Poisson-gamma as well as multivariate Poisson-log normal models) analysis approaches to establish CMFs for removing late night flash operations at rural and urban signalized intersections. Signalized intersections in North Carolina were examined: 61 sites with the treatment and 395 reference sites. CMFs were developed for total nighttime crashes, nighttime FI crashes, and nighttime frontal impact crashes.

Shams and Dissanayake (2014) quantified the safety benefits of constructing bypass lanes at rural unsignalized intersections. A case-control study was used to estimate the safety effectiveness of bypass lanes at both three- and four-leg unsignalized intersections. There were

382 treated sites (302 four-leg, 80 three-leg intersections) and 367 untreated sites (319 four-leg, 48 three-leg intersections) included in the study. CMFs were estimated for total crashes (i.e., all collision types and severity levels).

2.4 Survey of Current Practice and Crash Prediction Needs

The research team conducted a survey of state and local highway agencies and HSM users to inquire about their use and experiences with the HSM. The survey was intended to:

- Better understand users' experiences with the first edition of the HSM
- Identify and prioritize intersection configuration needs for future editions of the HSM
- Identify and prioritize intersection design elements and traffic control features considered by engineers and planners during the project development process that should be addressed in the new models
- Identify potential agencies (and contacts) to work with towards the completion of the research objectives

The survey was conducted online using SurveyMonkey, a web-based survey package. Responses from 24 state highway agencies, eight county/city highway agencies, and one FHWA representative were received. The survey results are summarized below. Responses to categorical questions are summarized by showing both the percentage of the responses and the frequency/number of responses shown in parentheses. For questions inquiring about priorities, as appropriate, the responses are sorted in priority order.

Survey Results: HSM Intersection Survey

- *What type of agency do you work for?*

State highway agency	73.5%	(25)
County agency	11.8%	(4)
Township agency	0%	(0)
City or other municipal agency	11.8%	(4)
Design consultant	0%	(0)
Federal agency	2.9%	(1)

- *What is your role within your agency?*

Safety engineer	52.9%	(18)
Traffic engineer	29.4%	(10)
Design engineer	5.9%	(2)
Planner	2.9%	(1)
Other	8.8%	(3)

- *Has your agency incorporated the HSM into your policies, practices, and procedures?*

Yes	63.6%	(21)
No	36.4%	(12)

- *How frequently does your agency use HSM procedures?*

Very often	11.8%	(4)
Regularly	26.5%	(9)
Occasionally	38.2%	(13)
Rarely	23.5%	(8)
Never	0%	(0)

- *Please indicate which parts of the HSM your agency uses frequently (select all that apply).*

Part A—Introduction, Human Factors, and Fundamentals of Safety	21.2%	(7)
Part B—Roadway Safety Management Process	42.4%	(14)
Part C—Predictive Methods	60.6%	(20)
Part D—Crash Modification Factors	78.8%	(26)
We have never used the HSM	3.0%	(1)

- *Has your agency implemented HSM Part C procedures (crash prediction methods) as part of the planning and/or design of at-grade intersection projects?*

Yes	38.2%	(13)
No	61.8%	(21)

- *For which types of at-grade intersections has your agency used the HSM Part C procedures (check all that apply)?*

Intersections along rural two-lane highways	70.0%	(14)
Intersections along rural multilane highways	40.0%	(8)
Intersections along urban and suburban arterials	65.0%	(13)

- *What improvements or enhancements would be desirable for the HSM at-grade intersection procedures (please describe)?*
    - Responses included adding specific SPFs for different types of control, SPFs for urban and suburban arterial intersections with six and eight lanes. One respondent suggested adding a method to estimate AADT on minor road approaches when precise AADT is not available for the minor road approaches.

- *What tools has your agency used to implement the HSM Part C procedures for intersections (check all that apply)?*

Interactive Highway Safety Design Model (IHSDM)	29.2%	(7)
Spreadsheet-based tools	75.0%	(18)
Tools developed by your agency	45.8%	(11)
Other (please describe below)	16.7%	(4)

- *Vision Zero Suite* and *ISATe* were two tools mentioned by respondents that answered “Other” for this question.

- *Has your agency developed its own SPFs for use with HSM Part C?*

Yes	26.5%	(9)
We plan to develop our own SPFs	17.6%	(6)
No	55.9%	(19)

- *If you answered "Yes" or "We plan to" to the previous question, what stage of the model development are you currently at for the following facility types?*

	Plan to start development sometime in the future	Plan to begin development within one month	Have begun development	Have completed initial development	Model development is complete
Intersections along rural two-lane highways	42.9% (6)	0% (0)	21.4% (3)	7.1% (1)	28.6% (4)
Intersections along rural multilane highways	46.2% (6)	0% (0)	23.1% (3)	7.7% (1)	23.1% (3)
Intersections along urban and suburban arterials	26.7% (4)	0% (0)	20.0% (3)	33.3% (5)	20.0% (3)

- *The following intersection types are not included in the current HSM Part C procedures. Please rate the priority for inclusion of each intersection type in HSM updates (1= lowest priority; 5 = highest priority). (NOTE: Roundabouts will not be addressed in this project, but will be addressed in NCHRP Project 17-70.)*

	Lowest Priority				Highest Priority	Weighted Average
	1	2	3	4	5	
Intersections with all-way stop control	6.2% (2)	15.6% (5)	21.9% (7)	25.0% (8)	31.2% (10)	3.59
Three-leg signalized intersections on rural two-lane highways	9.4% (3)	12.5% (4)	25.0% (8)	25.0% (8)	28.1% (9)	3.50
Intersections on high-speed expressways	22.6% (7)	3.2% (1)	19.4% (6)	12.9% (4)	41.9% (13)	3.48
Three-leg signalized intersections on rural multilane highways	15.6% (5)	18.8% (6)	21.9% (7)	25.0% (8)	18.8% (6)	3.13
Three-leg intersections where the through movement makes a turning maneuver at the intersection	6.3% (2)	25.0% (8)	34.4% (11)	28.1% (9)	6.2% (2)	3.03
Three-leg intersections with a commercial driveway forming the fourth leg	9.4% (3)	15.6% (5)	50.0% (16)	15.6% (5)	9.4% (3)	3.00
Five-leg intersections	24.2% (8)	21.2% (7)	24.2% (8)	21.2% (7)	9.1% (3)	2.70
Single-point diamond ramp terminal at crossroad	25.8% (8)	19.4% (6)	25.8% (8)	25.8% (8)	3.2% (1)	2.61
Diverging-diamond ramp terminals at crossroad	19.4% (6)	25.8% (8)	35.5% (11)	12.9% (4)	6.4% (2)	2.61
Intersections with yield control on minor approaches	31.2% (10)	37.5% (12)	21.9% (7)	9.4% (3)	0.0% (0)	2.09
Six-or-more-leg intersections	45.2% (14)	22.6% (7)	22.6% (7)	3.2% (1)	6.4% (2)	2.03
Intersections with no control (typically very low traffic volumes)	60.6% (20)	18.2% (6)	12.1% (4)	6.1% (2)	3.0% (1)	1.73

- *Are there any other intersection configurations not listed in the previous question that you believe should receive high priority for inclusion in the HSM?*
  - U-turn intersections, also referred to as J-turn intersections, were mentioned the most by survey respondents. Continuous flow intersections and continuous green "T" intersections were also mentioned frequently. Roundabouts and one-way crossing two-way intersections were also suggested by respondents; however, these intersection types are being addressed in other NCHRP projects.
- *Current HSM procedures include the effects on safety of number of intersection legs, intersection traffic control, major- and minor road AADTs, intersection skew angle, left- and right-turn lanes, left-turn phasing, right-turn-on-red, lighting, and red light cameras. What additional intersection design elements should desirably have safety effects included in HSM predictive methods? (Please check all that apply.)*

Intersection sight distance	71.9%	(23)
Number of approach lanes	62.5%	(20)
Horizontal alignment on approaches	59.4%	(19)
Offset left-turn lane	59.4%	(19)
Median width	50.0%	(16)
Vertical alignment on approaches	50.0%	(16)
Sight distance to traffic control device	43.8%	(14)
Median type	43.8%	(14)
Curb return radius	25.0%	(8)
Transverse rumble strips	21.9%	(7)
Approach width	21.9%	(7)
Bulbout	21.9%	(7)
Raised intersection	3.1%	(1)

  - Additional design elements suggested by survey respondents included lane width, presence of dual turn lanes (left and/or right), and right-turn channelization.
- *Which intersection traffic control features should desirably have safety effects included in HSM predictive methods?*
  - Typical responses included advance warning flashers, attributes of signals (flashing yellow arrow, presence of coordinated/adaptive signal, signal head size, right-turn overlap and retroreflective signal backplates), and pedestrian countdown signals.
- *For development of the next edition of the HSM, please rate the priority that should be assigned to each of the following (1 = lowest priority, 5 = highest priority).*

	Lowest Priority				Highest Priority	Weighted Average
	1	**2**	**3**	**4**	**5**	
Adding new at-grade intersection types to the HSM Part C procedures	6.3% (2)	15.6% (5)	18.8% (6)	31.2% (10)	28.1% (9)	3.59
Improving current HSM Part C procedures for existing intersection types (e.g., replacing crash severity distribution tables with crash SDFs)	6.1% (2)	18.2% (6)	24.2% (8)	24.2% (8)	27.3% (9)	3.48
Developing pedestrian crash prediction procedures for additional intersection types	9.1% (3)	12.1% (4)	33.3% (11)	21.2% (7)	24.2% (8)	3.39
Developing new bicycle crash prediction procedures	15.2% (5)	24.2% (8)	15.2% (5)	21.2% (7)	24.2% (8)	3.15

- *What types of data does your agency have available for intersections that might be useful for development of intersection crash prediction models? (Please check all that apply.)*

Intersection characteristics (intersection inventory data)	54.6%	(18)
AADT data for major road legs of intersections	93.9%	(31)
AADT data for minor road legs of intersections	69.7%	(23)
Pedestrian volumes at intersections	3.0%	(1)
Bicycle volumes at intersections	3.0%	(1)
Crash data for intersections	97.0%	(32)

- *For which of the following intersection types does your agency have sufficient locations for development of intersection crash prediction models (25 or more intersections are desirable)? (Please check all that apply.)*

Intersections with all-way stop control	70.4%	(19)
Intersections on high-speed expressways	48.2%	(13)
Three-leg intersections with a commercial driveway forming the fourth leg	37.0%	(10)
Intersections with no control (typically very low traffic volumes)	29.6%	(8)
Three-leg intersections where the through movement makes a turning maneuver at the intersection	18.5%	(5)
Single-point diamond ramp terminal at crossroad	18.5%	(5)
Intersections with yield control on minor approaches	14.8%	(4)
Five-leg intersections	14.8%	(4)
Diverging-diamond ramp terminals at crossroad	7.4%	(2)
Six-or-more-leg intersections	0.0%	(0)

- *Would your agency be willing to supply data to the NCHRP Project 17-68 research team for developing intersection crash prediction models?*

Yes	21.9%	(7)
Possibly (after further discussion)	75.0%	(24)
No	3.1%	(1)

2.5 Summary of Current Knowledge Related to Intersection Crash Prediction Modeling

The existing HSM crash predictive methods all have a common structure to estimate the expected average crash frequency, which include:

- SPFs;
- Crash severity and collision type distributions;
- CMFs;
- Calibration procedures;
- EB estimation.

However, the sophistication of the predictive methods has evolved as the chapters have been developed. In particular, the science of crash severity prediction has evolved. Specifically, Chapter 10 uses SPFs for all crash severities combined, with tabulated severity distributions used to disaggregate the crash frequency predictions by individual crash severity levels. Chapters 11 and 12 use separate SPFs by crash severity level, while Chapter 19 uses a single SPF for all FI crash severity levels combined and a separate SPF for PDO crashes and then uses crash SDFs to disaggregate the overall crash frequency by severity level.

SPFs for intersections are equations that relate the expected intersection crash frequency (possibly by type and/or severity) for some defined time period to characteristics of the intersection. In the context of the HSM intersection predictive methods, SPFs are used to estimate a predicted average intersection crash frequency for a given combination of major- and minor road traffic volumes. The progress made in methodological approaches that have been used to develop SPFs and to understand how various factors influence the frequency of crashes has been substantial since the publication of the first edition of the HSM.

Chapter 3.
Development of Models for Use in HSM Crash Prediction Methods: Intersections with All-Way Stop Control

This section of the report describes the development of crash prediction models for all-way stop-controlled intersections and presents the final models recommended for incorporation in the second edition of the HSM. None of the HSM Part C chapters in the first edition of the HSM include crash prediction models for all-way stop-controlled intersections. Crash prediction models are recommended for the following intersection types for the second edition of the HSM:

- Four-leg intersections with all-way stop control (4aST) on rural two-lane roads
- Three-leg intersections with all-way stop control (3aST) in urban and suburban areas
- Four-leg intersections with all-way stop control (4aST) in urban and suburban areas

Section 3.1 describes the site selection and data collection process for developing the crash prediction models for all-way stop-controlled intersections. Section 3.2 presents descriptive statistics of the databases used for model development. Section 3.3 presents the statistical analysis and SPFs developed for all-way stop-controlled intersections. Section 3.4 presents the CMFs recommended for use with the SPFs. Section 3.5 presents the results of an analysis to develop SDFs for use with the total SPF for all-way stop-controlled intersections, and Section 3.6 summarizes the recommendations for incorporating new crash prediction models for intersections with all-way stop in the second edition of the HSM.

3.1 Site Selection and Data Collection

A list of potential intersections for model development was initially created using Highway Safety Information System (HSIS) or Safety Analyst databases from five states:

- California (CA)
- Illinois (IL)
- Minnesota (MN)
- Nevada (NV)
- Ohio (OH)

Each intersection in the list was initially screened using Google Earth® to determine if the site was suitable for inclusion in model development. Several reasons a site could be deemed inappropriate for use in model development were:

- The traffic control at the intersection was something other than all-way stop control.
- A private driveway was located in close proximity to the intersection.
- One or more of the approaches to the intersection was a private/commercial access.
- Google Street View® was not available to identify leg specific attributes.
- One or more of the intersection legs was a one-way street.

Each intersection that was initially deemed appropriate for inclusion in model development was given a unique identification code and included in a refined database for detailed data collection.

Three types of data were collected for each intersection during detailed data collection: site characteristic, crash, and traffic volume data. Google Earth® was used to collect detailed site characteristics of the intersections. To reduce potential errors during data collection and to streamline data entry, a data collection tool was created using Visual Basic for Applications. Figure 4 shows a screenshot of the data collection tool used to collect detailed site characteristic data for all-way stop-controlled intersections.

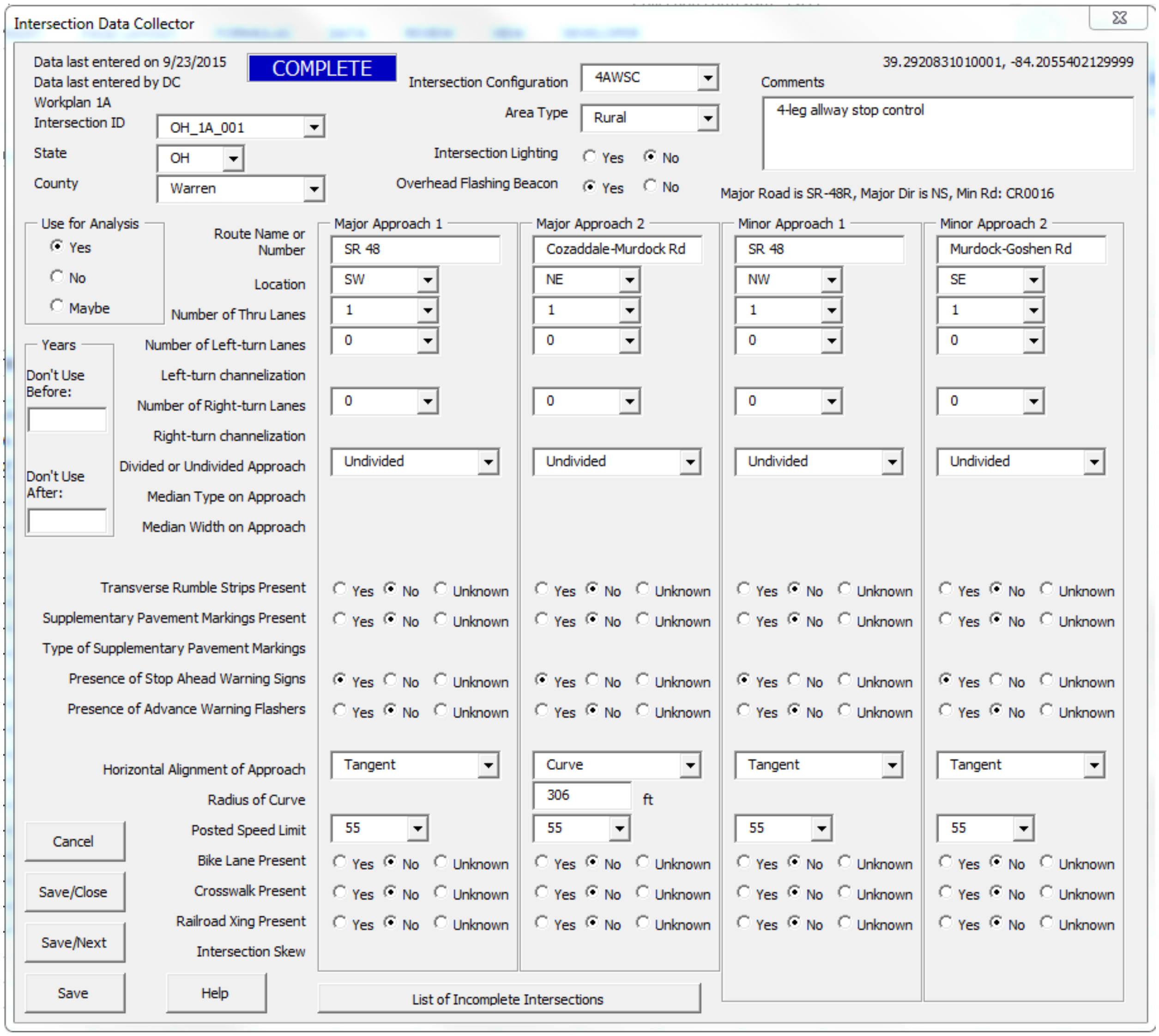


Figure 4. Data collection tool

The items in the data collection tool dynamically changed based on intersection type and presence of features. For example, if the data collector input that the intersection was a three-leg intersection, then the form dynamically changed to only include attributes for three legs. As an additional example, if the data collector selected "no median present" for an approach, then median related attributes would dynamically disappear from the form. Table 3 lists the intersection attributes collected, their definitions, and permitted values for all-way stop-controlled intersections. Once all necessary data were entered into the data collection tool and saved for a given intersection, the data collection tool was used to validate the inputs for that particular intersection consistent with the range and/or permitted values for the respective variables/parameters.

Table 3. Site characteristic variables collected for all-way stop-controlled intersections

Variable/Parameter	Definition	Range or Permitted Values
General Intersection Attributes		
Intersection configuration (i.e., number of legs and type of traffic control)	Indicates the number of legs and type of traffic control	3aST, 4aST
Area type (urban/rural)	Indicates whether the intersection is in a rural or urban area	Rural, urban
Presence of flashing beacons	Indicates if overhead flashing beacons are present at the intersection proper	Yes, no
Presence of intersection lighting	Indicates if overhead lighting is present at the intersection proper	Yes, no
Approach Specific Attributes		
Route name or number	Specify the route name or number of the approach	
Location at intersection	Side/quadrant of the intersection the approach is located	N, S, E, W, NE, NW, SE, SW
Number of through lanes	This includes dedicated through lanes and any lanes with shared movements. On the minor approach of a 3-leg intersection, if there is only one lane, then it should be classified as a through lane	0, 1, 2, 3
Presence/number of left-turn lanes	The number of lanes in which only a left-turn movement can be made	0, 1, 2, 3
Left-turn channelization	Type of left-turn channelization used on the intersection approach	Raised or depressed island, painted, none
Presence/number of right-turn lanes	The number of lanes in which only a right-turn movement can be made	0, 1, 2, 3
Right-turn channelization	Type of right-turn channelization used on the intersection approach	Raised or depressed island, painted, none
Median width	Measured from outside of outer most through lane of approaching lanes to outside of lane in opposing direction	Values in feet
Median type	Type of median separating opposing directions of travel	Raised, depressed, flush, barrier, two-way left-turn lane (TWLTL)
Presence of transverse rumble strips	Indicates the presence of transverse rumble strips on the intersection approach	Yes, no, unknown
Presence/type of supplementary pavement markings	Indicates the presence of supplementary pavement markings on the intersection approach	Yes, no, unknown
Presence of stop ahead warning signs	Indicates the presence of Stop Ahead warning signs on the intersection approach	Yes, no, unknown
Presence of advance warning flashers	Indicates the presence of advance warning flashers on the intersection approach	Yes, no, unknown
Horizontal alignment of intersection approach	Indicates whether the approaching roadway, within 250 ft of the intersection, is a tangent or curved section of roadway	Tangent, curve
Horizontal curve radius	Indicates the radius of the curve on the intersection approach if a curve is present within 250 ft of the intersection	2,000-ft Maximum Range: 45-1960 ft
Posted speed limit	Posted speed limit on the intersection approach	15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, unknown
Presence of crosswalk	Indicates the presence of a crosswalk perpendicular to the intersection approach	Yes, no, unknown
Presence of bike lane	Indicates the presence of a marked bike lane parallel to the intersection approach	Yes, no, unknown
Presence of railroad crossing	Indicates the presence of a railroad crossing on the intersection approach within 250 ft of the intersection	Yes, no, unknown

During detailed data collection, to the extent possible, the research team reviewed historical aerial images to determine if a site had recently been reconstructed or improved to determine which years of data should be used in model development.

Crash and traffic volume data were obtained for California and Minnesota using HSIS databases. For Illinois, Ohio, and Nevada, crash and traffic volume data were obtained from Safety Analyst databases. The goal was to obtain the most recent four to six years of crash and traffic volume data for each site for model development. All of the data (i.e., site characteristics, crash, and traffic volume) were assembled into one database for the purposes of model development.

3.2 Descriptive Statistics of Database

Data for 405 sites—12 rural three-leg, 199 rural four-leg, 33 urban three-leg, and 161 urban four-leg intersections—were available for development of crash prediction models for all-way stop-controlled intersections. The data collection sites were located in five states—California, Illinois, Minnesota, Nevada, and Ohio. To remain consistent with the standards for development of the intersection predictive models in the first edition of the HSM, the goal of this research was to develop crash prediction models with a minimum of 200 site-years of data, and preferably 450 site-years of data or more.

3.2.1 Traffic Volumes and Site Characteristics

Traffic volume and crash data were available for varying periods but were typically collected over a five- or six-year period. Table 4 shows the breakdown of all sites by area type and intersection type. Study period (date range), number of sites and site-years, and basic traffic volume statistics are shown by state in each category and across all states within a category.

Of the intersection characteristics collected in Google Earth® (see Table 3), many showed no or very little variability across sites within a category (i.e., most intersections were predominantly of one type for a specific variable) and were thus excluded from modeling. The remaining variables (percent of "Yes" by area type indicated in parentheses) of potential interest in modeling were:

- presence of intersection lighting (rural: 78%; urban: 93%)
- presence of a flashing beacon (rural: 44%; urban: 28%)
- presence of left-turn lanes on major road (rural: 12%; urban: 21%)
- presence of right-turn lanes on major road (rural: 32%; urban: 19%)
- presence of supplementary pavement marking on major road (rural: 20%; urban: 16%)
- presence of supplementary pavement marking on minor road (rural: 19%; urban: 14%)

The use of some of these site characteristics is discussed later in the SPF model development section (Section 3.3).

3.2.2 Crashes

Of the 405 intersections included in the database, 70 (17%) experienced no crashes over the entire study period; their breakdown by area type and intersection type is as follows:

- Rural three-leg intersections: 5 out of 12
- Rural four-leg intersections: 39 out of 199
- Urban three-leg intersections: 3 out of 33
- Urban four-leg intersections: 23 out of 161

Intersection crashes were defined as those crashes that occurred within 250 ft of the intersection and were classified as “at intersection” or “intersection-related”, consistent with recommended practice in the HSM for assigning crashes to an intersection.

Table 5 (rural intersections) and Table 6 (urban intersections) show all crashes combined, SV, and MV crash counts over the study period for each state within an intersection type. Crash counts by total, FI, and PDO severity levels are shown for all times of day and separately for nighttime. SV crashes at rural intersections include crashes with a bicycle or a pedestrian, while these two collision types are shown separately for urban intersections. This approach is consistent with Chapters 10, 11, and 12 in the first edition of the HSM.

Crash counts are also tallied by collision type and manner of collision across all states, separately for each intersection type, in Table 7 (rural intersections) and Table 8 (urban intersections).

Table 4. Major- and minor road AADTs and total AADT statistics by area type at all-way stop-controlled intersections

				Major Road AADT (veh/day)				Minor Road AADT (veh/day)				$AADT_{total}$ (veh/day)			
State	**Date Range**	**Number of Sites**	**Number of Site-Years**	**Min**	**Max**	**Mean**	**Median**	**Min**	**Max**	**Mean**	**Median**	**Min**	**Max**	**Mean**	**Median**
RURAL THREE-LEG INTERSECTIONS															
IL	2008-2012	8	40	400	5,200	2,049	1,725	50	6,500	1,956	750	450	11,700	4,005	2,550
OH	2009-2013	4	20	560	6,250	2,921	2,438	650	4,384	2,061	1,605	1,402	10,634	4,982	3,947
All states	**2008-2013**	**12**	**60**	**400**	**6,250**	**2,340**	**1,725**	**50**	**6,500**	**1,991**	**796**	**425**	**11,700**	**4,331**	**2,550**
RURAL FOUR-LEG INTERSECTIONS															
CA	2006-2011	29	174	1,696	12,983	5,946	5,300	684	9,985	2,686	2,100	2,628	21,427	8,632	7,667
IL	2008-2012	87	435	421	9,100	2,518	2,300	275	3,750	1,517	1,450	696	11,850	4,034	4,100
MN	2007-2011	17	85	716	8,233	4,378	4,980	614	7,059	3,259	2,980	1,330	15,292	7,637	8,270
OH	2009-2013	66	327	798	8,214	3,062	2,690	130	5,680	1,628	1,475	1,262	13,538	4,690	4,283
All states	**2006-2013**	**199**	**1,021**	**421**	**12,983**	**3,357**	**2,799**	**130**	**9,985**	**1,873**	**1,638**	**696**	**21,427**	**5,230**	**4,506**
URBAN THREE-LEG INTERSECTIONS															
CA	2006-2011	4	24	3,725	12,000	7,433	7,004	501	4,872	2,469	2,251	3,801	15,300	8,682	7,534
IL	2008-2012	17	85	175	15,000	4,535	3,360	300	11,000	2,895	2,000	475	16,000	7,429	5,841
OH	2009-2013	7	35	2,450	6,821	4,534	4,470	914	6,456	4,534	5,124	3,384	24,705	11,277	9,487
All states	**2006-2013**	**28**	**144**	**175**	**15,000**	**4,948**	**4,185**	**300**	**11,000**	**3,244**	**2,433**	**475**	**24,705**	**8,901**	**8,400**
URBAN FOUR-LEG INTERSECTIONS															
CA	2006-2011	13	78	2,730	11,792	7,300	7,660	400	8,250	4,959	5,200	3,818	18,207	12,260	12,515
IL	2008-2012	60	300	1,150	10,900	4,820	4,100	438	8,500	2,822	2,650	1,950	17,400	7,643	7,000
MN	2007-2011	28	138	1,283	10,896	5,823	5,966	417	6,700	3,129	3,008	2,124	17,127	8,953	8,428
NV	2007-2011	26	130	2,156	12,955	9,731	10,230	770	11,982	6,493	6,872	2,926	24,770	16,224	16,991
OH	2009-2013	34	170	1,789	10,232	4,913	4,659	504	4,871	2,171	2,032	2,701	13,684	7,084	6,969
All states	**2006-2013**	**161**	**816**	**1,150**	**12,955**	**6,008**	**5,400**	**400**	**11,982**	**3,503**	**3,000**	**1,950**	**24,770**	**9,511**	**8,108**

Table 5. All crashes combined and single- and MV crash counts by intersection type and crash severity—rural all-way stop-controlled intersections

State	Date Range	Number of Sites	Number of Site-Years	Time of Day	All Crashes Combined			SV Crashes[a]			MV Crashes		
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO
RURAL THREE-LEG INTERSECTIONS													
IL	2008-2012	8	40	All	34	9	25	8	4	4	26	5	21
				Night	11	4	7	6	3	3	5	1	4
OH	2009-2013	4	20	All	9	5	4	3	2	1	6	3	3
				Night	5	3	2	3	2	1	2	1	1
All states	**2008-2013**	**12**	**60**	**All**	**43**	**14**	**29**	**11**	**6**	**5**	**32**	**8**	**24**
				Night	**16**	**7**	**9**	**9**	**5**	**4**	**7**	**2**	**5**
RURAL FOUR-LEG INTERSECTIONS													
CA	2006-2011	29	174	All	252	77	175	26	4	22	226	73	153
				Night	49	12	37	7	0	7	42	12	30
IL	2008-2012	87	435	All	405	99	306	42	14	28	363	85	278
				Night	86	22	64	12	5	7	74	17	57
MN	2007-2011	17	85	All	55	22	33	12	6	6	43	16	27
				Night	13	7	6	5	1	4	8	6	2
OH	2009-2013	66	327	All	279	75	204	48	13	35	231	62	169
				Night	83	19	64	26	7	19	57	12	45
All states	**2006-2013**	**199**	**1,021**	**All**	**991**	**273**	**718**	**128**	**37**	**91**	**863**	**236**	**627**
				Night	**231**	**60**	**171**	**50**	**13**	**37**	**181**	**47**	**134**

[a] Total and FI SV crashes include pedestrian and bicycle crashes.

Table 6. All crashes combined, single- and MV, and pedestrian and bicycle crash counts by intersection type and crash severity—urban all-way stop-controlled intersections

State	Date Range	Number of Sites	Number of Site Years	Time of Day	All Crashes Combined			SV Crashes			Multiple-Vehicle Crashes			Pedestrian Crashes	Bicycle Crashes
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO	FI	FI
URBAN THREE-LEG INTERSECTIONS															
CA	2006-2011	5	30	All	18	9	9	10	6	4	8	3	5	0	0
				Night	7	4	3	4	2	2	3	2	1	0	0
IL	2008-2012	17	85	All	83	21	62	8	2	6	71	15	56	3	1
				Night	26	3	23	5	0	5	20	2	18	1	0
OH	2009-2013	11	55	All	78	16	62	22	5	17	55	10	45	0	1
				Night	24	4	20	12	3	9	12	1	11	0	0
All States	**2006-2013**	**33**	**170**	**All**	**179**	**46**	**133**	**40**	**13**	**27**	**134**	**28**	**106**	**3**	**2**
				Night	**57**	**11**	**46**	**21**	**5**	**16**	**35**	**5**	**30**	**1**	**0**
URBAN FOUR-LEG INTERSECTIONS															
CA	2006-2011	13	78	All	89	34	55	14	3	11	73	29	44	2	0
				Night	16	5	11	7	1	6	9	4	5	0	0
IL	2008-2012	60	300	All	608	132	476	20	2	18	566	108	458	14	8
				Night	127	33	94	5	0	5	118	29	89	4	0
MN	2007-2011	28	138	All	115	36	79	14	4	10	97	28	69	1	3
				Night	22	5	17	7	2	5	15	3	12	0	0
NV	2007-2011	26	130	All	180	67	113	28	9	19	152	58	94	0	0
				Night	67	30	37	16	6	10	51	24	27	0	0
OH	2009-2013	34	170	All	140	24	116	14	3	11	124	19	105	0	2
				Night	36	7	29	4	1	3	32	6	26	0	0
All States	**2006-2013**	**161**	**816**	**All**	**1132**	**293**	**839**	**90**	**21**	**69**	**1012**	**242**	**770**	**17**	**13**
				Night	**268**	**80**	**188**	**39**	**10**	**29**	**225**	**66**	**159**	**4**	**0**

Table 7. Crash counts by collision type and manner of collision and crash severity at rural all-way stop-controlled intersections

	Rural Three-Leg Intersections			Rural Four-Leg Intersections		
	Total	**FI**	**PDO**	**Total**	**FI**	**PDO**
SINGLE-VEHICLE CRASHES						
Collision with animal	0	0	0	5	2	3
Collision with bicycle	0	0	0	4	4	0
Collision with pedestrian	0	0	0	1	1	0
Overturned	0	0	0	6	5	1
Other SV collision	11	6	5	112	25	87
Total SV crashes	**11**	**6**	**5**	**128**	**37**	**91**
MULTIPLE-VEHICLE CRASHES						
Angle collision	11	3	8	453	136	317
Head-on collision	0	0	0	14	4	10
Rear-end collision	18	5	13	289	81	208
Sideswipe collision	2	0	2	61	7	54
Other MV collision	1	0	1	46	8	38
Total MV crashes	**32**	**8**	**24**	**863**	**236**	**627**
Total Crashes	**43**	**14**	**29**	**991**	**273**	**718**

Table 8. Crash counts by collision type and manner of collision and crash severity at urban all-way stop-controlled intersections

Collision Type	Urban Three-Leg Intersections			Urban Four-Leg Intersections		
	Total	**FI**	**PDO**	**Total**	**FI**	**PDO**
SINGLE-VEHICLE CRASHES						
Collision with animal	0	0	0	0	0	0
Collision with bicycle	2	2	0	13	13	0
Collision with pedestrian	3	3	0	17	17	0
Overturned	2	2	0	2	1	1
Other SV collision	38	11	27	88	20	68
Total SV crashes	**45**	**18**	**27**	**120**	**51**	**69**
MULTIPLE-VEHICLE CRASHES						
Angle collision	39	7	32	547	143	404
Head-on collision	1	1	0	6	3	3
Rear-end collision	69	18	51	316	62	254
Sideswipe collision	9	1	8	43	4	39
Other MV collision	16	1	15	104	30	70
Total MV crashes	**134**	**28**	**106**	**1012**	**242**	**770**
Total Crashes	**179**	**46**	**133**	**1132**	**293**	**839**

3.3 Safety Performance Functions—Model Development

Intersection SPFs were developed using either Equation 2 or Equation 3:

$$N_{spf\ int} = exp[a + b \times ln(AADT_{maj}) + c \times ln(AADT_{min})] \quad \text{(Eq. 2)}$$

$$N_{spf\ int} = exp[a + d \times ln(AADT_{total})] \quad \text{(Eq. 3)}$$

Where:

$N_{spf\ int}$	=	predicted average crash frequency for an intersection with base conditions (crashes/year)
$AADT_{maj}$	=	AADT on the major road (veh/day)
$AADT_{min}$	=	AADT on the minor road (veh/day)
$AADT_{total}$	=	AADT on the major and minor roads combined (veh/day)
a, b, c, and d	=	estimated regression coefficients

Based on a review of the number of states, sites, site-years, and crashes for the database assembled, data for all sites were used for model development to maximize the sample size rather than using a portion of the data for model development and a portion for model validation. All SPFs were developed using a NB regression model based on all sites combined within a given area type and intersection type. In all models, state was included as a random blocking effect, with sites nested within their respective state. A significance level of 0.20 for inclusion in a model was selected for an individual parameter. This was based on previous models included in the first edition of the HSM (Harwood et al., 2007); however, as presented below, all parameters in the final models for all-way stop-controlled intersections were significant at the 0.10 level. PROC GLIMMIX of SAS 9.3 was used for all modeling (SAS, 2013).

Intersection characteristics varied widely among the sites in the database as mentioned earlier. For example, only a very small number of intersections satisfied the conditions of no lighting, and no left- and right-turn lanes on the major road. Initially, an attempt was made to perform a cross-sectional analysis including characteristics such as the presence of left- and right-turn lanes and others (e.g., presence of flashing beacon, supplementary pavement marking on major or minor road) in the model as binary variables in the hope of estimating a corresponding CMF. However, except for lighting for some intersection types, none of the site characteristics was statistically significant. Therefore, model development progressed using only the absence of lighting at an intersection as the base condition, consistent with the base condition for intersection lighting in Chapters 10 and 12 in the HSM. None of the other roadway characteristics were considered in the modeling.

In the database, the distributions of intersections with and without lighting were as follows:

- Rural three-leg intersections: 8 lighted; 4 unlighted (33% unlighted)
- Rural four-leg intersections: 157 lighted; 42 unlighted (21% unlighted)
- Rural three- and four-leg intersections: 165 lighted; 46 unlighted (22% unlighted)
- Urban three-leg intersections: 28 lighted; 5 unlighted (15% unlighted)
- Urban four-leg intersections: 152 lighted; 9 unlighted (5.6% unlighted)
- Urban three- and four-leg intersections: 180 lighted; 14 unlighted (7.2% unlighted)

Based on these distributions, the following final modeling approach was chosen:

- Rural three-leg intersections: Because of the small number of intersections, SPFs were developed using both lighted and unlighted intersections combined; total crashes at lighted intersections were adjusted in reverse using the CMF for lighting based on the work by Elvik and Vaa (2004) and shown in Equation 10-24 in Chapter 10 in the HSM (i.e., divide rather than multiply the crashes by the CMF), and the proportion of total crashes for unlighted intersections that occurred at night in the current database; and
- All other intersections: SPFs were developed using lighted intersections only and adjusting in reverse for total crashes at lighted intersections using the CMF for lighting based on the work by Elvik and Vaa (2004) and shown in Equation 10-24 in Chapter 10 and Equation 12-36 in Chapter 12 in the HSM (i.e., divide rather than multiply the crashes by the CMF).

For consistency with Chapters 10 and 12 in the HSM, an attempt was made to develop SPFs for the following crash severity levels and collision types:

- Rural three- and four-leg intersections: total crashes, including pedestrian and bicycle crashes (similar to Equations 10-8 and 10-9 in the HSM)
- Urban three- and four-leg intersections: total, FI, and PDO crashes (excluding pedestrian and bicycle crashes), separately for single- and MV crashes (similar to Equations 12-21 and 12-24 in the HSM)

SPFs for vehicle-pedestrian and vehicle-bicycle crashes at urban intersections could not be developed as pedestrian and bicycle volumes were not available.

All potential models outlined above were estimated. Several models of the form shown in Equation 2 (using major- and minor road AADTs) did not converge. In those cases, SPFs of the form shown in Equation 3 (using $AADT_{total}$ which is the sum of $AADT_{maj}$ and $AADT_{min}$) were estimated.

Developing SPFs at urban intersections separately for single- and MV crashes produced no usable models (i.e., either the model did not converge or the coefficient of AADT was counterintuitive); therefore, single- and MV crashes were modeled together.

The final SPFs for total crashes (i.e., all severity levels combined) at rural all-way stop-controlled intersections are shown in Table 9; only models with $AADT_{total}$ were found to be significant. The table shows the model coefficients and overdispersion parameter (estimate), their standard error, and associated p-values (or significance level) for each intersection type. Figure 5 (three-leg) and Figure 6 (four-leg) graphically present the SPFs shown in Table 9 for various major- and minor approach AADT levels.

Similar to Tables 10-5 and 10-6 in the HSM, Tables 10 and 11 provide percentages for crash severity levels and collision types and manner of collision, respectively, for rural all-way stop-controlled intersections. These percentages were calculated based on all crash counts at all intersections—lighted and unlighted—in all states combined.

Table 9. SPF coefficients for intersections with all-way stop control on rural two-lane highways

Intersection Type	Parameter	Estimate	Standard Error	Pr > F	Significance Level
		TOTAL CRASHES[a]			
Three-Leg	Intercept	-9.05	3.28	--	--
	$\ln(AADT_{total})$	1.06	0.40	0.03	Significant at 95% level
	Overdispersion	0.94	0.58	--	--
Four-Leg	Intercept	-9.67	1.09	--	--
	$\ln(AADT_{total})$	1.12	0.12	<.01	Significant at 99% level
	Overdispersion	0.39	0.07	--	--

[a] Includes SV, MV, pedestrian, and bicycle crashes.
Base condition: absence of lighting.

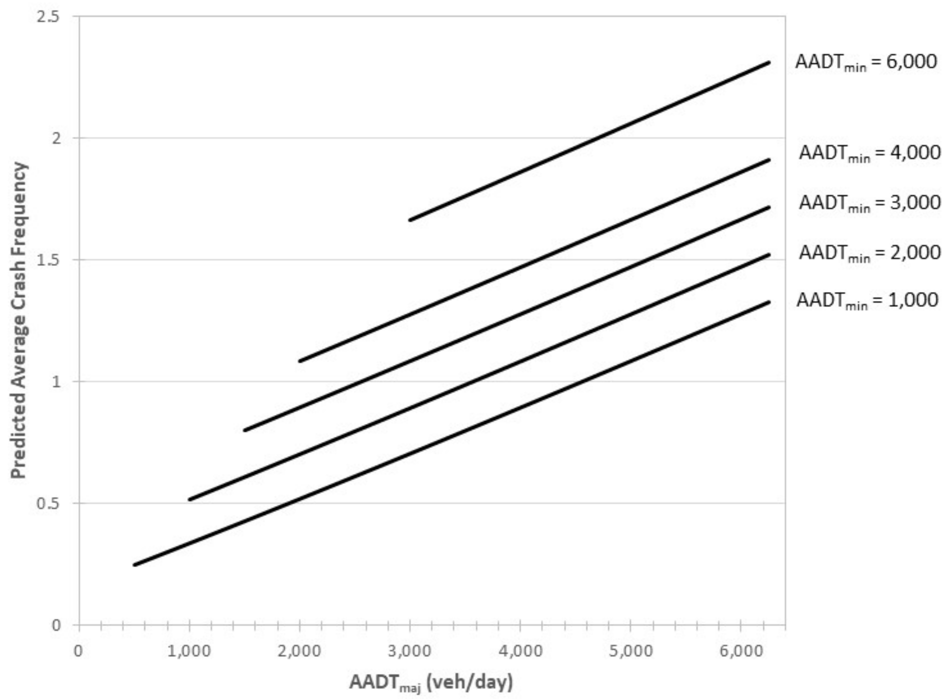


Figure 5. Graphical representation of the SPF for total crashes at rural three-leg, all-way stop-controlled intersections

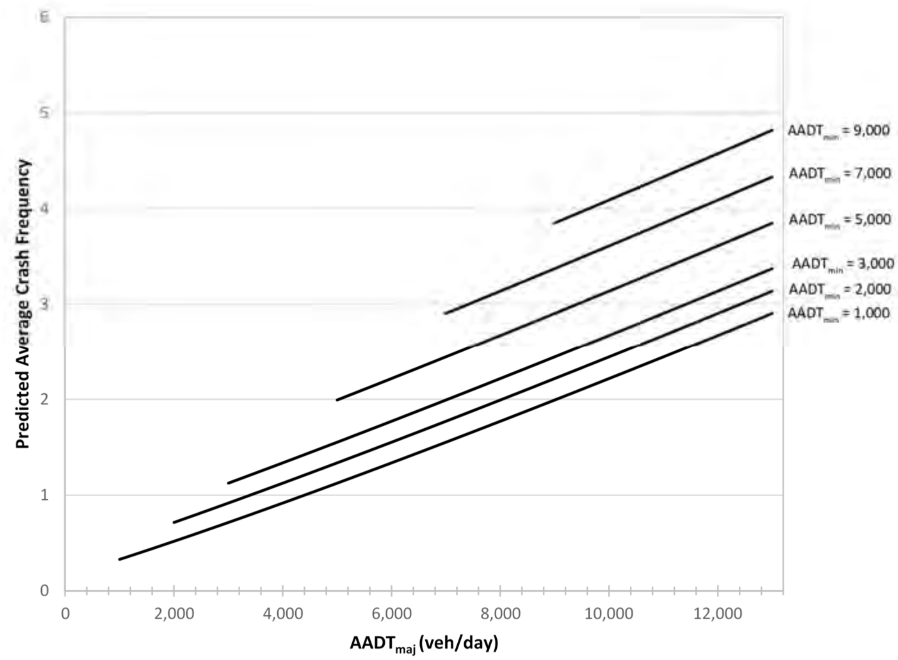


Figure 6. Graphical representation of the SPF for total crashes at rural four-leg, all-way stop-controlled intersections

Table 10. Distributions for crash severity level at rural all-way stop-controlled intersections

Crash Severity Level	Percentage of Total Crashes	
	Rural Three-Leg All-Way Stop-Controlled Intersections	**Rural Four-Leg All-Way Stop-Controlled Intersections**
Fatal	0.0	0.3
Incapacitating injury	4.7	3.6
Non-incapacitating injury	14.0	11.2
Possible injury	14.0	12.4
Total fatal plus injury	32.6	27.5
Property-damage-only	67.4	72.5
Total	**100.0**	**100.0**

Table 11. Distributions for collision type and manner of collision and crash severity at rural all-way stop-controlled intersections

Collision Type	Percentage of Total Crashes by Collision Type					
	Rural Three-Leg All-Way Stop-Controlled Intersections			Rural Four-Leg All-Way Stop-Controlled Intersections		
	Total	FI	PDO	Total	FI	PDO
Single-Vehicle Crashes						
Collision with animal	0.0	0.0	0.0	0.5	0.7	0.4
Collision with bicycle	0.0	0.0	0.0	0.4	1.5	0.0
Collision with pedestrian	0.0	0.0	0.0	0.1	0.4	0.0
Overturned	0.0	0.0	0.0	0.6	1.8	0.1
Other SV collision	25.6	42.9	17.2	11.3	9.2	12.1
Total SV crashes	**25.6**	**42.9**	**17.2**	**12.9**	**13.6**	**12.7**
Multiple-Vehicle Crashes						
Angle collision	25.6	21.4	27.6	45.7	49.8	44.2
Head-on collision	0.0	0.0	0.0	1.4	1.5	1.4
Rear-end collision	41.9	35.7	44.8	29.2	29.7	29.0
Sideswipe collision	4.7	0.0	6.9	6.2	2.6	7.5
Other MV collision	2.3	0.0	3.4	4.6	2.9	5.3
Total MV crashes	74.4	57.1	82.8	87.1	86.4	87.3
Total Crashes	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**

Table 12 shows the coefficients and associated statistics of the final SPFs for urban all-way stop-controlled intersections. Usable models were developed for FI and PDO severity levels, but none for total severity (i.e., all severity levels combined). For these intersection types, crashes for total severity can be estimated by summing predicted FI and PDO severity crashes. Figures 7-10 graphically present the SPFs shown in Table 12 for various major- and minor approach AADTs.

Table 12. SPF coefficients for intersections with all-way stop control on urban and suburban arterials

Intersection Typea	Parameter	Estimate	Standard Error	Pr > F	Significance Level
FI Crashes[a]					
Three-Leg Intersections	Intercept	-8.19	2.78	--	--
	$\ln(AADT_{total})$	0.77	0.31	0.02	Significant at 95% level
	Overdispersion	0.07	0.20	--	--
Four-Leg Intersections	Intercept	-11.62	1.88	--	--
	ln(AADTmaj)	0.92	0.24	<.01	Significant at 99% level
	ln(AADTmin)	0.32	0.17	0.06	Significant at 90% level
	Overdispersion	0.66	0.14	--	--
PDO Crashes					
Three-Leg Intersections	Intercept	-7.94	2.40	--	--
	$\ln(AADT_{total})$	0.85	0.26	<.01	Significant at 99% level
	Overdispersion	0.37	0.19	--	--
Four-Leg Intersections	Intercept	-8.58	1.58	--	--
	ln(AADTmaj)	0.64	0.20	<.01	Significant at 99% level
	ln(AADTmin)	0.36	0.15	0.01	Significant at 99% level
	Overdispersion	0.78	0.12	--	--

[a] Includes single-and MV crashes only (i.e., pedestrian and bicycle crashes are excluded).
Base Condition: Absence of lighting.

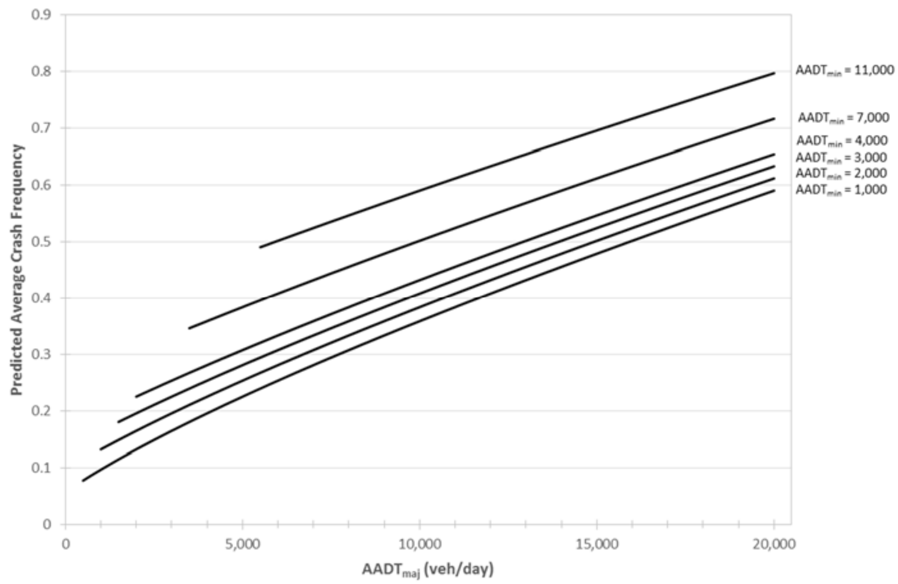


Figure 7. Graphical representation of the SPF for FI crashes at urban and suburban three-leg, all-way stop-controlled intersections

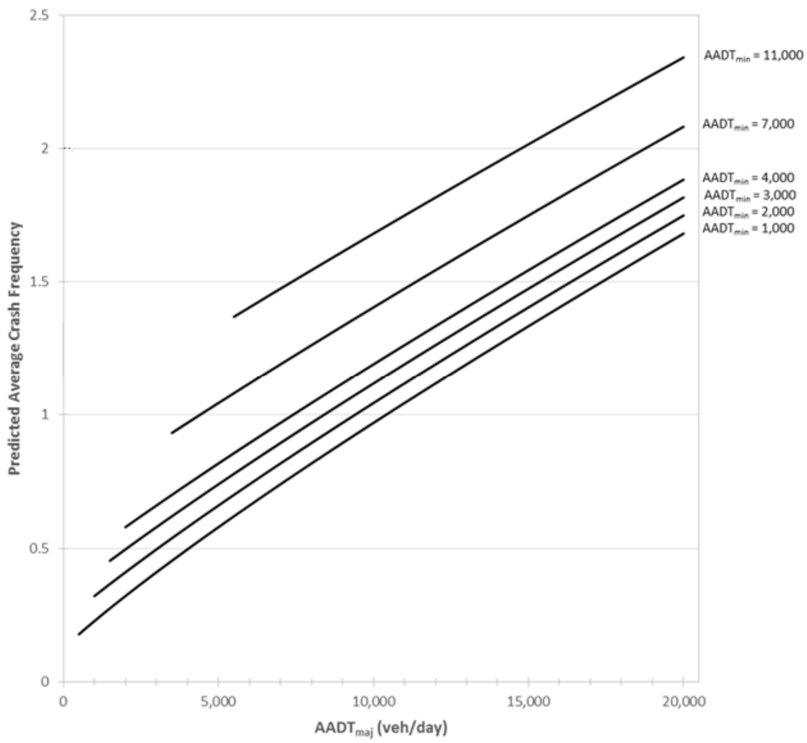


Figure 8. Graphical representation of the SPF for PDO crashes at urban and suburban three-leg, all-way stop-controlled intersections

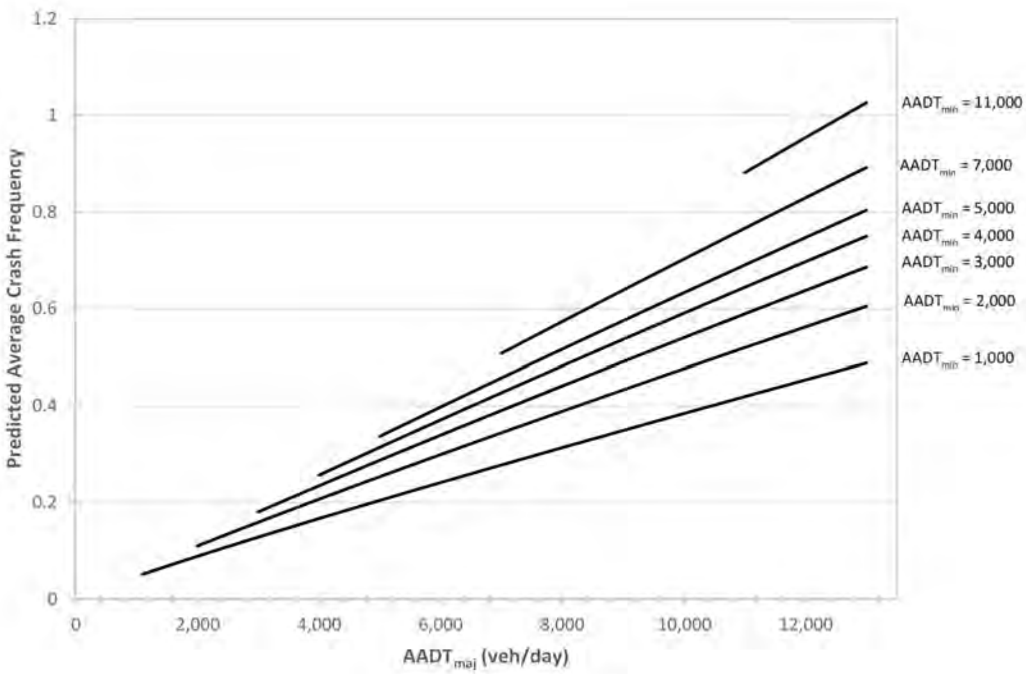


Figure 9. Graphical representation of the SPF for FI crashes at urban and suburban four-leg, all-way stop-controlled intersections

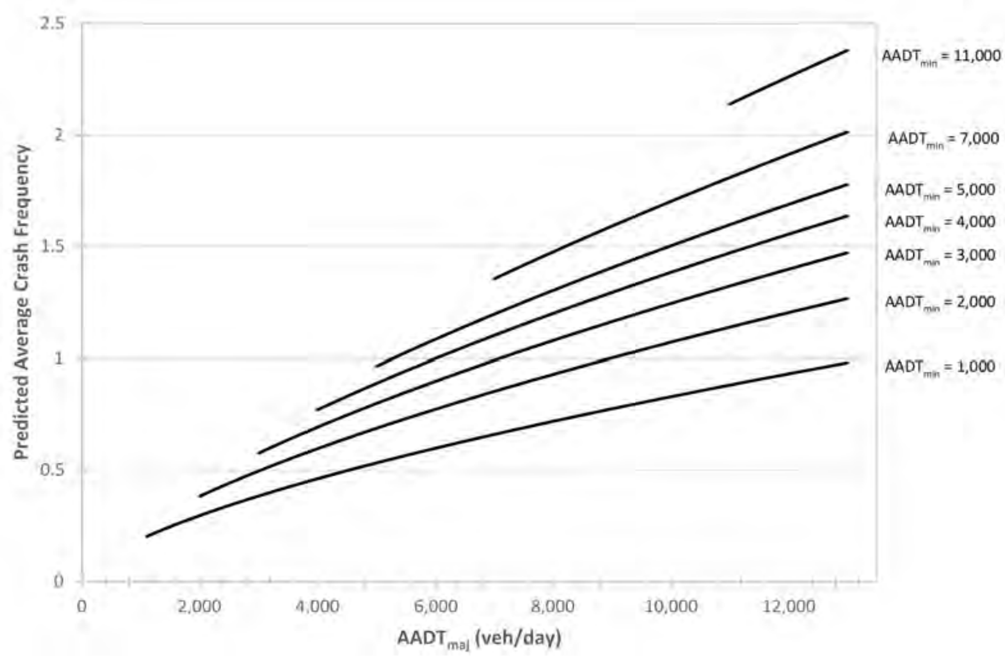


Figure 10. Graphical representation of the SPF for PDO crashes at urban and suburban four-leg, all-way stop-controlled intersections

Table 13 (similar to Table 11 for rural intersections) provides percentages of total crashes by collision type and severity level for urban all-way stop-controlled intersections. These percentages were calculated based on all crash counts at all intersections—lighted and unlighted—in all states combined.

Table 13. Distributions for collision type and manner of collision and crash severity at urban and suburban all-way stop-controlled intersections

Collision Type	Percentage of Total Crashes by Collision Type					
	Urban Three-Leg All-Way Stop-Controlled Intersections			Urban Four-Leg All-Way Stop-Controlled Intersections		
	Total	FI	PDO	Total	FI	PDO
SINGLE-VEHICLE CRASHES						
Collision with animal	0.0	0.0	0.0	0.0	0.0	0.0
Overturned	1.1	4.8	0.0	0.2	0.4	0.1
Other SV collision	21.76	26.2	20.3	8.0	7.8	8.1
Total SV crashes	**22.9**	**31.0**	**20.3**	**8.2**	**8.2**	**8.2**
MULTIPLE-VEHICLE CRASHES						
Angle collision	22.3	16.7	24.1	49.4	53.6	48.1
Head-on collision	0.6	2.4	0.0	0.5	1.1	0.4
Rear-end collision	39.4	42.9	38.3	28.5	23.3	30.2
Sideswipe collision	5.1	2.4	6.0	3.9	1.5	4.6
Other MV collision	9.7	4.8	11.3	9.4	12.4	8.5
Total MV crashes	**77.1**	**69.0**	**79.7**	**91.8**	**91.8**	**91.8**
Total Crashes	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**

For urban intersections, the predicted average crash frequency excludes vehicle-pedestrian and vehicle-bicycle crashes. To calculate a predicted average crash frequency of an intersection that includes vehicle-pedestrian and vehicle-bicycle crashes, the predictive model is given by

$$\boldsymbol{N_{predicted\ int}} = \left(\boldsymbol{N_{bi}} + \boldsymbol{N_{pedi}} + \boldsymbol{N_{bikei}}\right) \times \boldsymbol{C_i} \qquad \text{(Eq. 4)}$$

Where:

$N_{predicted\ int}$ = predicted average crash frequency for an individual intersection for the selected year (crashes/year)
N_{bi} = predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle crashes) (crashes/year)
N_{pedi} = redicted average crash frequency of vehicle-pedestrian crashes of an intersection (crashes/year)
N_{bikei} = predicted average crash frequency of vehicle-bicycle crashes of an intersection (crashes/year)
C_i = calibration factor to adjust the SPF for intersection type i to local conditions

Similar to Table 12-16 in the HSM, Table 14 provides pedestrian crash adjustment factors for urban all-way stop-controlled intersections. The number of vehicle-pedestrian crashes per year for an all-way stop-controlled intersection is estimated as:

$$N_{pedi} = N_{bi} \times f_{pedi} \quad \text{(Eq. 9)}$$

Where:

f_{pedi} = pedestrian crash adjustment factor for intersection type *i*

Table 14. Pedestrian crash adjustment factors for urban and suburban all-way stop-controlled intersections

Intersection Type	Pedestrian Crash Adjustment Factor (f_{pedi})
Three-Leg	0.017
Four-Leg	0.015

Similar to Table 12-17 in the HSM, Table 15 provides bicycle crash adjustment factors for urban all-way stop-controlled intersections. The number of vehicle-bicycle crashes per year for an all-way stop-controlled intersection is estimated as:

$$N_{bikei} = N_{bi} \times f_{bikei} \quad \text{(Eq. 12)}$$

Where:

f_{bikei} = bicycle crash adjustment factor for intersection type *i*

Table 15. Bicycle crash adjustment factors for urban and suburban all-way stop-controlled intersections

Intersection Type	Bicycle Crash Adjustment Factor (f_{bikei})
Three-Leg	0.011
Four-Leg	0.011

Following the development of the crash prediction models for rural and urban and suburban all-way stop-controlled intersections, the research team conducted compatibility testing of the new models to confirm that the new models provide reasonable results over a broad range of input conditions and that the new models integrate seamlessly with existing intersection crash prediction models in the first edition of the HSM. The graphical representations of the crash prediction models in Figures 5-10 provide some sense of the reasonableness of the new models for all-way stop-controlled intersections. Nothing from these figures suggests that the models provide unreasonable results. In addition, the new models for all-way stop-controlled intersections were compared to the associated minor road stop-controlled intersection SPFs in the HSM. Figure 11 illustrates a comparison of the predicted average crash frequency for total crashes based on the 4aST model for rural two-lane roads (Table 9) to the predicted average crash frequency based on the 4ST model in Chapter 10 of the HSM. In the figure, the dashed lines represent the predicted average crash frequency for the 4aST model, and the solid lines represent the predicted average crash frequency for the 4ST model in the HSM. Similarly, Figure 12 illustrates a comparison of the predicted average crash frequency for FI crashes based on the 4aST model for urban and suburban arterials (Table 12) to the predicted average crash frequency based on the 4ST model in Chapter 12 of the HSM. In both instances, as major approach AADT increases, the 4aST SPFs predict fewer crashes than the 4ST SPFs in the HSM which seems reasonable as the traffic control at all-way stop-controlled intersections provides more direction in terms of defining the right of way through the intersection for drivers so it is reasonable to expect fewer crashes at all-way stop-controlled intersections than intersections with minor road stop control, given the same traffic volumes. Although not presented herein, a similar trend was found in terms of the 3aST SPFs predicting fewer crashes than the 3ST SPFs in the HSM as the major road AADT increases for intersections on urban and suburban arterials.

In summary, the models for all-way stop-controlled intersections appear to provide reasonable results over a broad range of input conditions and can be integrated seamlessly with existing intersection crash prediction models in the first edition of the HSM.

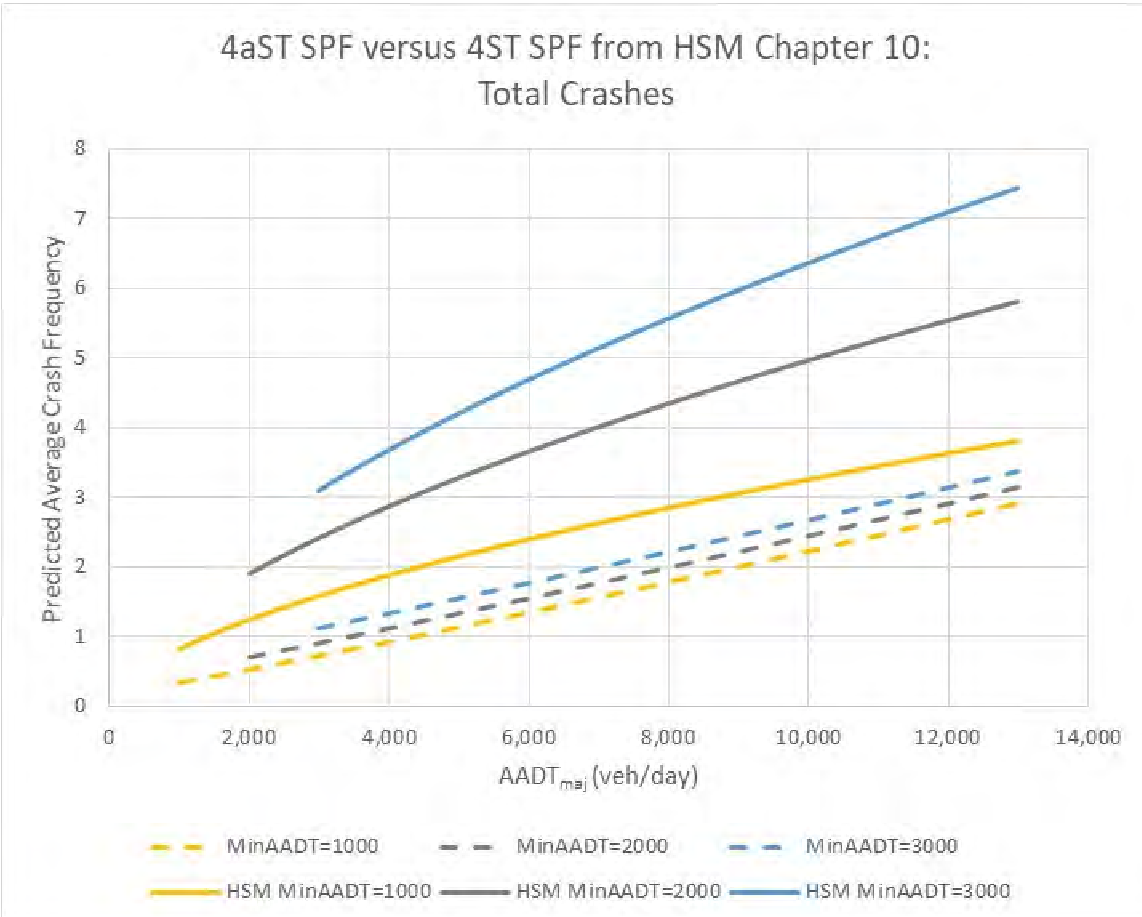


Figure 11. Comparison of new crash prediction model to existing model in HSM: 4aST vs 4ST on rural two-lane roads (total crashes)

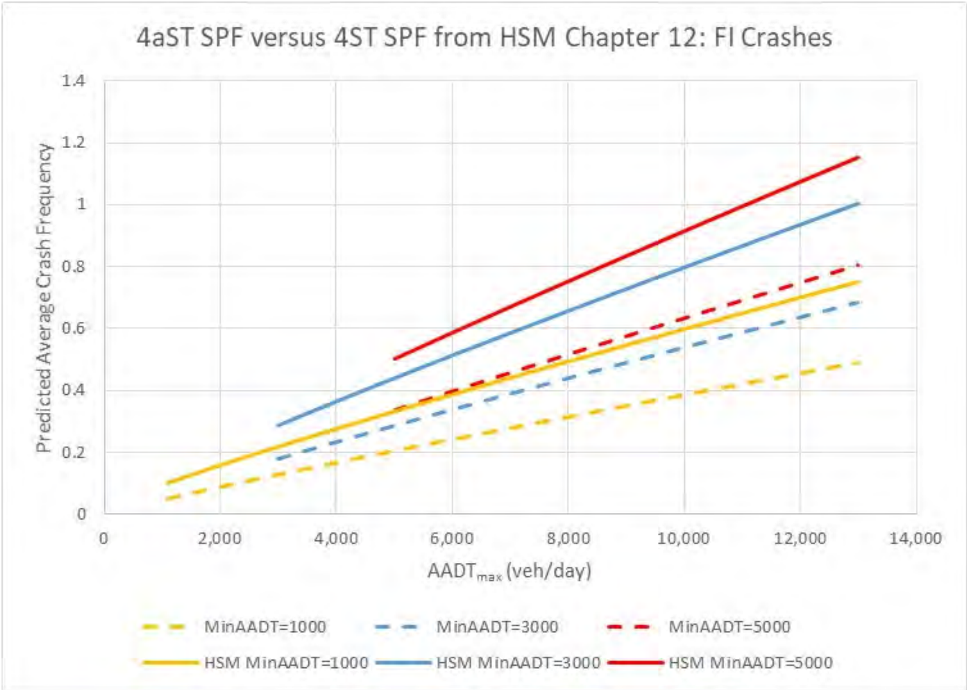


Figure 12. Comparison of new crash prediction model to existing model in HSM: 4aST vs 4ST on urban and suburban arterials (FI crashes)

3.4 Crash Modification Factor

During the development of the crash prediction models for all-way stop-controlled intersections, three potential sources of CMFs for use with the SPFs were considered:

- CMFs developed as part of this research based on a cross-sectional study design and regression modeling
- CMFs already incorporated into the first edition of the HSM and applicable to all-way stop-controlled intersections
- High quality CMFs applicable to all-way stop-controlled intersections developed using defensible study designs (e.g., observational before-after evaluation studies using SPFs—the EB method), as referenced in FHWA's CMF Clearinghouse with four or five-star quality ratings or based on a review of relevant intersection safety literature

Based on the regression modeling as part of this research, no geometric features or traffic control devices were identified for CMF development. Based on a review of the CMFs already incorporated in the first edition of the HSM and other potential high-quality CMFs developed using defensible study designs, the only CMF that was identified for potential use with the crash prediction models for all-way stop-controlled intersections was the CMF for intersection lighting based on the work by Elvik and Vaa (2004), which is identified for use with the intersection crash prediction models in Chapters 10, 11, and 12 of the first edition of the HSM.

Thus, the only CMF recommended for use with the final SPFs for all-way stop-controlled intersections is the CMF for intersection lighting based on the work by Elvik and Vaa (2004). With this CMF, the base condition is the absence of intersection lighting. The CMF for lighted intersections is similar to the CMF in Equation 10-24 (rural intersections) and Equation 12-36 (urban intersections) in the HSM and has the form:

$$CMF_i = 1 - 0.38 \times p_{ni} \quad \text{(Eq. 39)}$$

Where:

CMF_i = crash modification factor for the effect of lighting on total crashes; and
p_{ni} = proportion of total crashes for unlighted intersections that occur at night.

This CMF applies to total intersection crashes. Table 16 (similar to Tables 10-15 and 12-27 in the HSM) presents values for the nighttime crash proportion, p_{ni}, by area type and intersection type.

Table 16. Nighttime proportions for unlighted all-way stop-controlled intersections by area and intersection type

Intersection Type	Proportion of Nighttime Crashes (p_{ni})
RURAL INTERSECTIONS	
Three-Leg	0.363
Four-Leg	0.284
URBAN AND SUBURBAN INTERSECTIONS	
Three-Leg	0.187
Four-Leg	0.277

Recent research by Washington State DOT has raised concerns about whether use of the lighting CMF in the HSM is appropriate. Based on their research, van Schalkwyk et al. (2016) concluded that the contribution of continuous illumination to nighttime crash reduction is negligible. However, we have recommended this CMF for application to all-way stop-controlled intersections because this CMF has been used in the HSM first edition. If any decision to remove or change the lighting CMFs is made, this should be done consistently for all facility types as part of the development of the HSM second edition.

3.5 Severity Distribution Functions

Development of SDFs was explored for all-way stop-controlled intersections using methods outlined in Section 2.2.3 of this report. SDFs were not used in the development of crash prediction methods in the first edition of the HSM but were subsequently used in the Supplement to the HSM for freeways and ramps (AASHTO, 2014). The database used to explore SDFs for all-way stop-controlled intersections consisted of the same crashes and intersections as the database used to estimate the SPFs for rural three-leg, rural four-leg, urban three-leg, and urban four-leg intersections, but restructured so that the basic observation unit (i.e., database row) is a crash instead of an intersection.

No traffic or geometric variables showed consistent and statistically significant effects in the SDFs for rural three-leg, rural four-leg, urban three-leg, or urban four-leg all-way stop-controlled intersections. Therefore, distributions for rural all-way stop-controlled intersections in Table 10 and SPFs by severity for urban all-way stop-controlled intersections in Table 12 are recommended for addressing severity at all-way stop-controlled intersections.

3.6 Summary of Recommended Models for Incorporation in the HSM

In summary, several crash prediction models were developed for intersections with all-way stop control for consideration in the second edition of the HSM, including models for:

- Rural three-leg intersections with all-way stop control
- Rural four-leg intersections with all-way stop control
- Urban and suburban three-leg intersections with all-way stop control
- Urban and suburban four-leg intersections with all-way stop control

The final models for rural four-leg intersections with all-way stop control (total crashes), urban and suburban three-leg intersections with all-way stop control (FI and PDO crashes), and urban and suburban four-leg intersections with all-way stop control (FI and PDO crashes) are recommended for inclusion in the second edition of the HSM (see Tables 9 and 12). The model for rural three-leg intersections with all-way stop control is not recommended for inclusion in the second edition of the HSM because the number of sites used to develop the model is not considered sufficient for including the model in the HSM.

In addition, no traffic or geometric variables showed consistent and statistically significant effects in the SDFs for all-way stop-controlled intersections. Therefore, it is recommended for the second edition of the HSM that crash severity for all-way stop-controlled intersections on rural two-lane highways and urban and suburban arterials be addressed in a manner consistent with existing methods in Chapter 10 and Chapter 12 of the HSM, respectively, without use of SDFs.

Appendix A presents recommended text for incorporating the final recommended models for all-way stop-controlled intersections into Chapters 10 and 12 of the HSM.

Chapter 4. Development of Models for Use in HSM Crash Prediction Methods: Three-Leg Intersections with Signal Control on Rural Highways

This section of the report describes the development of crash prediction models for three-leg intersections with signal control on rural highways and presents the final models recommended for incorporation in the second edition of the HSM. Chapters 10 (Rural Two-Lane, Two-Way Roads) and 11 (Rural Multilane Highways) in the first edition of the HSM do not include crash prediction models for three-leg intersections with signal control. Crash prediction models are recommended for the following intersection types for the second edition of the HSM:

- Three-leg intersections with signal control (3SG) on rural two-lane, two-way roads
- Three-leg intersections with signal control (3SG) on rural multilane highways

Section 4.1 describes the site selection and data collection process for developing the crash prediction models for three-leg intersections with signal control on rural highways. Section 4.2 presents descriptive statistics of the databases used for model development. Section 4.3 presents the statistical analysis and SPFs developed for three-leg intersections with signal control on rural highways. Section 4.4 presents the CMFs recommended for use with the SPFs. Section 4.5 presents the results of an analysis to develop SDFs for use with the SPFs for three-leg intersections with signal control on rural highways, and Section 4.6 summarizes the recommendations for incorporating new crash prediction models for three-leg intersections with signal control on rural highways in the second edition of the HSM.

4.1 Site Selection and Data Collection

A list of potential intersections for model development was derived from HSIS or Safety Analyst databases for several states, as well as lists provided by state transportation agencies. The intersections were located in ten states:

- California (CA)
- Florida (FL)
- Illinois (IL)
- Kentucky (KY)
- Michigan (MI)
- Minnesota (MN)
- New Hampshire (NH)
- Ohio (OH)
- Pennsylvania (PA)
- Washington (WA)

Each intersection in the list was initially screened using Google Earth® to determine if the site was suitable for inclusion in model development. Several reasons a site could be deemed inappropriate for use in model development were:

- The traffic control at the intersection was something other than signal control.
- The number of intersection legs was not three.
- The intersection was in an urban area.
- A private driveway was located in close proximity to the intersection.
- One or more of the approaches to the intersection was a private/commercial access.
- Google Street View® was not available to identify leg specific attributes.
- One or more of the intersection legs was a one-way street.

Each intersection that was initially deemed appropriate for inclusion in model development was given a unique identification code and included in a refined database for detailed data collection.

Three types of data were collected for each intersection during detailed data collection: site characteristic, crash, and traffic volume data. Google Earth® was used to collect detailed site characteristics of the intersections. To reduce potential errors during data collection and to streamline data entry, a data collection tool was created using Visual Basic for Applications, very similar to the tool shown in Figure 4. The data collection tool was suited to only collect data relevant to rural three-leg signalized intersections. Table 17 lists all of the intersection attributes collected (and respective definitions and permitted values) for rural three-leg signalized intersections using the data collection tool. Once all necessary data were entered into the data collection tool and saved for a given intersection, the data collection tool was used to validate the inputs for that particular intersection consistent with the range and/or permitted values for the respective variables/parameters.

Table 17. Site characteristic variables collected for three-leg intersections with signal control on rural highways

Variable/Parameter	Definition	Range or Permitted Values
General Intersection Attributes		
Intersection configuration (i.e., number of legs and type of traffic control)	Indicates the number of legs and type of traffic control	3SG
Area type (urban/rural)	Indicates whether the intersection is in a rural or urban area	Rural
Presence of intersection lighting	Indicates if overhead lighting is present at the intersection proper	Yes, no
Approach Specific Attributes		
Route name or number	Specify the route name or number of the approach	
Location at intersection	Side/quadrant of the intersection the approach is located	N, S, E, W, NE, NW, SE, SW
Number of through lanes	This includes dedicated through lanes and any lanes with shared movements. On the minor approach of a 3-leg intersection, if there is only one lane, then it should be classified as a through lane	0, 1, 2, 3
Presence/number of left-turn lanes	The number of lanes in which only a left-turn movement can be made	0, 1, 2, 3
Left-turn channelization	Type of left-turn channelization used on the intersection approach	Raised or depressed island, painted, none
Presence/number of right-turn lanes	The number of lanes in which only a right-turn movement can be made	0, 1, 2, 3

Table 17. Site characteristic variables collected for three-leg intersections with signal control on rural highways (Continued)

Variable/Parameter	Definition	Range or Permitted Values
Right-turn channelization	Type of right-turn channelization used on the intersection approach	Raised or depressed island, painted, none
Median width	Measured from outside of outer most through lane of approaching lanes to outside of lane in opposing direction	Values in feet
Median type	Type of median separating opposing directions of travel	Raised, depressed, flush, barrier, TWLTL
Permit right-turn-on-red	Indicates if turning right on red is permitted on the intersection approach	Yes, no, not applicable
Presence of transverse rumble strips	Indicates the presence of transverse rumble strips on the intersection approach	Yes, no, unknown
Presence/type of supplementary pavement markings	Indicates the presence and type of supplementary pavement markings on the intersection approach	Yes, no, unknown If yes, type of marking: "Signal Ahead", other
Presence of signal ahead warning signs	Indicates the presence of signal ahead warning signs on the intersection approach	Yes, no, unknown
Presence of advance warning flashers	Indicates the presence of advance warning flashers on the intersection approach	Yes, no, unknown
Horizontal alignment of intersection approach	Indicates whether the approaching roadway, within 250 ft of the intersection, is a tangent or curved section of roadway	Tangent, curve
Horizontal curve radius	Indicates the radius of the curve on the intersection approach if a curve is present within 250 ft of the intersection	2,000-ft Maximum Range: 100-2000 ft
Posted speed limit	Posted speed limit on the intersection approach (mph)	25, 30, 35, 40, 45, 50, 55, 60, 65, unknown
Presence of crosswalk	Indicates the presence of a crosswalk perpendicular to the intersection approach	Yes, no, unknown
Presence of bike lane	Indicates the presence of a marked bike lane parallel to the intersection approach	Yes, no, unknown
Presence of railroad crossing	Indicates the presence of a railroad crossing on the intersection approach within 250 ft of the intersection	Yes, no, unknown

During detailed data collection, to the extent possible, the research team reviewed historical aerial images to determine if a site had recently been reconstructed or improved to determine the appropriate years of data for use in model development. Due to very low intersection totals, Minnesota and Pennsylvania sites were removed from the dataset for model development.

Table 18 lists the crash and traffic volume data sources for the eight states included in the study. The goal was to obtain the most recent four to six years of crash and traffic volume data for each site for model development. All of the data (i.e., site characteristics, crash, and traffic volume) were assembled into one database for model development.

Table 18. Traffic volume and crash data sources for rural three-leg signalized intersections

State	Traffic Volume Data Source	Crash Data Source
California	HSIS	HSIS
Florida	State agency	State agency
Illinois	Safety Analyst	Safety Analyst
Kentucky	State agency	State agency
Michigan	Safety Analyst	Safety Analyst
New Hampshire	Safety Analyst	Safety Analyst
Ohio	Safety Analyst	Safety Analyst
Washington	HSIS and Safety Analyst	HSIS

4.2 Descriptive Statistics of Database

A total of 161 sites—89 on two-lane and 72 on multilane highways—were available for development of crash prediction models. The data collections sites were located in eight states: California, Florida, Illinois, Kentucky, Michigan, New Hampshire, Ohio, and Washington. To remain consistent with the standards for development of the intersection predictive models in the first edition of the HSM, the goal of this research was to develop crash prediction models with a minimum of 200 site-years of data, and preferably 450 site-years of data or more.

4.2.1 Traffic Volumes and Site Characteristics

Traffic volume and crash data were available for varying periods but were typically available over a 5- or 6-year period. Table 19 shows the breakdown of all sites by roadway classification and intersection type. Study period (date range), number of sites and site-years, and basic traffic volume statistics are shown by state in each category and across all states within a category.

Of the intersection characteristics collected in Google Earth®(see Table 17), most showed no or very little variability across sites within a roadway classification (i.e., most intersections were predominantly of one type for a specific variable). However, the following three intersection characteristics had sufficient variability for inclusion in the development of SPFs using CMFs (percent of "Yes" by roadway classification is indicated in parentheses):

- presence of intersection lighting (two-lane: 81%; multilane: 74%)
- presence of left-turn lanes on major road (two-lane: 89%; multilane: 93%)
- presence of right-turn lanes on major road (two-lane: 65%; multilane: 67%)

The use of these site characteristics is discussed later in the SPF model development section.

4.2.2 Crashes

Of the 161 intersections included in the study, only 11 intersections (6.8%) experienced no crashes over the entire study period; their breakdown by roadway classification is as follows:

- Rural three-leg signalized intersections on two-lane highways: 3 out of 89
- Rural three-leg signalized intersections on multilane highways: 8 out of 72

Intersection crashes were defined as those crashes that occurred within 250 ft of the intersection and were classified as at intersection or intersection-related, consistent with recommended practice in the HSM for assigning crashes to an intersection.

Table 20 shows total, FI, and PDO crash counts by roadway classification and crash severity for each state over the entire study period. Counts are also shown for nighttime crashes only. For one of the 10 intersections on two-lane highways and one of the 17 intersections on multilane highways in Florida, crash severity was unknown. Similarly, for eight of the nine intersections on two-lane highways and 13 of the 15 intersections on multilane highways in Kentucky, crash severity was unknown. In those cases, only total crash counts are shown, and thus FI and PDO crashes do not add up to the total crash count.

Table 19. Major- and minor road AADT statistics by roadway classification for rural three-leg intersections with signal control

State	Date Range	Number of Sites	Number of Site-Years	Major Road AADT (veh/day)				Minor Road AADT (veh/day)			
				Min	Max	Mean	Median	Min	Max	Mean	Median
INTERSECTIONS ON RURAL TWO-LANE HIGHWAYS											
CA	2007-2011	24	112	3,130	23,591	14,521	14,233	100	23,320	3,034	1,166
FL	2007-2010	10	40	6,600	21,425	12,520	11,425	3,650	18,225	6,730	5,560
IL	2008-2012	6	30	2,900	5,300	4,575	4,800	1,900	5,100	3,425	3,375
KY	2010-2014	9	45	6,707	11,800	8,969	9,452	2,330	7,866	4,447	4,106
MI	2008-2013	6	34	7,353	21,058	13,829	12,875	4,005	16,329	9,802	9,523
NH	2009-2013	7	35	10,927	16,000	12,315	11,277	1,400	8,800	4,725	4,600
OH	2009-2013	17	85	3,832	14,930	7,949	8,197	190	10,336	2,772	729
WA	2007-2011	10	42	5,669	18,612	12,259	12,334	1,881	13,068	6,685	6,434
All States	**2007-2014**	**89**	**423**	**2,900**	**23,591**	**11,334**	**10,239**	**100**	**23,320**	**4,568**	**4,106**
INTERSECTIONS ON RURAL MULTILANE HIGHWAYS											
CA	2007-2011	21	77	1,001	56,000	19,675	19,360	101	27,000	4,734	1,500
FL	2007-2010	17	65	7,375	25,000	17,181	16,925	800	17,300	7,188	6,000
KY	2010-2014	15	75	8,249	25,151	15,304	15,241	677	9,108	4,394	3,762
MI	2008-2013	4	24	9,735	11,705	11,077	11,434	6,786	8,547	7,777	7,887
OH	2009-2013	15	73	2,456	28,402	11,182	10,694	156	11,510	2,952	2,620
All States	**2007-2014**	**72**	**314**	**1,001**	**56,000**	**15,928**	**15,834**	**101**	**27,000**	**5,040**	**3,752**

Table 20. Crash counts by roadway classification and crash severity for rural three-leg intersections with signal control

State	Date Range	Number of Sites	Number of Site-Years	Time of Day	Total	FI	PDO
INTERSECTIONS ON RURAL TWO-LANE HIGHWAYS							
CA	2007-2011	24	112	All	98	45	53
				Night	25	12	13
FL[a]	2007-2010	10	40	All	103	68	34
				Night	33	25	7
IL	2008-2012	6	30	All	56	15	41
				Night	8	3	5
KY[a]	2010-2014	9	45	All	168	29	10
				Night	32	5	2
MI	2008-2013	6	34	All	142	27	115
				Night	42	6	36
NH	2009-2013	7	35	All	37	13	24
				Night	5	1	4
OH	2009-2013	17	85	All	164	40	124
				Night	32	8	24
WA	2007-2011	10	42	All	119	45	74
				Night	30	9	21
All States[a]	**2007-2014**	**89**	**423**	**All**	**887**	**282**	**475**
				Night	**207**	**69**	**112**
INTERSECTIONS ON RURAL MULTILANE HIGHWAYS							
CA	2007-2011	21	77	All	104	45	59
				Night	24	10	14
FL[a]	2007-2010	17	65	All	257	139	117
				Night	53	34	18
KY[a]	2010-2014	15	75	All	356	76	15
				Night	71	17	5
MI	2008-2013	4	24	All	95	17	78
				Night	10	1	9
OH	2009-2013	15	73	All	163	42	121
				Night	33	12	21
All States[a]	**2007-2014**	**72**	**314**	**All**	**975**	**319**	**390**
				Night	**191**	**74**	**67**

[a] Crash records did not indicate severity level for a number of crashes for some intersections; FI and PDO crashes will not add up to total crashes.

Crash counts are tallied by collision type and manner of collision across all states in Table 21 for intersections on rural two-lane highways. Crash counts tallied across all states by collision type and severity at intersections on multilane highways are shown in Table 22.

Table 21. Crash counts by collision type and manner of collision and crash severity at three-leg intersections with signal control on rural two-lane highways

Collision Type	Total[a]	FI	PDO
SINGLE-VEHICLE CRASHES			
Collision with animal	16	0	16
Collision with bicycle	3	2	1
Collision with pedestrian	0	0	0
Overturned	16	13	3
Ran off road	1	0	1
Other SV crash	137	35	90
Total SV crashes	**173**	**50**	**111**
MULTIPLE-VEHICLE CRASHES			
Angle collision	171	74	75
Head-on collision	24	16	8
Rear-end collision	408	120	220
Sideswipe collision	43	7	22
Other MV collision	68	15	39
Total MV crashes	**714**	**232**	**364**
Total Crashes	**887**	**282**	**475**

[a] Crash records did not indicate severity level for a number of crashes for some intersections in Florida and Kentucky; FI and PDO crashes will not add up to total crashes in some cases.

Table 22. Crash counts by collision type and crash severity at three-leg intersections with signal control on rural multilane highways

Collision Type	Total[a]	FI (KABC)	FI[b] (KAB)	PDO
Head-on collision	23	13	4	7
Sideswipe collision	78	11	7	42
Rear-end collision	421	117	58	172
Angle collision	273	130	86	86
SV collision	110	31	18	59
Other	70	17	9	24
Total Crashes	**975**	**319**	**182**	**390**

[a] Crash records did not indicate severity level for a number of crashes for some intersections in Florida and Kentucky; FI and PDO crashes will not add up to total crashes in some cases.
[b] Using the KABCO scale, these include only KAB crashes. Crashes with severity level C (possible injury) are not included.

4.3 Safety Performance Functions—Model Development

SPFs of the form shown in Equation 2 were developed separately for intersections on two-lane and multilane highways, using all crash types combined (single- and MV and pedestrian and bicycle crashes).

$$N_{spf\ int} = exp[a + b \times \ln(AADT_{maj}) + c \times \ln(AADT_{min})] \quad \text{(Eq. 2)}$$

Where:

$N_{spf\ int}$ = predicted average crash frequency for an intersection with base conditions (crashes/year);
$AADT_{maj}$ = AADT on the major road (veh/day)
$AADT_{min}$ = AADT on the minor road (veh/day)
a, b, and c = estimated regression coefficients

Based on a review of the number of states, sites, site-years, and crashes for the database assembled, data for all sites were used for model development to maximize the sample size rather than using a portion of the data for model development and a portion for model validation.

All SPFs were developed using a NB regression model based on all sites combined within a given area type and intersection type. In all models, state was included as a random blocking effect, with sites nested within their respective state. A significance level of 0.20 for inclusion in a model was selected for an individual parameter. This was based on previous models included in the first edition of the HSM (Harwood et al., 2007). PROC GLIMMIX of SAS 9.3 was used for all modeling (SAS, 2013). For intersections on either highway type, models were developed for total, FI, and PDO crashes. For intersections on multilane highways, a model was also developed for a subset of FI crashes including only KAB crashes (i.e., crashes with severity level C [possible injury] not included) as done in Chapter 11 in the HSM.

For comparison, the base conditions in Chapter 10 of the HSM for four-leg intersections with signal control on rural two-lane highways are the absence of intersection lighting and that of left- and right-turn lanes. In Chapter 11 of the HSM, the SPFs for four-leg intersections with signal control on rural multilane highways have no specific base conditions. Considering the absence of intersection lighting and that of left- and right-turn lanes as possible base conditions for three-leg intersections with signal control for both rural two-lane and multilane highways, the distribution of intersections by the three characteristics is as follows:

- Intersections on two-lane highways: 72 lighted; 17 unlighted (19% unlighted)
- Intersections on multilane highways: 53 lighted; 19 unlighted (26% unlighted)
- Intersections on two-lane highways: 58 with right-turn lane on one approach; 31 with none (35% with none)
- Intersections on multilane highways: 48 with right-turn lane on one approach; 24 with none (33% with none)
- Intersections on two-lane highways: 79 with left-turn lane on one approach; 10 with none (11% with none)
- Intersections on multilane highways: 66 with left-turn lane on one approach; 1 with left-turn lane on two approaches; 5 with none (7% with none)

In an effort to include all intersections in the models, crashes at intersections that did not meet base conditions for these three characteristics were first adjusted using the following CMFs in reverse (i.e., divide rather than multiply the crashes by the product of the CMFs):

- Lighting: use CMF_{4i}, shown in Equation 10-24 in the HSM (same as Equation 11-22 in the HSM) and the proportion of total crashes for unlighted intersections that occurred at night in the current database, shown in Table 23 (similar to Tables 10-15 and 11-24 in the HSM); this CMF was applied to total, FI, and PDO crashes before modeling

- Installation of left-turn lanes: use CMF_{2i} shown in Table 30 (same as Tables 10-13 and 11-22 in the HSM with the addition of a CMF for three-leg intersections with signal control)
- Installation of right-turn lanes: use CMF_{3i} shown in Table 31 (same as Tables 10-14 and 11-23 in the HSM with the addition of a CMF for three-leg intersections with signal control)

Table 23. Nighttime crash counts and proportions for unlighted three-leg intersections with signal control by roadway type used for modeling

Roadway Type	Number of Sites[a]	Number of Nighttime Crashes	Total Crashes	Proportion of Crashes that Occurred at Night (p_{ni})
Two-lane	17	58	247	**0.235**
Multilane	19	50	244	**0.205**

[a] Number of unlighted intersections only.

The final SPF models for crashes at intersections on rural two-lane highways are shown in Table 24, separately for each crash severity. The table shows the model coefficients and overdispersion parameter (estimate), their standard error, and associated p-values (or significance level) for each severity level. Figures 13-15 graphically present the SPFs shown in Table 24 for various major- and minor approach AADTs.

Table 24. SPF coefficients for three-leg intersections with signal control on rural two-lane highways

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
INTERSECTIONS ON RURAL TWO-LANE HIGHWAYS					
Total Crashes	Intercept	-5.88	1.89	--	--
	ln($AADT_{maj}$)	0.54	0.18	<.01	Significant at 99% level
	ln($AADT_{min}$)	0.23	0.07	<.01	Significant at 99% level
	Overdispersion	0.31	0.06	--	--
FI Crashes	Intercept	-9.69	3.22	--	--
	ln($AADT_{maj}$)	0.78	0.31	0.01	Significant at 99% level
	ln($AADT_{min}$)	0.24	0.12	0.04	Significant at 95% level
	Overdispersion	0.72	0.19	--	--
PDO Crashes	Intercept	-6.49	2.50	--	--
	ln($AADT_{maj}$)	0.50	0.24	0.04	Significant at 95% level
	ln($AADT_{min}$)	0.26	0.09	<.01	Significant at 99% level
	Overdispersion	0.49	0.11	--	--

Base Condition: Absence of intersection lighting and absence of left- and right-turn lanes

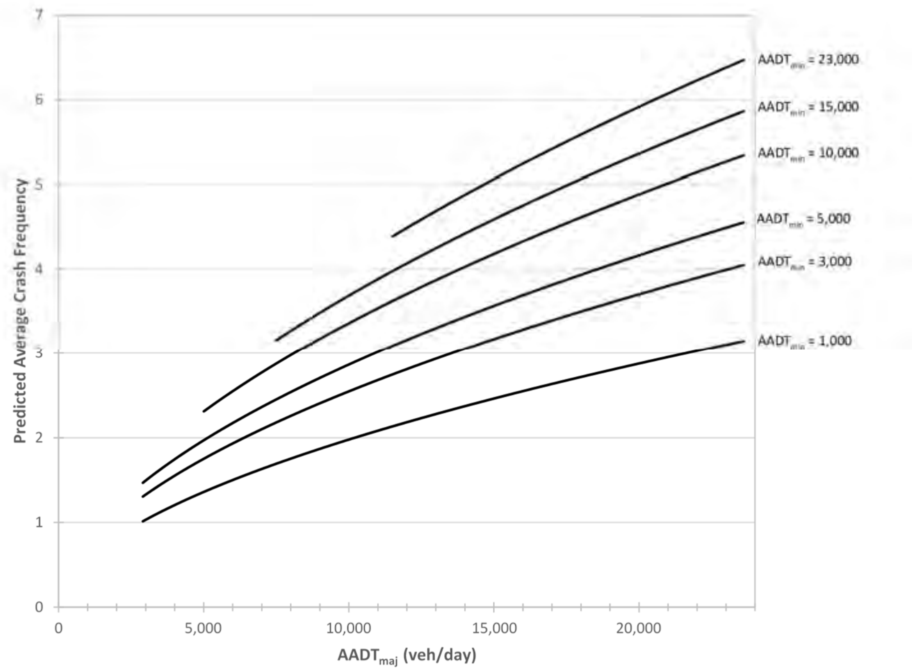


Figure 13. Graphical representation of the SPF for total crashes at three-leg intersections with signal control on rural two-lane highways

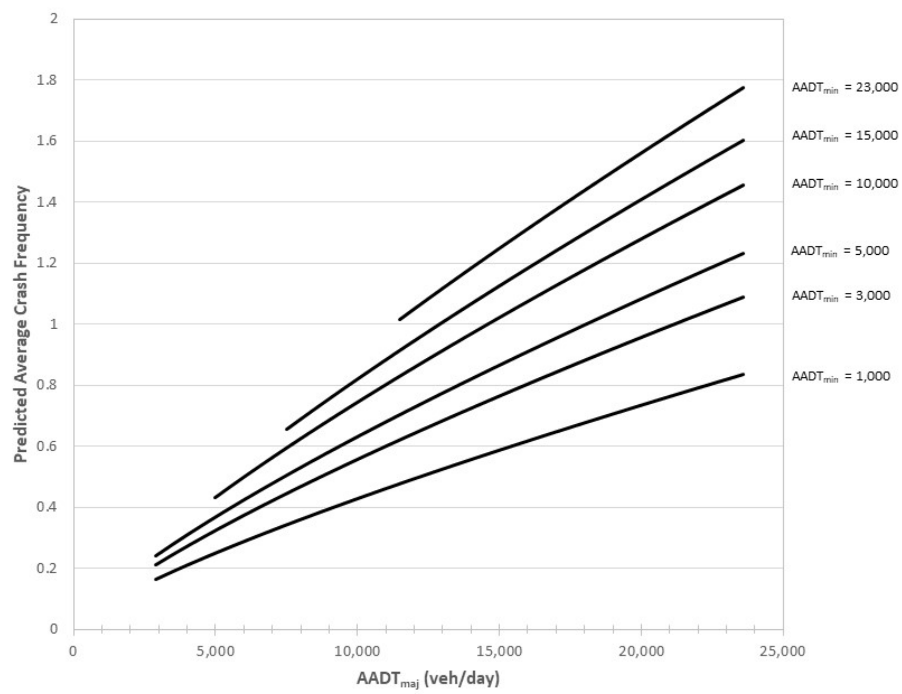


Figure 14. Graphical representation of the SPF for FI crashes at three-leg intersections with signal control on rural two-lane highways

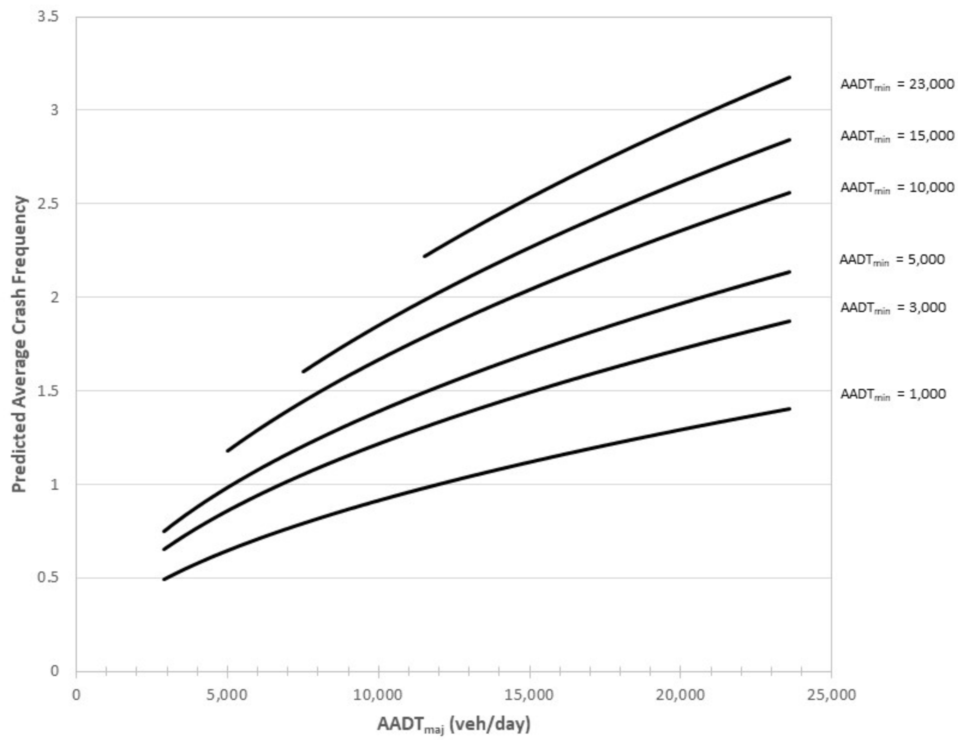


Figure 15. Graphical representation of the SPF for PDO crashes at three-leg intersections with signal control on rural two-lane highways

The final SPF models for crashes at three-leg intersections with signal control on rural multilane highways are shown in Table 25, separately for each crash severity. No usable model could be obtained for FI crashes considering KAB severities only. Figures 16-18 graphically present the SPFs shown in Table 25 for various major- and minor approach AADTs.

Table 25. SPF coefficients for three-leg intersections with signal control on rural multilane highways

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
INTERSECTIONS ON RURAL MULTILANE HIGHWAYS					
Total Crashes	Intercept	-6.28	1.97	--	--
	$\ln(AADT_{maj})$	0.52	0.21	0.02	Significant at 95% level
	$\ln(AADT_{min})$	0.31	0.08	<.01	Significant at 99% level
	Overdispersion	0.40	0.08	--	--
FI Crashes	Intercept	-11.03	3.81	--	--
	$\ln(AADT_{maj})$	0.79	0.39	0.05	Significant at 95% level
	$\ln(AADT_{min})$	0.39	0.14	<.01	Significant at 99% level
	Overdispersion	1.15	0.26	--	--
FI Crashes[a]	No usable model could be obtained				
PDO Crashes	Intercept	-6.40	2.49	--	--
	$\ln(AADT_{maj})$	0.44	0.27	0.10	Significant at 90% level
	$\ln(AADT_{min})$	0.30	0.10	<.01	Significant at 99% level
	Overdispersion	0.53	0.13	--	--

[a] Using the KABCO scale, these include only KAB crashes. Crashes with severity level C (possible injury) are not included.
Base Condition: Absence of intersection lighting and absence of left- and right-turn lanes

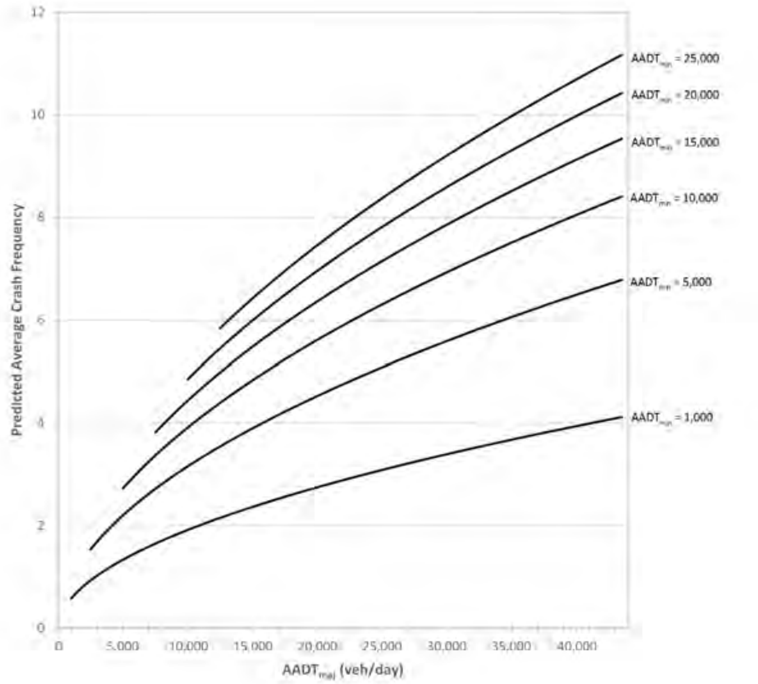


Figure 16. Graphical representation of the SPF for total crashes at three-leg intersections with signal control on rural multilane highways

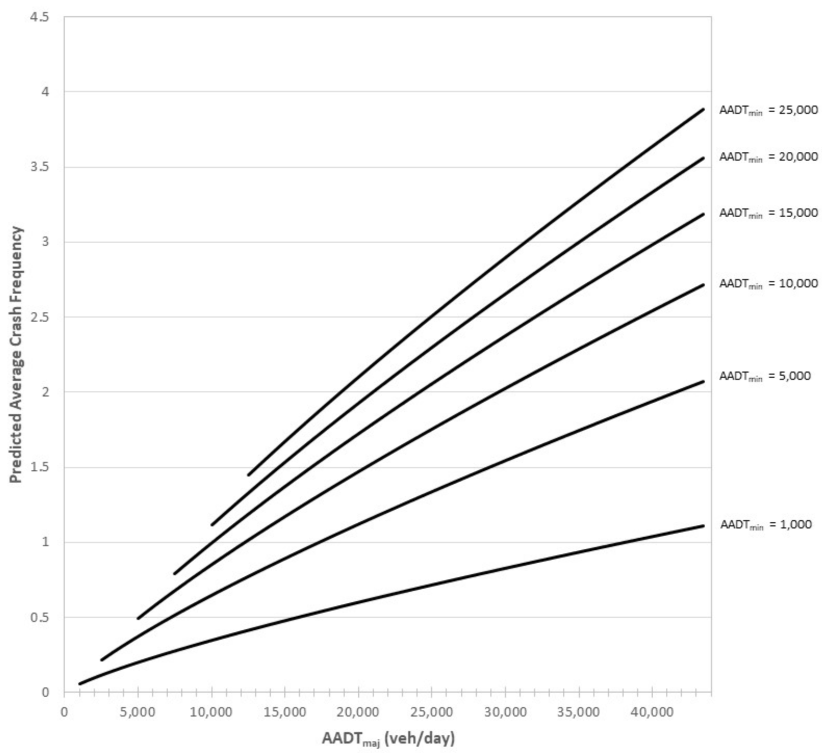


Figure 17. Graphical representation of the SPF for FI crashes at three-leg intersections with signal control on rural multilane highways

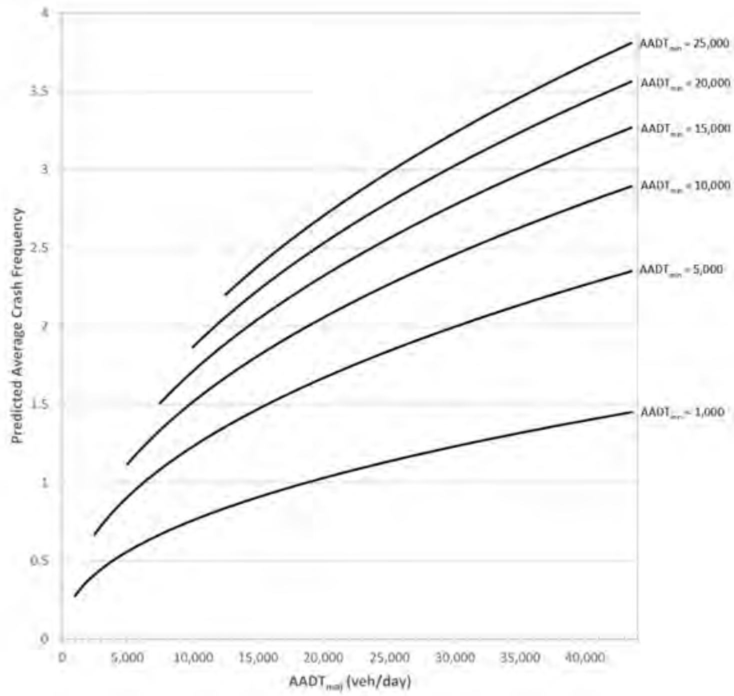


Figure 18. Graphical representation of the SPF for PDO crashes at three-leg intersections with signal control on rural multilane highways

Similar to Tables 10-5 and 10-6 in the HSM, respectively, Tables 26 and 27 provide percentages for crash severity levels and collision types and manner of collision, respectively, for three-leg intersections with signal control on rural two-lane highways. These percentages were calculated based on all crash counts at all intersections in all states combined, excluding those sites in Florida and Kentucky with missing crash severity information.

Table 26. Distributions for crash severity level at three-leg intersections with signal control on rural two-lane highways

Crash Severity Level	Percentage of Total Crashes
Fatal	0.1
Incapacitating injury	2.4
Non-incapacitating injury	14.3
Possible injury	20.5
Total fatal plus injury	37.3
Property-damage-only	62.7
Total	**100.0**

Table 27. Distributions for collision type and manner of collision and crash severity at three-leg intersections with signal control on rural two-lane highways

Collision Type	Percentage of Total Crashes		
	Total	FI	PDO
SINGLE-VEHICLE CRASHES			
Collision with animal	1.8	0.0	3.4
Collision with bicycle	0.3	0.7	0.2
Collision with pedestrian	0.0	0.0	0.0
Overturned	1.8	4.6	0.6
Ran off road	0.1	0.0	0.2
Other SV crash	15.4	12.4	18.9
Total SV crashes	**19.4**	**17.7**	**23.3**
MULTIPLE-VEHICLE CRASHES			
Angle collision	19.3	26.2	15.8
Head-on collision	2.7	5.7	1.7
Rear-end collision	46.0	42.6	46.3
Sideswipe collision	4.8	2.5	4.6
Other MV collision	7.7	5.3	8.2
Total MV crashes	80.5	82.3	76.6
Total Crashes	100.0	100.0	100.0

Similar to Table 11-9 in the HSM, Table 28 provides percentages to break down total, FI (both with and without level C injuries), and PDO crash severities into specific collision types for three-leg intersections with signal control on rural multilane highways. These percentages were calculated based on all crash counts at all intersections in all states combined, excluding those sites in Florida and Kentucky with missing crash severity information (for FI and PDO statistics only).

Table 28. Distributions of intersection crashes by collision type and crash severity at three-leg intersections with signal control on rural multilane highways

Collision Type	Percentage of Total Crashes			
	Total	FI	FI[a]	PDO
Head-on collision	2.4	4.1	2.2	1.8
Sideswipe collision	8.0	3.4	3.8	10.8
Rear-end collision	43.2	36.7	31.9	44.1
Angle collision	28.0	40.8	47.3	22.1
SV collision	11.3	9.7	9.9	15.1
Other	7.2	5.3	4.9	6.2
Total Crashes	**100.0**	**100.0**	**100.0**	**100.0**

[a] Using the KABCO scale, these include only KAB crashes. Crashes with severity level C (possible injury) are not included.

Following the development of the crash prediction models for three-leg intersections with signal control on rural highways, the research team conducted compatibility testing of the new models to confirm that the new models provide reasonable results over a broad range of input conditions and that the new models integrate seamlessly with existing intersection crash prediction models in the first edition of the HSM. The graphical representations of the crash prediction models in Figures 13-18 provide some sense of the reasonableness of the new models for three-leg intersections with signal control on rural highways. Nothing from these figures suggests that the models provide unreasonable results. In addition, the models for three-leg intersections with signal control on rural highways were compared to the corresponding models for intersections on urban and suburban arterials in Chapter 12 of the HSM.

Figure 19 illustrates a comparison of the predicted average crash frequency for total crashes based on the 3SG model for rural two-lane roads (Table 24) to the predicted average crash frequency based on the 3SG model in Chapter 12 of the HSM. As previously, the dashed lines in the figure represent the predicted average crash frequency for the new model (i.e., 3SG model for rural two-lane roads), and the solid lines represent the predicted average crash frequency for the 3SG model in the HSM. Similarly, Figure 20 illustrates a comparison of the predicted average crash frequency for total crashes based on the 3SG model for rural multilane highways (Table 25) to the predicted average crash frequency based on the 3SG model in Chapter 12 of the HSM. In both instances, for lower major approach AADTs, higher average crash frequencies were predicted for three-leg signalized intersections on rural highways compared to urban and suburban arterials. As the major road AADT increased the predicted average crash frequencies drew closer together, and in some cases, the predicted average crash frequencies for three-leg intersections with signal control on urban and suburban arterials exceeded the predicted average crash frequencies for three-leg intersections with signal control on rural highways. This seems reasonable as drivers in rural areas during low volume conditions may not be as attentive to the task of driving as they would in an urban area and may be more susceptible to a crash as a result in rural environments given the same traffic volumes. Then, as traffic volumes increase in the rural areas, drivers' awareness to the driving task may increase to similar levels as in urban and suburban areas, resulting in similar levels of safety performance at three-leg signalized intersections in both rural and urban environments.

In summary, the models for three-leg signalized intersections on rural highways appear to provide reasonable results over a broad range of input conditions and can be integrated seamlessly with existing intersection crash prediction models in the first edition of the HSM.

Figure 19. Comparison of new crash prediction model to existing model in HSM: 3SG for rural two-lane roads vs 3SG for urban and suburban arterials (total crashes)

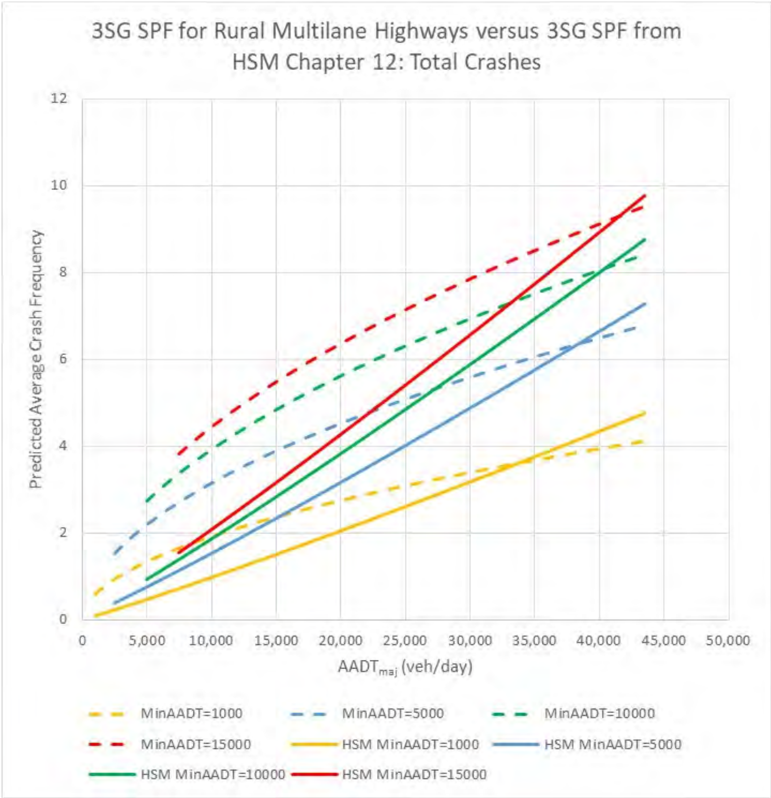


Figure 20. Comparison of new crash prediction model to existing model in HSM: 3SG for rural multilane highways vs 3SG for urban and suburban arterials (total crashes)

4.4 Crash Modification Factors

During the development of the crash prediction models for three-leg intersections with signal control on rural highways, three potential sources of CMFs for use with the SPFs were considered:

- CMFs developed as part of this research based on a cross-sectional study design and regression modeling
- CMFs already incorporated into the first edition of the HSM and applicable to three-leg intersections with signal control on rural highways
- High-quality CMFs applicable to three-leg intersections with signal control on rural highways developed using defensible study designs (e.g., observational before-after evaluation studies using SPFs—the EB method), as referenced in FHWA's CMF Clearinghouse with four or five-star quality ratings or based on a review of relevant intersection safety literature

After considering developing CMFs through regression modeling as part of this research and based on a review of the CMFs already incorporated in the first edition of the HSM and other potential high-quality CMFs developed using defensible study designs, three CMFs were identified for potential use with the crash prediction models for three-leg intersections with signal control on rural highways, including:

- The CMF for intersection lighting based on the work by Elvik and Vaa (2004), which is identified for use with the intersection crash prediction models in Chapters 10 and 11 of the first edition of the HSM
- The CMFs for providing a left-turn lane on one or two intersection approaches at a rural three-leg signalized intersection based on the work by Harwood et al. (2002), similar to the CMFs identified for use with intersection crash prediction models in Chapters 10 and 11 of the first edition of the HSM and included in HSM Part D
- The CMFs for providing a right-turn lane on one or two intersection approaches at a rural three-leg signalized intersection based on the work by Harwood et al. (2002), similar to the CMFs identified for use with intersection crash prediction models in Chapters 10 and 11 of the first edition of the HSM and included in HSM Part D

The CMFs recommended for use with the SPFs for three-leg intersections with signal control on rural highways are presented below.

4.4.1 Lighting CMF

With the CMF for intersection lighting based on the work by Elvik and Vaa (2004), the base condition is the absence of intersection lighting. The CMF for lighted intersections is similar to the CMF in Equation 10-24 (two-lane highways) and Equation 11-22 (multilane highways) in the HSM and has the form (AASHTO, 2010):

$$CMF_i = 1 - 0.38 \times p_{ni} \quad \text{(Eq. 39)}$$

Where:

CMF_i = crash modification factor for the effect of lighting on total crashes; and
p_{ni} = proportion of total crashes for unlighted intersections that occur at night.

This CMF applies to total intersection crashes. Table 29 (similar to Table 23; and similar to Tables 10-15 and 11-24 in the HSM) presents default values for the nighttime crash proportion, p_{ni}, by roadway type.

Table 29. Nighttime crash proportions for unlighted three-leg intersections with signal control

Roadway Type	Proportion of Crashes that Occur at Night (p_{ni})
Rural two-lane	0.235
Rural multilane	0.205

Recent research by Washington State DOT has raised concerns about whether use of the lighting CMF in the HSM is appropriate. Based on their research, van Schalkwyk et al. (2016) concluded that the contribution of continuous illumination to nighttime crash reduction is negligible. However, we have recommended this CMF for application to three-leg intersections with signal control on rural highways because this CMF has been used in the HSM first edition. If any decision to remove or change the lighting CMFs is made, this should be done consistently for all facility types as part of the development of the HSM second edition.

4.4.2 Intersection Approaches with Left-Turn Lanes CMF

With the CMF for providing a left-turn lane on one or two intersection approaches at a rural three-leg signalized intersection based on the work by Harwood et al. (2002), the base condition is the absence of left-turn lanes on intersection approaches. The CMFs for providing a left-turn lane on one or two intersection approaches are presented in Table 30. Table 30 is presented in the same format as Table 14-10 in the HSM Part D (AASHTO, 2010). These CMFs apply to all severity levels for three-leg intersections with signal control on both rural two-lane and multilane highways.

Table 30. CMF for installation of left-turn lanes on intersection approaches (Harwood et al., 2002; AASHTO, 2010)

Treatment	Setting (Intersection Type)	Traffic Volume AADT (veh/day)	Crash Type (Severity)	CMF		Std. Error
				One approach	Two approaches	
Installation of left-turn lanes	Rural (three-leg intersections with signal control)	Unspecified	All types (All severities)	0.85	0.72	N/A[a]

[a] Standard error of CMF is unknown.

4.4.3 Intersection Approaches with Right-Turn Lanes CMF

With the CMF for providing a right-turn lane on one or two intersection approaches at a rural three-leg signalized intersection based on the work by Harwood et al. (2002), the base condition is the absence of right-turn lanes on intersection approaches. The CMFs for providing a right-turn lane on one or two intersection approaches are presented in Table 31. Table 31 is presented in the same format as Table 14-15 in HSM Part D (AASHTO, 2010).

Table 31. CMF for installation of right-turn lanes on intersection approaches (Harwood et al., 2002; AASHTO, 2010)

Treatment	Setting (Intersection Type)	Traffic Volume AADT (veh/day)	Crash Type (Severity)	CMF	Std. Error
Installation of right-turn on one intersection approach	Rural (three-leg intersections with signal control)	Major road 7,000 to 55,100, minor road 550 to 8,400	All types (All severities)	0.96	0.02
			All types (Injury)	0.91	0.04
Installation of right-turn on two intersection approaches			All types (All severities)	0.92	0.03
			All types (Injury)	0.83	N/A[a]

[a] Standard error of CMF is unknown.

4.5 Severity Distribution Functions

The development of SDFs was explored for three-leg intersections with signal control on rural two-lane and multilane highways using methods outlined in Section 2.2.3 of this report. SDFs were not used in the development of crash prediction methods in the first edition of the HSM but were subsequently used in the Supplement to the HSM for freeways and ramps (AASHTO, 2014). Due to sample size issues, the data for three-leg intersections with signal control on rural two-lane and multilane highways were combined for development of the SDFs. Therefore, the SDFs that were developed are applicable to three-leg intersections with signal control on both rural two-lane and multilane highways. SDFs were not developed separately for three-leg intersections with signal control on rural two-lane and multilane highways. The database used to explore SDFs consisted of the same crashes and intersections as the databases used to estimate the SPFs, but restructured so that the basic observation unit (i.e., database row) is a crash instead of an intersection.

For three-leg intersections with signal control on both rural two-lane and multilane highways, the SDF takes the following form:

$$P_{3SG,at,K} = \frac{\exp(V_{KAB})}{1+\exp(V_{KAB})} \times P_{K|KAB,3SG,at} \quad \text{(Eq. 40)}$$

$$P_{3SG,at,A} = \frac{\exp(V_{KAB})}{1+\exp(V_{KAB})} \times P_{A|KAB,3SG,at} \quad \text{(Eq. 41)}$$

$$P_{3SG,at,B} = \frac{\exp(V_{KAB})}{1+\exp(V_{KAB})} \times \left(1 - P_{K|KAB,3SG,at} - P_{A|KAB,3SG,at}\right) \quad \text{(Eq. 42)}$$

$$P_{3SG,at,C} = 1 - \left(P_{3SG,at,K} + P_{3SG,at,A} + P_{3SG,at,B}\right) \quad \text{(Eq. 43)}$$

Where:

$P_{3SG,at,K}$ = probability of a fatal crash (given that a fatal or injury crash occurred) for three-leg signalized intersections (3SG) based on all collision types (at)

$P_{3SG,at,A}$ = probability of an incapacitating injury crash (given that a fatal or injury crash occurred) for three-leg signalized intersections (3SG) based on all collision types (at)

$P_{3SG,at,B}$ = probability of a non-incapacitating injury crash (given that a fatal or injury crash occurred) for three-leg signalized intersections (3SG) based on all collision types (at)

$P_{3SG,at,C}$ = probability of a possible injury crash (given that a fatal or injury crash occurred) for three-leg signalized intersections (3SG) based on all collision types (at)

V_{KAB} = systematic component of crash severity likelihood for severity KAB

$P_{K|KAB,3SG,at}$ = probability of a fatal crash given that the crash has a severity of either fatal, incapacitating injury, or non-incapacitating injury for three-leg signalized intersections (3SG) based on all collision types (at)

$P_{A|KAB,3SG,at}$ = probability of an incapacitating injury crash given that the crash has a severity of either fatal, incapacitating injury, or non-incapacitating injury for three-leg signalized intersections (3SG) based on all collision types (at)

The basic model form for the systematic components of crash severity likelihood at three-leg intersections with signal control on both rural two-lane and multilane highways is illustrated by Equation 44.

$$V_{KAB} = a + (b \times 0.001 \times AADT_{maj}) + (c \times I_{light}) + (d \times n_{majLTL}) + (e \times n_{majthru}) \quad \text{(Eq. 44)}$$

Where:

$AADT_{maj}$ = AADT on the major road (veh/day)

I_{light} = intersection lighting indicator variable (1 if lighting is present,0 otherwise)

n_{majLTL} = total number of left-turn lanes on both major road approaches (0, 1, or 2)

$n_{majthru}$ = total number of through lanes on the major road

a, b, c, d, and e = estimated SDF coefficients

The SDF coefficients for three-leg intersections with signal control on rural two-lane and multilane highways are provided in Table 32.

Table 32. SDF coefficients for three-leg intersections with signal control on rural two-lane and multilane highways

Severity (z)	Variable	a	b	c	d	e
Fatal, incapacitating injury, or non-incapacitating injury (KAB)	V_{KAB}	0.368	-0.0639	-0.760	-0.605	0.594

For three-leg intersections with signal control on rural two-lane and multilane highways, values of 0.0259 and 0.159 are used for $P_{K|KAB}$ and $P_{A|KAB}$, respectively.

4.6 Summary of Recommended Models for Incorporation in the HSM

In summary, several crash prediction models were developed for three-leg intersections with signal control on rural two-lane and multilane highways for consideration in the second edition of the HSM, including models for:

- Three-leg intersections with signal control (3SG) on rural two-lane highways
- Three-leg intersections with signal control (3SG) on rural multilane highways

The final predictive model for estimating total crashes at three-leg intersections with signal control on rural two-lane highways presented in Table 24 and the final predictive models for estimating total and FI crashes at three-leg intersections with signal control on multilane highways presented in Table 25 are recommended for inclusion in the second edition of the HSM, consistent with existing methods in HSM Chapters 10 and 11.

Logical interpretations do exist for the SDFs reported in Section 4.5. For example, the negative parameter for AADT may be capturing lower (on average) impact speeds of crashes at locations with higher traffic volumes. The negative parameter for number of left-turn lanes on the major road approaches may be capturing the severity impacts of separating through and turning vehicles with high-speed differentials. The positive parameter for number of through lanes may be capturing a compound effect of higher operating speeds plus longer crossing distances resulting in more severe crashes. These interpretations cannot, however, be verified or validated with existing crash databases.

Additionally, the types of severity effects found for three-leg intersections with signal control on rural two-lane and multilane highways were not consistently found across other intersection types. More generally, uncovering consistent and statistically significant impacts of intersection characteristics on crash severity proved challenging. These challenges were also implied by the SDF results of NCHRP Project 17-45, where only area type (urban, rural) and presence of protected left-turn phasing were included in crossroad ramp terminal SDFs. With these challenges in mind, ongoing and future research efforts will continue to explore the most promising approaches for addressing crash severity in the HSM predictive methods.

Due to the challenges and inconsistencies observed to-date in developing SDFs for three-leg intersections with signal control on rural two-lane and multilane highways, it is recommended for the second edition of the HSM that crash severity for three-leg intersections with signal control on rural two-lane and multilane highways be addressed in a manner consistent with existing methods in Chapters 10 and 11 of the HSM, respectively, without use of SDFs.

Appendix A presents recommended text for incorporating the final recommended models for three-leg intersections with signal control on rural two-lane and multilane highways into Chapters 10 and 11 of the HSM.

Chapter 5. Development of Models for Use in HSM Crash Prediction Methods: Intersections on High-Speed Urban and Suburban Arterials

This section of the report describes the development of crash prediction models for intersections on high-speed urban and suburban arterials and presents the final models recommended for incorporation in the second edition of the HSM. The HSM Part C chapters in the first edition of the HSM include crash prediction models for minor approach stop control and signalized intersections; however, the intersections used in the modeling process were generally not located on high-speed facilities. For the development of crash prediction models in this effort, intersections were only selected if they were located on arterials with a speed limit of at least 50 mph. Crash prediction models are recommended for the following intersection types for the second edition of the HSM:

- Three-leg intersections with minor road stop control (3ST) on high-speed urban and suburban arterials
- Three-leg intersections with signal control (3SG) on high-speed urban and suburban arterials
- Four-leg intersections with minor road stop control (4ST) on high-speed urban and suburban arterials
- Four-leg intersections with signal control (4SG) on high-speed urban and suburban arterials

Section 5.1 describes the site selection and data collection process for developing the crash prediction models for intersections on high-speed urban and suburban arterials. Section 5.2 presents descriptive statistics of the databases used for model development. Section 5.3 presents the statistical analysis and SPFs developed for intersections on high-speed urban and suburban arterials. Section 5.4 presents the CMFs recommended for use with the SPFs. Section 5.5 presents the results of an analysis to develop SDFs for use with the SPFs for intersections on high-speed urban and suburban arterials, and Section 5.6 summarizes the recommendations for incorporating new crash prediction models for intersections located on high-speed urban and suburban arterials in the second edition of the HSM.

5.1 Site Selection and Data Collection

A list of potential intersections for model development was derived from HSIS or Safety Analyst databases in four states:

- California (CA)
- Illinois (IL)
- Minnesota (MN)
- Washington (WA)

Each intersection in the list was initially screened using Google Earth® to determine if the site was suitable for inclusion in model development. Several reasons a site could be deemed inappropriate for use in model development were:

- The traffic control at the intersection was something other than signal control or minor approach stop control
- The speed limit on the major road was less than 50 mph
- The intersection was in a rural area
- A private driveway was located in close proximity to the intersection
- One or more of the approaches to the intersection was a private/commercial access
- Google Street View® was not available to identify leg specific attributes
- One or more of the intersection legs was a one-way street

Each intersection that was initially deemed appropriate for inclusion in model development was given a unique identification code and included on a refined database for detailed data collection.

Three types of data were collected for each intersection during detailed data collection: site characteristic, crash, and traffic volume data. Google Earth® was used to collect detailed site characteristics of the intersections. To reduce potential errors during data collection and to streamline data entry, a data collection tool was created using Visual Basic for Applications, very similar to the tool shown in Figure 4. Table 33 lists all the intersection attributes collected (and respective definitions and permitted values) for intersections on high-speed arterials on urban and suburban arterials using the data collection tool. The data collection tool dynamically changed as certain intersection configuration data were entered because some data elements were only applicable to signalized intersections or minor approach stop-controlled intersections. Once all necessary data were entered into the data collection tool and saved for a given intersection, the data collection tool was used to validate the inputs for that particular intersection consistent with the range and/or permitted values for the respective variables/parameters.

Table 33. Site characteristic variables collected for intersections on high-speed urban and suburban arterials

Variable/Parameter	Definition	Range or Permitted Values
General Intersection Attributes		
Intersection configuration (i.e., number of legs and type of traffic control)	Indicates the number of legs and type of traffic control	3ST, 4ST, 3SG, 4SG
Area type (urban/rural)	Indicates whether the intersection is in a rural or urban area	Urban
Presence of intersection lighting	Indicates if overhead lighting is present at the intersection proper	Yes, no
Presence of flashing beacons	Indicates if overhead flashing beacons are present at the intersection proper	Yes, no (only applicable to stop-controlled intersections)
Approach Specific Attributes		
Route name or number	Specify the route name or number of the approach	
Location at intersection	Side/quadrant of the intersection the approach is located	N, S, E, W, NE, NW, SE, SW

Table 33. Site characteristic variables collected for intersections on high-speed urban and suburban arterials (Continued)

Variable/Parameter	Definition	Range or Permitted Values
Number of through lanes	This includes dedicated through lanes and any lanes with shared movements. On the minor approach of a 3-leg intersection, if there is only one lane, then it should be classified as a through lane	0, 1, 2, 3
Presence/number of left-turn lanes	The number of lanes in which only a left-turn movement can be made	0, 1, 2, 3
Left-turn channelization	Type of left-turn channelization used on the intersection approach	Raised or depressed island, painted, none
Presence/number of right-turn lanes	The number of lanes in which only a right-turn movement can be made	0, 1, 2, 3
Right-turn channelization	Type of right-turn channelization used on the intersection approach	Raised or depressed island, painted, none
Median width	Measured from outside of outer most through lane of approaching lanes to outside of lane in opposing direction	Values in feet
Median type	Type of median separating opposing directions of travel	Raised, depressed, flush, barrier, TWLTL
Permit right-turn-on-red	Indicates if turning right on red is permitted on the intersection approach	Yes, no, not applicable (only applicable to signalized intersections)
Presence of transverse rumble strips	Indicates the presence of transverse rumble strips on the intersection approach	Yes, no, unknown
Presence/type of supplementary pavement markings	Indicates the presence and type of supplementary pavement markings on the intersection approach	Yes, no, unknown If yes and intersection is signalized, type of marking: “Signal Ahead”, other. If yes and intersection is stop-controlled, type of marking: “Stop Ahead”, other.
Presence of stop ahead warning signs	Indicates the presence of stop ahead warning signs on the intersection approach	Yes, no, unknown (only applicable to stop-controlled intersections)
Presence of signal ahead warning signs	Indicates the presence of signal ahead warning signs on the intersection approach	Yes, no, unknown (only applicable to signalized intersections)
Presence of advance warning flashers	Indicates the presence of advance warning flashers on the intersection approach	Yes, no, unknown
Horizontal alignment of intersection approach	Indicates whether the approaching roadway, within 250 ft of the intersection, is a tangent or curved section of roadway	Tangent, curve
Horizontal curve radius	Indicates the radius of the curve on the intersection approach if a curve is present within 250 ft of the intersection	2,000-ft Maximum Range: 25-2000 ft
Posted speed limit	Posted speed limit on the intersection approach (mph)	15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, unknown
Presence of crosswalk	Indicates the presence of a crosswalk perpendicular to the intersection approach	Yes, no, unknown
Presence of bike lane	Indicates the presence of a marked bike lane parallel to the intersection approach	Yes, no, unknown
Presence of railroad crossing	Indicates the presence of a railroad crossing on the intersection approach within 250 ft of the intersection	Yes, no, unknown
Skew	Indicates the difference between 90 degrees and the actual angle between the approach and the major road	0-90 degrees (only applicable to minor road approaches at stop-controlled intersections)

During detailed data collection, to the extent possible, the research team reviewed historical aerial images to determine if a site had recently been under construction or recent improvements were made to the site to determine the appropriate years of data for use in model development.

Table 34 lists the crash and traffic volume data sources for the four states included in the study. The goal was to obtain the most recent four to six years of crash and traffic volume data for each site for model development. All of the data (i.e., site characteristics, crash, and traffic volume) were assembled into one database for the purposes of model development.

Table 34. Traffic volume and crash data sources for intersections on high-speed urban and suburban arterials

State	Traffic Volume Data Source	Crash Data Source
California	HSIS	HSIS
Illinois	Safety Analyst	Safety Analyst
Minnesota	HSIS	State agency
Washington	HSIS and Safety Analyst	HSIS

5.2 Descriptive Statistics of Database

A total of 504 sites—121 three-leg stop-controlled, 50 three-leg signalized, 125 four-leg stop-controlled, and 208 four-leg signalized intersections—were available for development of crash prediction models. The data collections sites were located in four states—California, Illinois, Minnesota, and Washington. To remain consistent with the standards for development of the intersection predictive models in the first edition of the HSM, the goal of this research was to develop crash prediction models with a minimum of 200 site-years of data, and preferably 450 site-years of data or more.

5.2.1 Traffic Volumes and Site Characteristics

Traffic volume and crash data were available for a 5-year period. Table 35 shows the breakdown of all sites by area type and intersection type. Study period (date range), number of sites and site-years, and basic traffic volume statistics are shown by state in each category and across all states within a category.

Of the intersection characteristics collected in Google Earth® (see Table 33), many showed no or very little variability (i.e., most intersections were predominantly of one type for a specific variable) across sites within a category and were thus excluded from modeling. The remaining variables (percent of "Yes" by type of traffic control indicated in parentheses) of potential interest in modeling were:

- presence of intersection lighting: (stop-controlled: 59%; signalized: 97%)
- presence of left-turn lanes on major road (stop-controlled: 67%; signalized: 96%)
- presence of right-turn lanes on major road (stop-controlled: 48%; signalized: 93%)

The use of these site characteristics is discussed later in the SPF model development section.

5.2.2 Crash Counts

Of the 504 intersections included in the study, only 18 (3.6%) experienced no crashes over the entire 5-year study period; the breakdown by area type and intersection type is as follows:

- Three-leg intersections with stop control: 10 out of 121
- Three-leg intersections with signal control: 1 out of 50
- Four-leg intersections with stop control: 6 out of 125
- Four-leg intersections signal control: 1 out of 208

Intersection crashes were defined as those crashes that occurred within 250 ft of the intersection and were classified as at intersection or intersection-related, consistent with recommended practice in the HSM for assigning crashes to an intersection.

Table 36 (three-leg intersections) and Table 37 (four-leg intersections) show all crashes combined, single- and MV, and pedestrian and bicycle crash counts over the study period for each state by intersection type.

Crash counts are also tallied by collision type and manner of collision across all states, separately by intersection type, in Table 38 (three-leg intersections) and Table 39 (four-leg intersections).

Table 35. Major- and minor road AADT statistics by intersection type on high-speed urban and suburban arterials

State	Date Range	Number of Sites	Number of Site-Years	Major Road AADT (veh/day)				Minor Road AADT (veh/day)			
				Min	Max	Mean	Median	Min	Max	Mean	Median
URBAN THREE-LEG STOP-CONTROLLED INTERSECTIONS (3ST)											
CA	2007-2011	47	229	2,631	58,494	18,077	16,289	21	5,901	622	291
IL	2008-2012	27	121	1,450	18,200	6,625	4,750	93	7,900	1,516	950
MN	2010-2014	47	217	7,764	39,000	15,261	13,820	105	11,335	1,844	1,152
All States	**2007-2014**	**121**	**567**	**1,450**	**58,494**	**14,428**	**12,225**	**21**	**11,335**	**1,296**	**675**
URBAN THREE-LEG SIGNALIZED INTERSECTIONS (3SG)											
CA	2007-2011	18	78	7,058	55,000	22,500	16,500	40	29,800	3,672	860
IL	2008-2012	9	44	4,200	10,780	8,231	8,500	200	6,800	3,050	2,350
MN	2010-2014	16	79	9,670	59,000	23,119	18,999	409	10,925	4,784	4,137
WA	2007-2011	7	35	18,089	28,952	21,832	20,487	1,875	16,000	7,529	6,863
All States	**2007-2014**	50	236	4,200	59,000	20,036	16,395	40	29,800	4,456	2,860
URBAN FOUR-LEG STOP-CONTROLLED INTERSECTIONS (4ST)											
CA	2007-2011	50	236	2,765	30,525	12,575	12,354	20	9,100	1,037	391
IL	2008-2012	31	134	1,350	12,900	5,713	4,600	12	3,750	1,146	1,040
MN	2010-2014	44	206	7,000	47,200	15,864	12,940	170	11,282	1,996	1,466
All States	**2007-2014**	125	576	1,350	47,200	12,023	10,491	12	11,282	1,402	950
URBAN FOUR-LEG SIGNALIZED INTERSECTIONS (4SG)											
CA	2007-2011	47	215	6,269	51,600	21,502	19,341	141	15,200	4,050	2,295
IL	2008-2012	62	296	4,100	22,221	10,422	9,350	800	15,300	4,383	3,750
MN	2010-2014	89	425	3,104	59,800	25,117	21,600	202	24,028	7,521	6,420
WA	2007-2011	10	50	10,841	36,802	23,694	24,325	708	30,029	11,225	9,176
All States	**2007-2014**	208	986	3,104	59,800	19,851	17,410	141	30,029	5,979	4,994

Table 36. All crashes combined, single- and MV, and pedestrian and bicycle crash counts by crash severity and intersection type for three-leg intersections on high-speed urban and suburban arterials

State	Date Range	Number of Sites	Number of Site Years	Time of Day	All Crashes Combined			Single-Vehicle Crashes			Multiple-Vehicle Crashes			Pedestrian Crashes	Bicycle Crashes
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO	FI	FI
URBAN THREE-LEG STOP-CONTROLLED INTERSECTIONS (3ST)															
CA	2007-2011	47	229	All	212	84	125	44	14	30	165	70	95	3	0
				Night	59	16	42	22	3	19	36	13	23	1	0
IL	2008-2012	27	121	All	185	67	118	10	1	9	175	66	109	0	0
				Night	24	6	18	5	1	4	19	5	14	0	0
MN	2010-2014	47	217	All	312	114	198	102	28	74	210	86	124	0	0
				Night	88	27	61	47	13	34	41	14	27	0	0
All States	**2007-2014**	**121**	**567**	**All**	**709**	**265**	**441**	**156**	**43**	**113**	**550**	**222**	**328**	**3**	**0**
				Night	**171**	**49**	**121**	**74**	**17**	**57**	**96**	**32**	**64**	**1**	**0**
URBAN THREE-LEG SIGNALIZED INTERSECTIONS (3SG)															
CA	2007-2011	18	78	All	205	93	111	29	8	21	175	85	90	1	0
				Night	51	23	27	15	4	11	35	19	16	1	0
IL	2008-2012	9	44	All	166	54	111	13	6	7	152	48	104	0	1
				Night	36	17	19	7	4	3	29	13	16	0	0
MN	2010-2014	16	79	All	364	109	255	44	8	36	320	101	219	0	0
				Night	71	23	48	17	2	15	54	21	33	0	0
WA	2007-2011	7	35	All	138	49	88	16	7	9	121	42	79	1	0
				Night	38	16	22	10	5	5	28	11	17	0	0
All States	**2007-2014**	50	236	**All**	**873**	**305**	**565**	**102**	**29**	**73**	**768**	**276**	**492**	**2**	**1**
				Night	**196**	**79**	**116**	**49**	**15**	**34**	**146**	**64**	**82**	**1**	**0**

Table 37. All crashes combined, single- and MV, and pedestrian and bicycle crash counts by crash severity and intersection type for four-leg intersections on high-speed urban and suburban arterials

State	Date Range	Number of Sites	Number of Site Years	Time of Day	All Crashes Combined			Single-Vehicle Crashes			Multiple-Vehicle Crashes			Pedestrian Crashes	Bicycle Crashes
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO	FI	FI
URBAN FOUR-LEG STOP-CONTROLLED INTERSECTIONS (4ST)															
CA	2007-2011	50	236	All	250	101	145	28	6	22	218	95	123	4	0
				Night	68	28	38	11	2	9	55	26	29	2	0
IL	2008-2012	31	134	All	276	109	167	20	6	14	256	103	153	0	0
				Night	52	19	33	9	1	8	43	18	25	0	0
MN	2010-2014	44	206	All	464	190	274	105	34	71	359	156	203	0	0
				Night	111	36	75	55	13	42	56	23	33	0	0
All States	**2007-2014**	125	576	**All**	**990**	**400**	**586**	**153**	**46**	**107**	**833**	**354**	**479**	**4**	**0**
				Night	**231**	**83**	**146**	**75**	**16**	**59**	**154**	**67**	**87**	**2**	**0**
URBAN FOUR-LEG SIGNALIZED INTERSECTIONS (4SG)															
CA	2007-2011	47	215	All	1,022	414	600	93	28	65	921	386	535	8	0
				Night	259	109	145	43	13	30	211	96	115	5	0
IL	2008-2012	62	296	All	2,292	661	1,621	112	25	87	2,170	636	1,534	7	3
				Night	513	153	356	63	9	54	446	144	302	3	1
MN	2010-2014	89	425	All	3,050	980	2,070	241	91	150	2,809	889	1,920	0	0
				Night	601	209	392	95	30	65	506	179	327	0	0
WA	2007-2011	10	50	All	383	131	247	31	6	25	347	125	222	3	2
				Night	86	32	51	13	2	11	70	30	40	3	0
All States	**2007-2014**	208	986	**All**	**6,747**	**2,186**	**4,538**	**477**	**150**	**327**	**6,247**	**2,036**	**4,211**	**18**	**5**
				Night	**1,459**	**503**	**944**	**214**	**54**	**160**	**1,233**	**449**	**784**	**11**	**1**

Table 38. Crash counts by collision type and manner of collision, crash severity, and intersection type at three-leg intersections on high-speed urban and suburban arterials

Collision Type	Three-Leg Stop-Controlled Intersections (3ST)			Three-Leg Signalized Intersections (3SG)		
	Total	FI	PDO	Total	FI	PDO
SINGLE-VEHICLE CRASHES						
Collision with parked vehicle	0	0	0	0	0	0
Collision with animal	1	0	1	0	0	0
Collision with fixed object	37	8	29	46	16	30
Collision with other object	0	0	0	0	0	0
Other SV collision	106	29	77	48	9	39
Noncollision	12	6	6	8	4	4
Total SV crashes[a]	**156**	**43**	**113**	**102**	**29**	**73**
MULTIPLE-VEHICLE CRASHES						
Rear-end collision	239	80	159	494	175	319
Head-on collision	12	5	7	10	8	2
Angle collision	209	105	104	145	64	81
Sideswipe collision	54	17	37	66	10	56
Other MV collisions	36	15	21	53	19	34
Total MV crashes[a]	550	222	328	768	276	492
Total Crashes[a]	706	265	441	870	305	565

[a] Note crash counts do not include pedestrian and bicycle crashes.

Table 39. Crash counts by collision type and manner of collision, crash severity, and intersection type at four-leg intersections on high-speed urban and suburban arterials

Collision Type	Four-Leg Stop-Controlled Intersections (4ST)			Four-Leg Signalized Intersections (4SG)		
	Total	FI	PDO	Total	FI	PDO
SINGLE-VEHICLE CRASHES						
Collision with parked vehicle	0	0	0	0	0	0
Collision with animal	2	0	2	1	0	1
Collision with fixed object	31	5	26	174	36	138
Collision with other object	2	0	2	2	0	2
Other SV collision	109	34	75	263	93	170
Noncollision	9	7	2	37	21	16
Total SV crashes[a]	**153**	**46**	**107**	**477**	**150**	**327**
MULTIPLE-VEHICLE CRASHES						
Rear-end collision	210	59	151	3,906	1,141	2,765
Head-on collision	27	18	9	95	57	38
Angle collision	435	231	204	1,359	617	742
Sideswipe collision	78	10	68	449	53	396
Other MV collisions	83	36	47	438	168	270
Total MV crashes[a]	833	354	479	6,247	2,036	4,211
Total Crashes[a]	986	400	586	6,724	2,186	4,538

[a] Note crash counts do not include pedestrian and bicycle crashes.

5.3 Safety Performance Functions—Model Development

SPFs of the form shown in Equation 2 were developed separately for three- and four-leg intersections, for multiple- and SV crashes.

$$N_{spf\ int} = exp[a + b \times \ln(AADT_{maj}) + c \times \ln(AADT_{min})] \quad \text{(Eq. 2)}$$

Where:

$N_{spf\ int}$ = predicted average crash frequency for an intersection with base conditions (crashes/year)
$AADT_{maj}$ = AADT on the major road (veh/day)
$AADT_{min}$ = AADT on the minor road (veh/day)
a, b, and c = estimated regression coefficients

For intersections on high-speed urban and suburban arterials, the SPFs were developed consistent with the methodology in Chapter 12 of the HSM for predicting intersections crashes in urban and suburban areas as illustrated in Equation 4 and Equation 5.

$$N_{predicted\ int} = C_i \times (N_{bi} + N_{pedi} + N_{bikei}) \quad \text{(Eq. 4)}$$

$$N_{bi} = N_{spf\ int} \times (CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{yi}) \quad \text{(Eq. 5)}$$

Where:

$N_{predicted\ int}$ = predicted average crash frequency for an individual intersection for the selected year (crashes/year)
N_{bi} = predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle crashes) (crashes/year)
N_{pedi} = predicted average crash frequency of vehicle-pedestrian crashes of an intersection (crashes/year)
N_{bikei} = predicted average crash frequency of vehicle-bicycle crashes of an intersection (crashes/year)
$N_{spf\ int}$ = predicted total average crash frequency of intersection-related crashes for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions) (crashes/year)
$CMF_{1i} \ldots CMF_{yi}$ = crash modification factors specific to intersection type *i* and specific geometric design and traffic control features *y*
C_i = calibration factor to adjust the SPF for intersection type *i* to local conditions

The SPF portion of N_{bi}, $N_{spf\ int}$, is the sum of two more disaggregate predictions by collision type, as shown in Equation 6.

$$N_{spf\ int} = N_{bimv} + N_{bisv} \quad \text{(Eq. 6)}$$

Where:

N_{bimv} = predicted average crash frequency of MV crashes of an intersection for base conditions (crashes/year)
N_{bisv} = predicted average crash frequency of SV crashes of an intersection for base conditions (crashes/year)

Separate model structures are used to estimate the yearly number of vehicle-pedestrian (N_{pedi}) and vehicle-bicycle (N_{bikei}) crashes at stop-controlled and signalized intersections on high-speed urban and suburban arterials. The average number of annual vehicle-pedestrian and vehicle-bicycle crashes are estimated with Equations 9 and 12, respectively.

$$N_{pedi} = N_{bi} \times f_{pedi} \qquad \text{(Eq. 9)}$$

Where:

f_{pedi} = pedestrian crash adjustment factor for intersection type *i*

$$N_{bikei} = N_{bi} \times f_{bikei} \qquad \text{(Eq. 12)}$$

Where:

f_{bikei} = bicycle crash adjustment factor for intersection type *i*

All of the vehicle-pedestrian and vehicle-bicycle crashes predicted with Equations 9 and 12 are assumed to be FI crashes (none as PDO).

Based on a review of the number of states, sites, site-years, and crashes for the database assembled, data for all sites were used for model development to maximize the sample size rather than using a portion of the data for model development and a portion for model validation.

All SPFs were developed using a NB regression model based on all sites combined within a given area type and intersection type. In all models, state was included as a random blocking effect, with sites nested within their respective state. A significance level of 0.20 for inclusion in a model was selected for an individual parameter. This was based on previous models included in the first edition of the HSM (Harwood et al., 2007). PROC GLIMMIX of SAS 9.3 was used for all modeling (SAS). Models were developed for total, FI, and PDO crashes, separately for multiple- and SV crashes.

In general, the base conditions for intersection models in Chapter 12 of the HSM are the absence of intersection lighting and that of left- and right-turn lanes. A very small number of intersections in the database for model development met all three requirements (three intersections on two-lane and one on multilane highways). The distribution of intersections by the three characteristics is as follows:

- Intersections on two-lane highways: 72 lighted; 17 unlighted (19% unlighted)
- Intersections on multilane highways: 53 lighted; 19 unlighted (26% unlighted)

- Intersections on two-lane highways: 58 with right-turn lane on one approach; 31 with none (35% with none)
- Intersections on multilane highways: 48 with right-turn lane on one approach; 24 with none (33% with none)
- Intersections on two-lane highways: 79 with left-turn lane on one approach; 10 with none (11% with none)
- Intersections on multilane highways: 66 with left-turn lane on one approach; 1 with left-turn lane on two approaches; 5 with none (7% with none)

To include all intersections in the models, crashes at intersections that did not meet base conditions for these three characteristics were first adjusted using the following CMFs in reverse (i.e., divide rather than multiply the crashes by the product of the CMFs):

- Lighting: use CMF_i, shown in Equation 12-36 in the HSM and the proportion of total crashes for unlighted intersections that occurred at night in the current database, shown in Table 40 (similar to Table 12-27 in the HSM); this CMF was applied to total, FI, and PDO crashes before modeling
- Installation of left-turn lanes: use CMF_i shown in Table 52 (see Table 12-24 in the HSM)
- Installation of right-turn lanes: use CMF_i shown in Table 53 (see Table 12-26 in the HSM)

Table 40. Nighttime crash counts and proportions for unlighted intersections on high-speed urban and suburban arterials used for modeling

Intersection Type	Number of Sites[a]	Number of Nighttime Crashes	Total Crashes	Proportion of Crashes that Occurred at Night (p_{ni})
Three-Leg Stop-Controlled Intersections (3ST)	53	75	258	**0.291**
Three-Leg Signalized Intersections (3SG)	1	13	63	**0.206**
Four-Leg Stop-Controlled Intersections (4ST)	47	91	356	**0.256**
Four-Leg Signalized Intersections (4SG)	6	39	159	**0.245**
Three- and Four-Leg Signalized Intersections (3SG and 4SG)	7	52	222	**0.234**

[a] Number of unlighted intersections only.

The final SPF models for crashes at intersections on high-speed urban and suburban arterials are shown, separately for each intersection type, in the following order:

- Table 41: MV total crashes
- Table 42: MV FI crashes
- Table 43: MV PDO crashes

- Table 44: SV total crashes
- Table 45: SV FI crashes
- Table 46: SV PDO crashes

Each table shows the model coefficients and overdispersion parameter (estimate), their standard error, and associated p-values (or significance level) for each severity level. Figures 21-44 graphically present the SPFs shown in Tables 41-46 for various major- and minor road AADTs.

Both major- and minor road AADT coefficients are significant at the 80-percent level or better in all MV crash models. However, for the SV crash models, major- and minor road AADT coefficients were not always significant at the 80-percent level or better. For completeness, these models are provided and are considered the most reasonable models for estimating SV crashes at intersections on high-speed urban and suburban arterials.

Table 41. SPF coefficients for intersections on high-speed urban and suburban arterials—MV total crashes

Intersection Type	Parameter	Estimate	Standard Error	Pr > F	Significance Level
MULTIPLE-VEHICLE TOTAL CRASHES					
Three-Leg Stop-Controlled Intersections (3ST)	Intercept	-8.26	2.56	--	--
	$\ln(AADT_{maj})$	0.58	0.25	0.03	Significant at 95% level
	$\ln(AADT_{min})$	0.49	0.11	<.01	Significant at 99% level
	Overdispersion	0.85	0.18	--	--
Three-Leg Signalized Intersections (3SG)	Intercept	-4.41	1.31	.	
	$\ln(AADT_{maj})$	0.43	0.14	<.01	Significant at 99% level
	$\ln(AADT_{min})$	0.19	0.06	<.01	Significant at 99% level
	Overdispersion	0.21	0.06	--	--
Four-Leg Stop-Controlled Intersections (4ST)	Intercept	-5.78	2.09	--	--
	$\ln(AADT_{maj})$	0.48	0.21	0.02	Significant at 95% level
	$\ln(AADT_{min})$	0.36	0.10	<.01	Significant at 99% level
	Overdispersion	0.91	0.17	--	--
Four-Leg Signalized Intersections (4SG)	Intercept	-9.65	0.92	--	--
	$\ln(AADT_{maj})$	0.98	0.08	<.01	Significant at 99% level
	$\ln(AADT_{min})$	0.28	0.05	<.01	Significant at 99% level
	Overdispersion	0.31	0.03	--	--

Base Condition: Absence of intersection lighting; for stop control intersections, absence of turn lanes on non-stop control approaches; for signal control intersections, absence of turn lanes on all intersection approaches

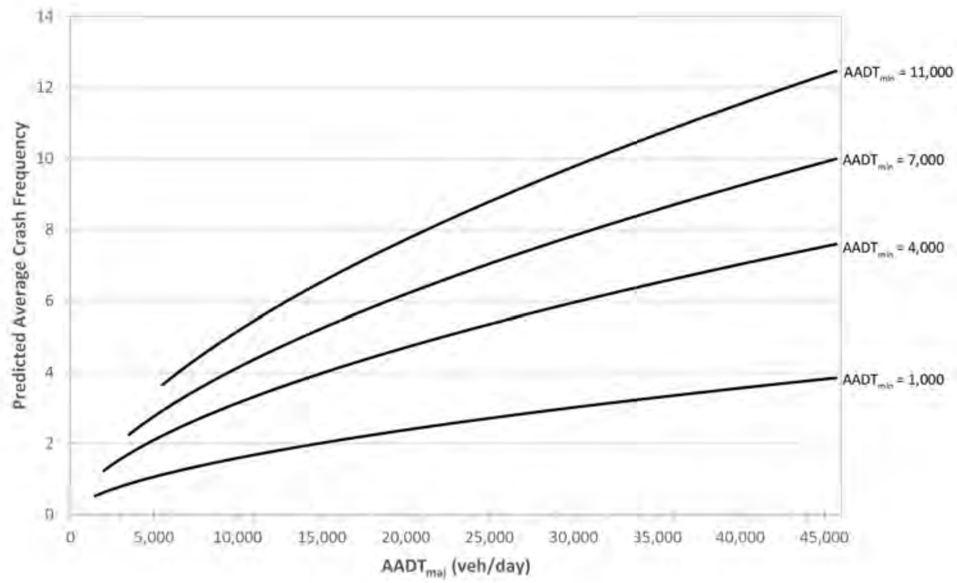


Figure 21. Graphical representation of the SPF for MV total crashes at three-leg stop-controlled intersections on high-speed urban and suburban arterials

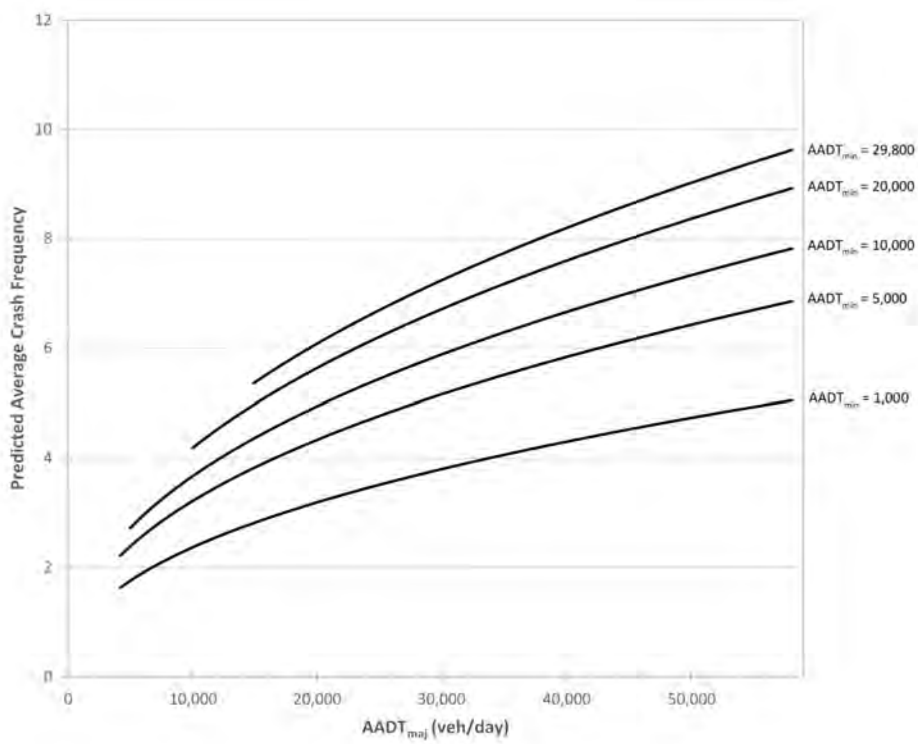


Figure 22. Graphical representation of the SPF for MV total crashes at three-leg signalized intersections on high-speed urban and suburban arterials

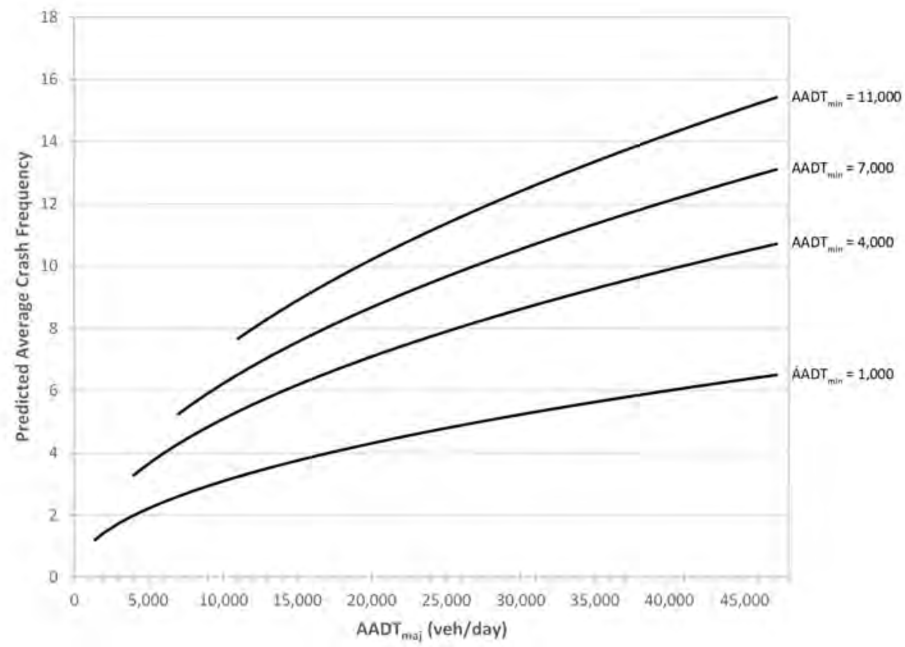


Figure 23. Graphical representation of the SPF for MV total crashes at four-leg stop-controlled intersections on high-speed urban and suburban arterials

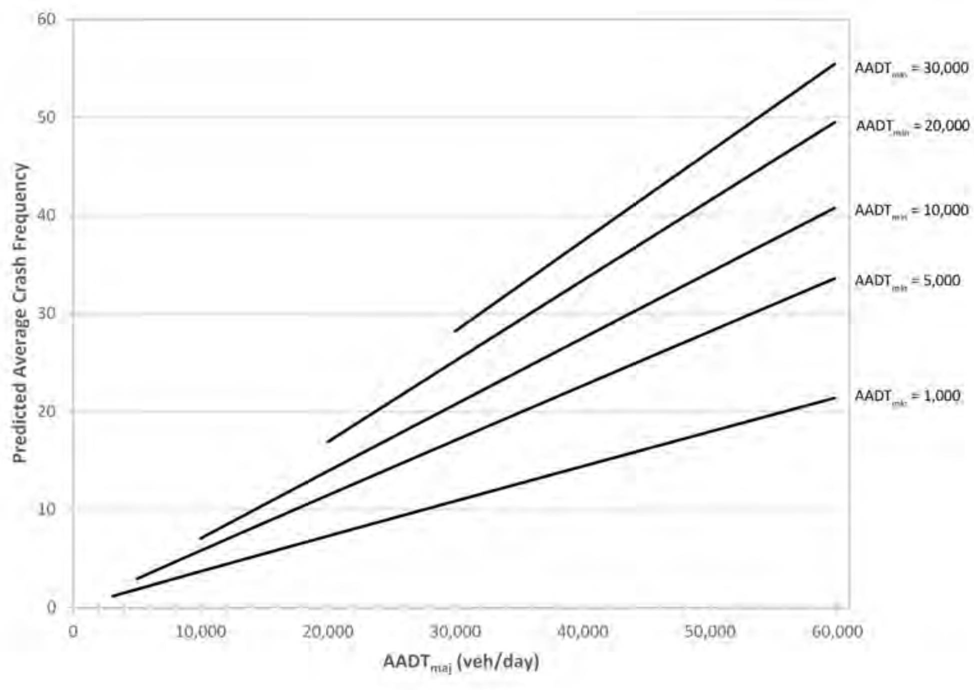


Figure 24. Graphical representation of the SPF for MV total crashes at four-leg signalized intersections on high-speed urban and suburban arterials

Table 42. SPF coefficients for intersections on high-speed urban and suburban arterials—MV FI crashes

Intersection Type	Parameter	Estimate	Standard Error	Pr > F	Significance Level?
MULTIPLE-VEHICLE FI CRASHES					
Three-Leg Stop-Controlled Intersections (3ST)	Intercept	-6.84	2.71	--	--
	$\ln(AADT_{maj})$	0.40	0.27	0.14	Significant at 85% level
	$\ln(AADT_{min})$	0.38	0.12	<.01	Significant at 99% level
	Overdispersion	0.76	0.19	--	--
Three-Leg Signalized Intersections (3SG)	Intercept	-7.28	1.25	--	--
	$\ln(AADT_{maj})$	0.64	0.13	<.01	Significant at 99% level
	$\ln(AADT_{min})$	0.17	0.06	<.01	Significant at 99% level
	Overdispersion	0.09	0.05	--	--
Four-Leg Stop-Controlled Intersections (4ST)	Intercept	-7.93	2.23	--	--
	$\ln(AADT_{maj})$	0.55	0.22	0.01	Significant at 99% level
	$\ln(AADT_{min})$	0.45	0.11	<.01	Significant at 99% level
	Overdispersion	0.89	0.18	--	--
Four-Leg Signalized Intersections (4SG)	Intercept	-9.61	0.97	--	--
	$\ln(AADT_{maj})$	0.86	0.09	<.01	Significant at 99% level
	$\ln(AADT_{min})$	0.29	0.05	<.01	Significant at 99% level
	Overdispersion	0.31	0.04	--	--

Base Condition: Absence of intersection lighting; for stop control intersections, absence of turn lanes on non-stop control approaches; for signal control intersections, absence of turn lanes on all intersection approaches

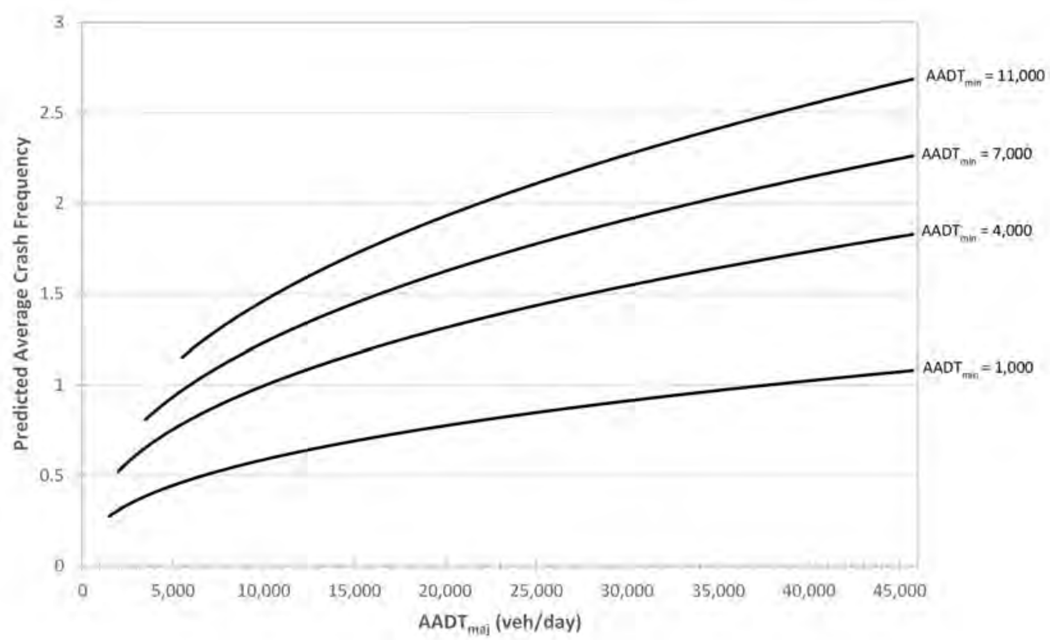


Figure 25. Graphical representation of the SPF for MV FI crashes at three-leg stop-controlled intersections on high-speed urban and suburban arterials

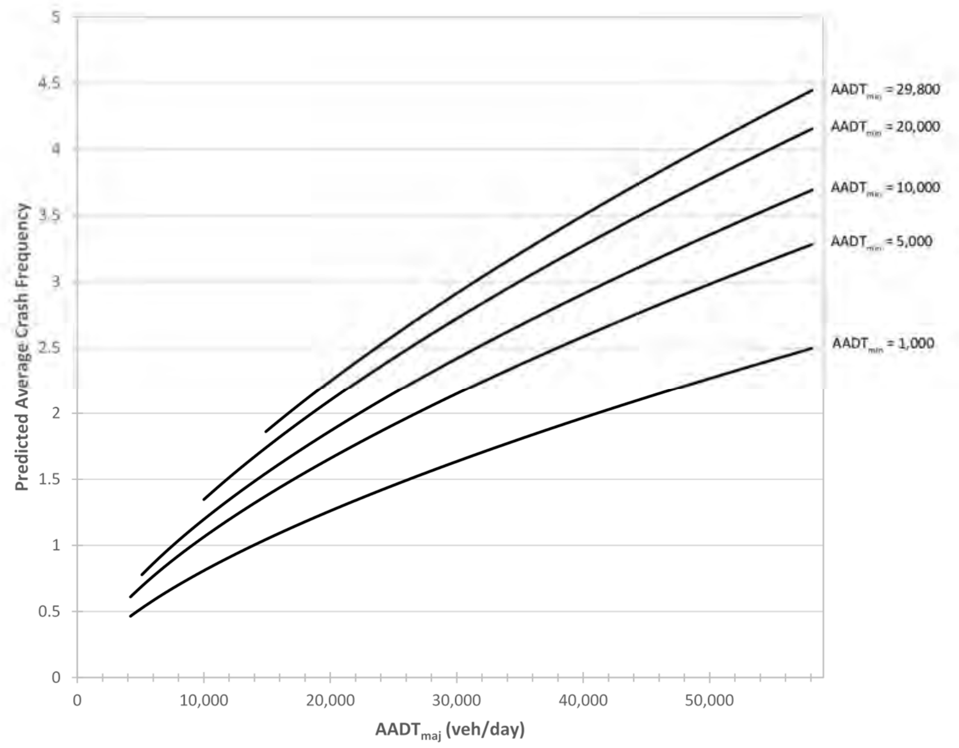


Figure 26. Graphical representation of the SPF for MV FI crashes at three-leg signalized intersections on high-speed urban and suburban arterials

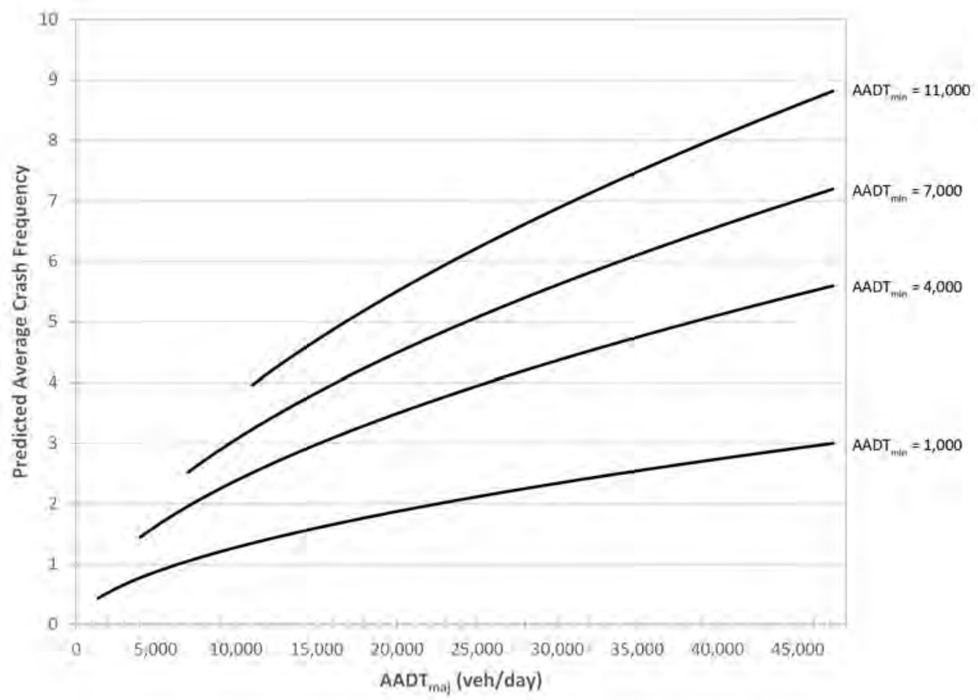


Figure 27. Graphical representation of the SPF for MV FI crashes at four-leg stop-controlled intersections on high-speed urban and suburban arterials

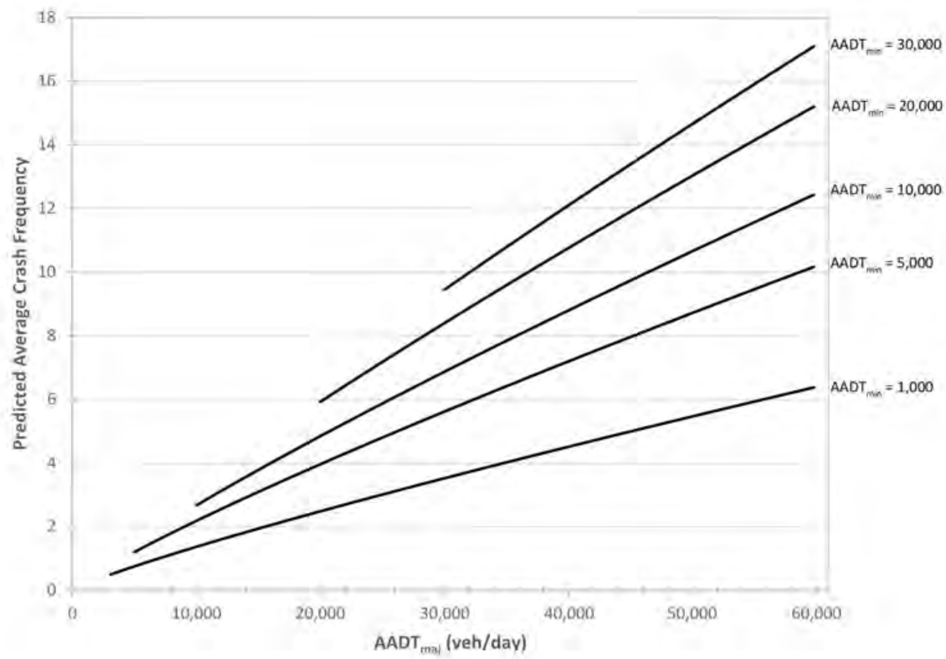


Figure 28. Graphical representation of the SPF for MV FI crashes at four-leg signalized intersections on high-speed urban and suburban arterials

Table 43. SPF coefficients for intersections on high-speed urban and suburban arterials—MV PDO crashes

Intersection Type	Parameter	Estimate	Standard Error	Pr > F	Significance Level?
MULTIPLE-VEHICLE PDO CRASHES					
Three-Leg Stop-Controlled Intersections (3ST)	Intercept	-9.89	3.00	--	--
	$\ln(AADT_{maj})$	0.65	0.29	0.03	Significant at 95% level
	$\ln(AADT_{min})$	0.56	0.14	<.01	Significant at 99% level
	Overdispersion	1.11	0.25	--	--
Three-Leg Signalized Intersections (3SG)	Intercept	-3.08	1.67	--	--
	$\ln(AADT_{maj})$	0.25	0.18	0.17	Significant at 80% level
	$\ln(AADT_{min})$	0.19	0.08	0.02	Significant at 95% level
	Overdispersion	0.34	0.10	--	--
Four-Leg Stop-Controlled Intersections (4ST)	Intercept	-5.46	2.14	--	--
	$\ln(AADT_{maj})$	0.42	0.21	0.05	Significant at 95% level
	$\ln(AADT_{min})$	0.32	0.10	<.01	Significant at 99% level
	Overdispersion	0.94	0.18	--	--
Four-Leg Signalized Intersections (4SG)	Intercept	-10.70	1.05	--	--
	$\ln(AADT_{maj})$	1.04	0.10	<.01	Significant at 99% level
	$\ln(AADT_{min})$	0.29	0.05	<.01	Significant at 99% level
	Overdispersion	0.38	0.04	--	--

Base Condition: Absence of intersection lighting; for stop control intersections, absence of turn lanes on non-stop control approaches; for signal control intersections, absence of turn lanes on all intersection approaches

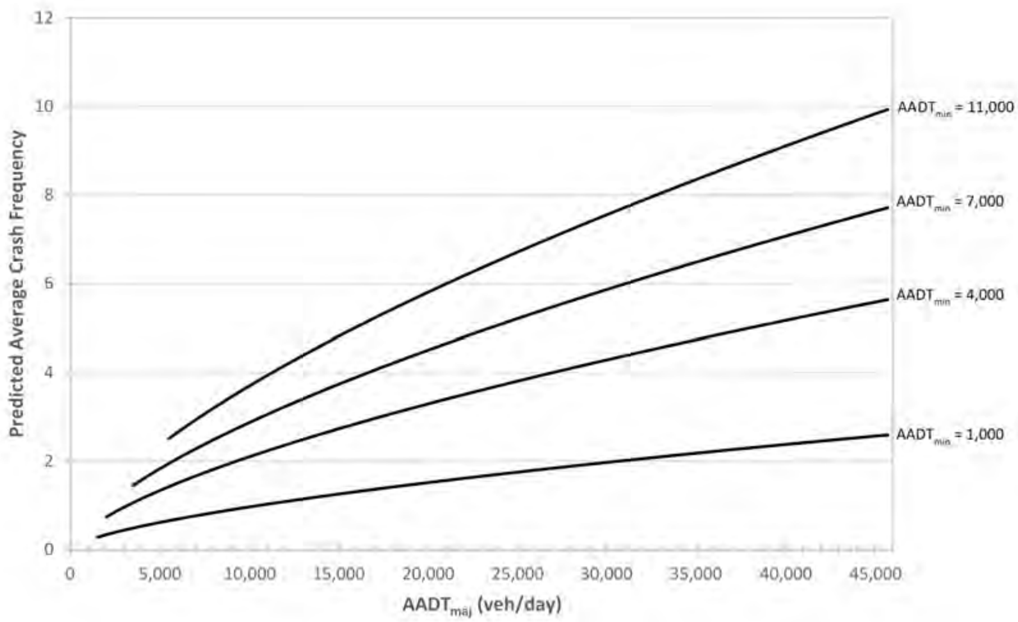


Figure 29. Graphical representation of the SPF for MV PDO crashes at three-leg stop-controlled intersections on high-speed urban and suburban arterials

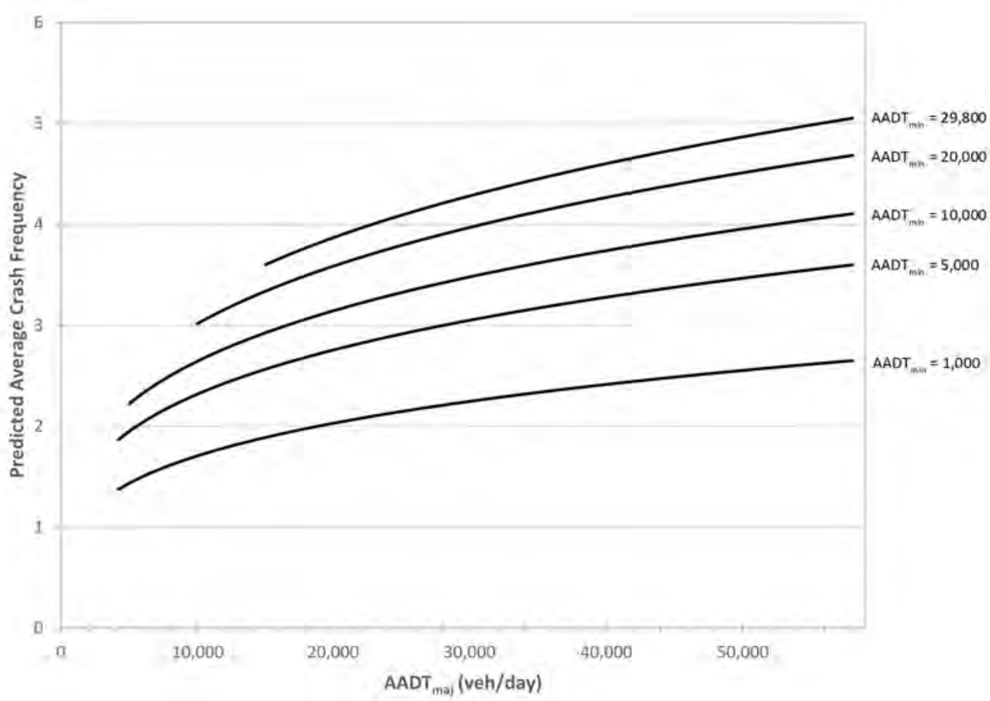


Figure 30. Graphical representation of the SPF for MV PDO crashes at three-leg signalized intersections on high-speed urban and suburban arterials

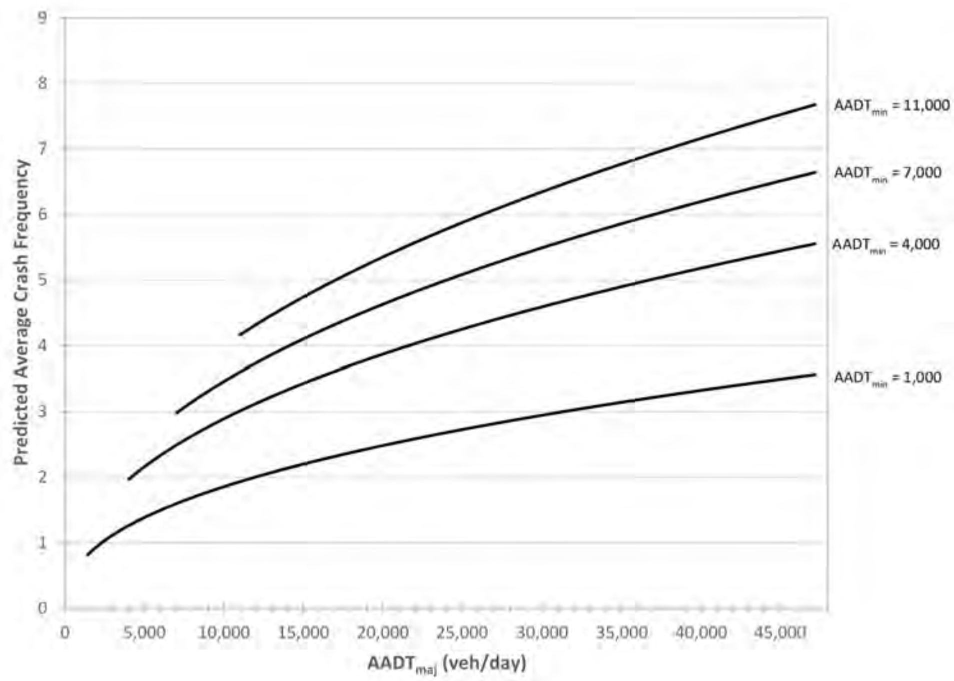


Figure 31. Graphical representation of the SPF for MV PDO crashes at four-leg stop-controlled intersections on high-speed urban and suburban arterials

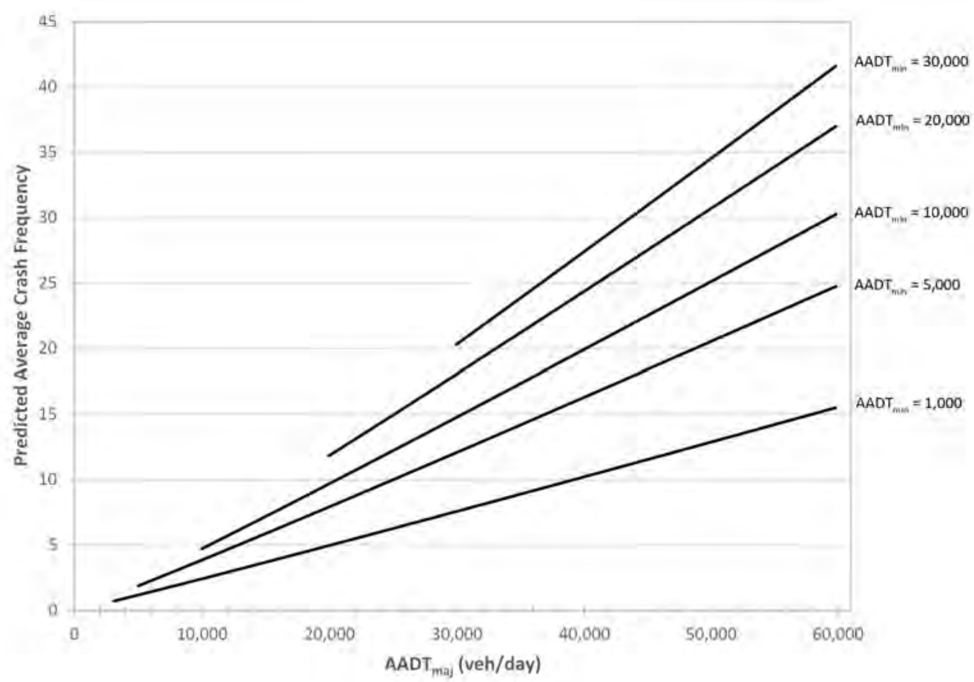


Figure 32. Graphical representation of the SPF for MV PDO crashes at four-leg signalized intersections on high-speed urban and suburban arterials

Table 44. SPF coefficients for intersections on high-speed urban and suburban arterials—SV total crashes

Intersection Type	Parameter	Estimate	Standard Error	Pr > F	Significance Level?
SV TOTAL CRASHES					
Three-Leg Stop-Controlled Intersections (3ST)	Intercept	-12.28	3.34	--	--
	$\ln(AADT_{maj})$	0.92	0.32	<.01	Significant at 99% level
	$\ln(AADT_{min})$	0.36	0.13	<.01	Significant at 99% level
	Overdispersion	0.69	0.22	--	--
Three-Leg Signalized Intersections (3SG)	Intercept	-6.77	2.31	--	--
	$\ln(AADT_{maj})$	0.60	0.24	0.02	Significant at 95% level
	$\ln(AADT_{min})$	0.04	0.11	0.73	Not significant
	Overdispersion	0.57	0.21	--	--
Four-Leg Stop-Controlled Intersections (4ST)	Intercept	-7.63	2.93	--	--
	$\ln(AADT_{maj})$	0.44	0.28	0.13	Significant at 85% level
	$\ln(AADT_{min})$	0.39	0.15	0.01	Significant at 99% level
	Overdispersion	1.12	0.31	--	--
Four-Leg Signalized Intersections (4SG)	Intercept	-6.04	1.12	--	--
	$\ln(AADT_{maj})$	0.52	0.11	<.01	Significant at 99% level
	$\ln(AADT_{min})$	0.10	0.07	0.13	Significant at 85% level
	Overdispersion	0.55	0.08	--	--

Base Condition: Absence of intersection lighting; for stop control intersections, absence of turn lanes on non-stop control approaches; for signal control intersections, absence of turn lanes on all intersection approaches

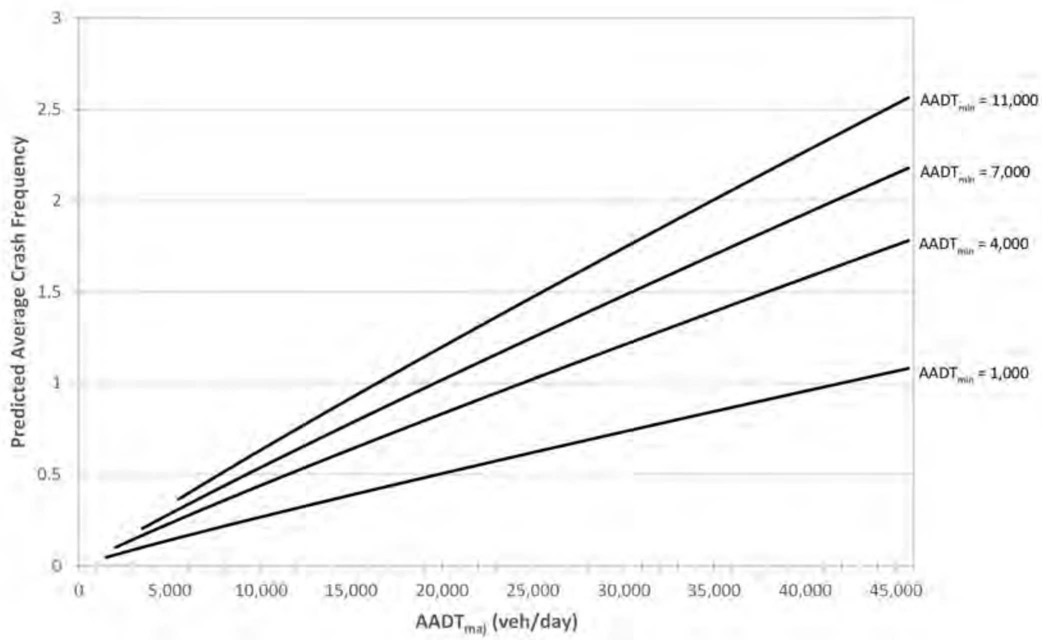


Figure 33. Graphical representation of the SPF for SV total crashes at three-leg stop-controlled intersections on high-speed urban and suburban arterials

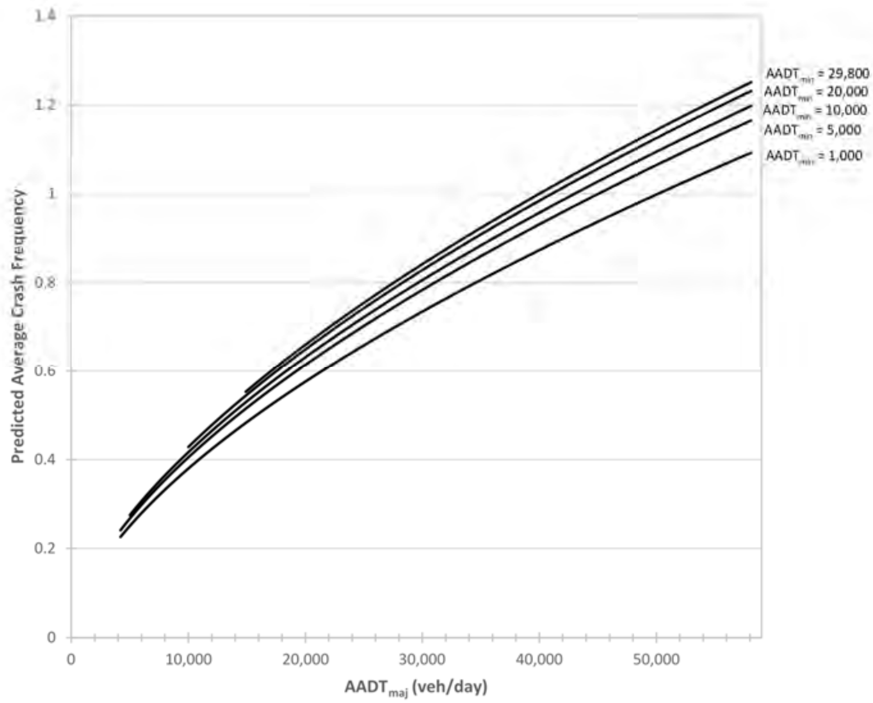


Figure 34. Graphical representation of the SPF for SV total crashes at three-leg signalized intersections on high-speed urban and suburban arterials

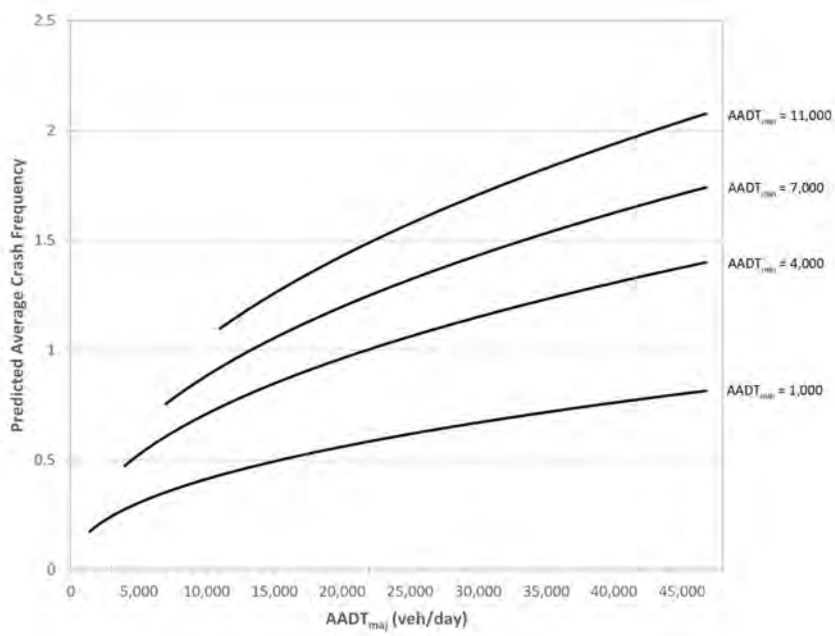


Figure 35. Graphical representation of the SPF for SV total crashes at four-leg stop-controlled intersections on high-speed urban and suburban arterials

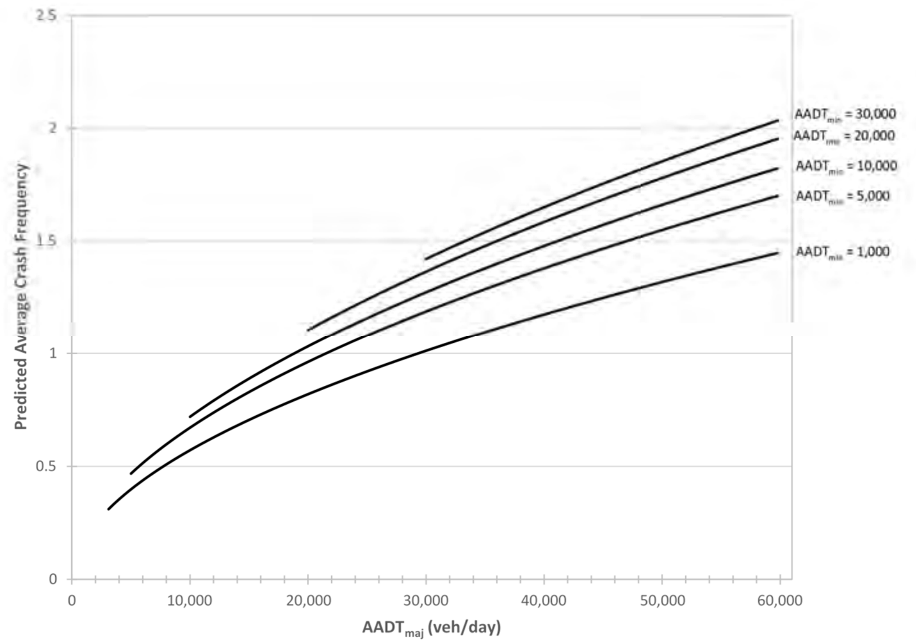


Figure 36. Graphical representation of the SPF for SV total crashes at four-leg signalized intersections on high-speed urban and suburban arterials

Table 45. SPF coefficients for intersections on high-speed urban and suburban arterials—SV FI crashes

Intersection Type	Parameter	Estimate	Standard Error	Pr > F	Significance Level?
SV FI CRASHES					
Three-Leg Stop-Controlled Intersections (3ST)	Intercept	-14.00	6.64	--	--
	$\ln(AADT_{maj})$	0.79	0.64	0.22	Not significant
	$\ln(AADT_{min})$	0.53	0.26	0.05	Significant at 95% level
	Overdispersion	2.10	0.70	--	--
Three-Leg Signalized Intersections (3SG)	Intercept	-7.41	3.69	--	--
	$\ln(AADT_{maj})$	0.63	0.39	0.11	Significant at 85% level
	$\ln(AADT_{min})$	-0.09	0.17	0.61	Not significant
	Overdispersion	1.04	0.57	--	--
Four-Leg Stop-Controlled Intersections (4ST)	Intercept	-13.96	4.85	--	--
	$\ln(AADT_{maj})$	0.91	0.46	0.05	Significant at 95% level
	$\ln(AADT_{min})$	0.45	0.27	0.10	Significant at 90% level
	Overdispersion	1.64	0.56	--	--
Four-Leg Signalized Intersections (4SG)	Intercept	-9.89	1.88	--	--
	$\ln(AADT_{maj})$	0.83	0.18	<.01	Significant at 99% level
	$\ln(AADT_{min})$	0.04	0.10	0.65	Not significant
	Overdispersion	0.98	0.19	--	--

Base Condition: Absence of intersection lighting; for stop control intersections, absence of turn lanes on non-stop control approaches; for signal control intersections, absence of turn lanes on all intersection approaches

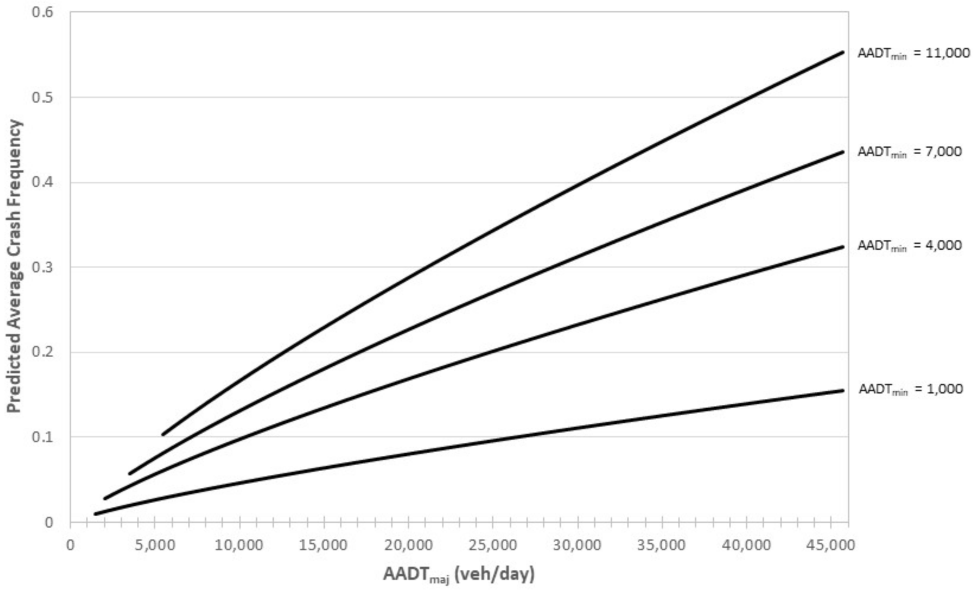


Figure 37. Graphical representation of the SPF for SV FI crashes at three-leg stop-controlled Intersections on high-speed urban and suburban arterials

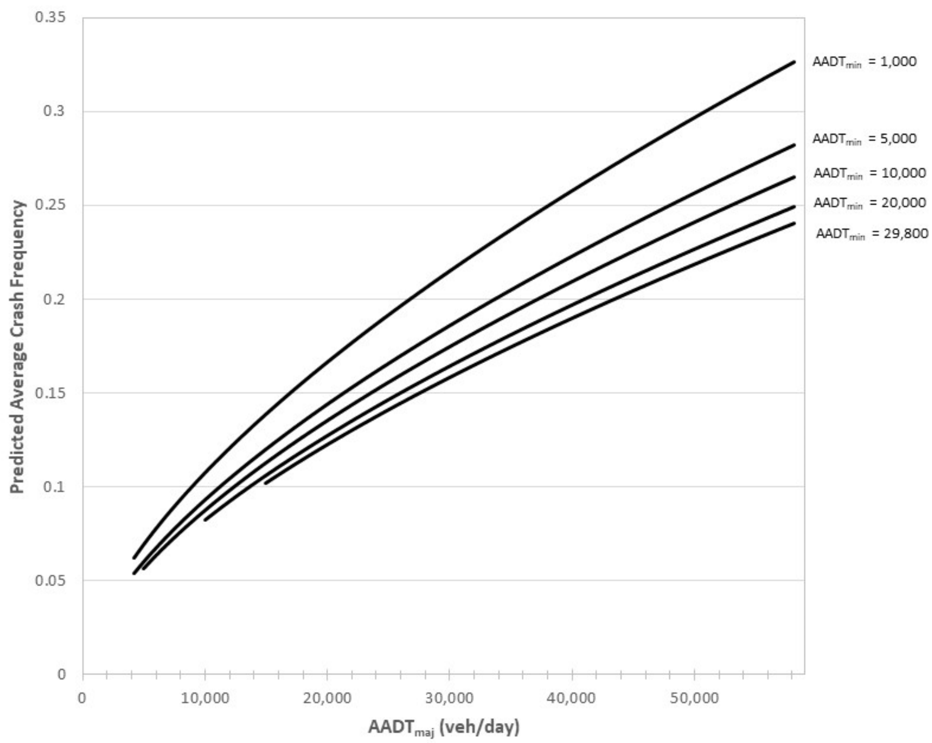


Figure 38. Graphical representation of the SPF for SV FI crashes at three-leg signalized intersections on high-speed urban and suburban arterials

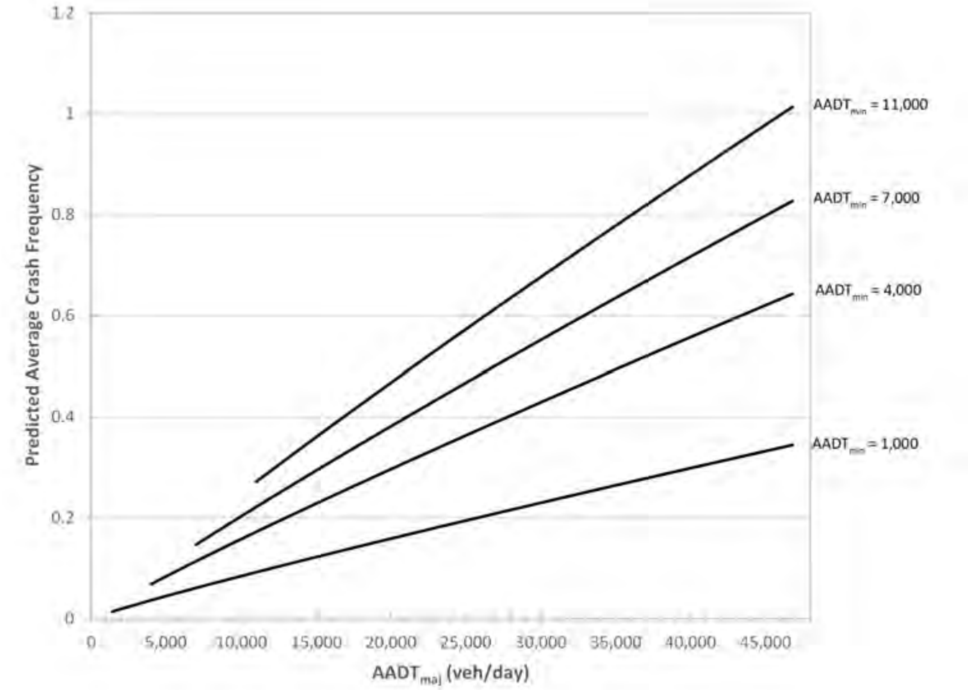


Figure 39. Graphical representation of the SPF for SV FI crashes at four-leg stop-controlled intersections on high-speed urban and suburban arterials

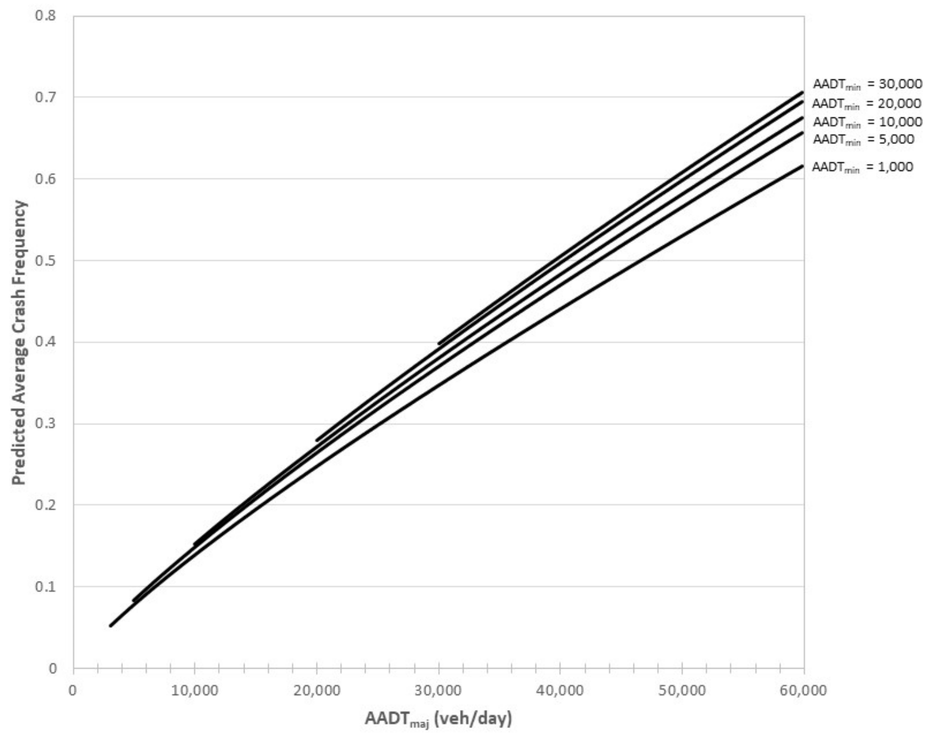


Figure 40. Graphical representation of the SPF for SV FI crashes at four-leg signalized intersections on high-speed urban and suburban arterials

Table 46. SPF coefficients for intersections on high-speed urban and suburban arterials—SV PDO crashes

Intersection Type	Parameter	Estimate	Standard Error	Pr > F	Significance Level?
SV PDO CRASHES					
Three-Leg Stop-Controlled Intersections (3ST)	Intercept	-12.07	3.58	--	--
	$\ln(AADT_{maj})$	0.92	0.34	< .01	Significant at 99% level
	$\ln(AADT_{min})$	0.31	0.15	0.04	Significant at 95% level
	Overdispersion	0.75	0.25	--	--
Three-Leg Signalized Intersections (3SG)	Intercept	-7.54	2.69	--	--
	$\ln(AADT_{maj})$	0.61	0.28	0.04	Significant at 95% level
	$\ln(AADT_{min})$	0.08	0.13	0.52	Not significant
	Overdispersion	0.74	0.27	--	--
Four-Leg Stop-Controlled Intersections (4ST)	Intercept	-6.15	3.11	--	--
	$\ln(AADT_{maj})$	0.24	0.30	0.44	Not significant
	$\ln(AADT_{min})$	0.41	0.16	0.01	Significant at 99% level
	Overdispersion	1.40	0.38	--	--
Four-Leg Signalized Intersections (4SG)	Intercept	-5.10	1.35	--	--
	$\ln(AADT_{maj})$	0.37	0.13	< .01	Significant at 99% level
	$\ln(AADT_{min})$	0.11	0.08	0.17	Significant at 80% level
	Overdispersion	0.84	0.12	--	--

Base Condition: Absence of intersection lighting; for stop control intersections, absence of turn lanes on non-stop control approaches; for signal control intersections, absence of turn lanes on all intersection approaches

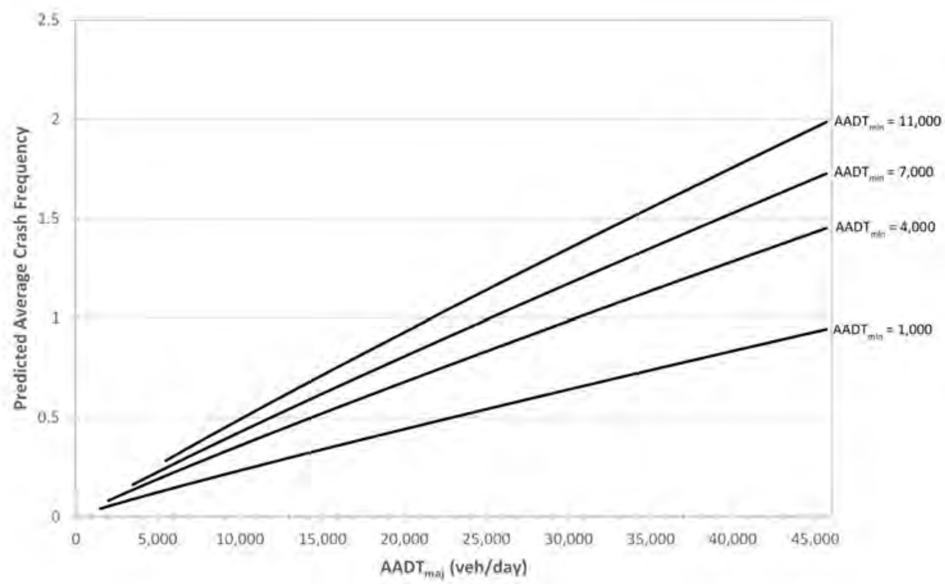


Figure 41. Graphical representation of the SPF for SV PDO crashes at three-leg stop-controlled intersections on high-speed urban and suburban arterials

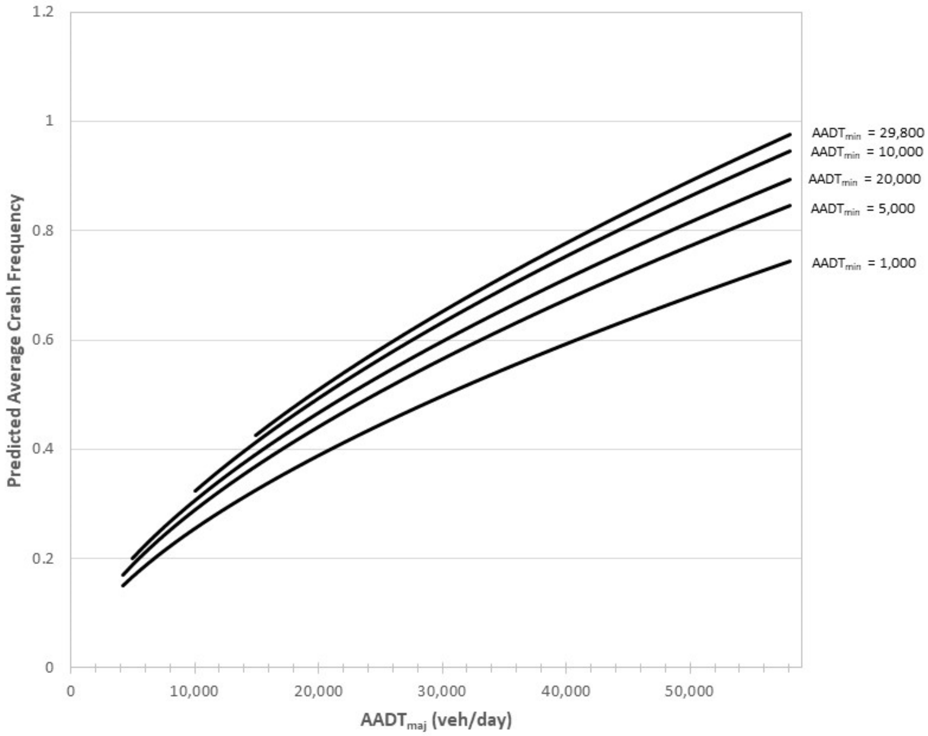


Figure 42. Graphical representation of the SPF for SV PDO crashes at three-leg signalized intersections on high-speed urban and suburban arterials

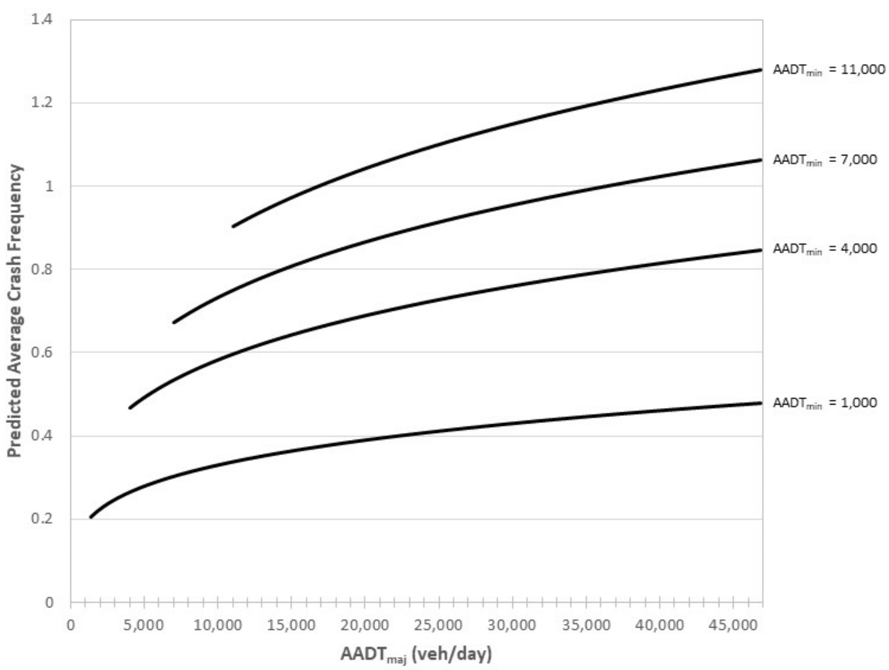


Figure 43. Graphical representation of the SPF for SV PDO crashes at four-leg stop-controlled intersections on high-speed urban and suburban arterials

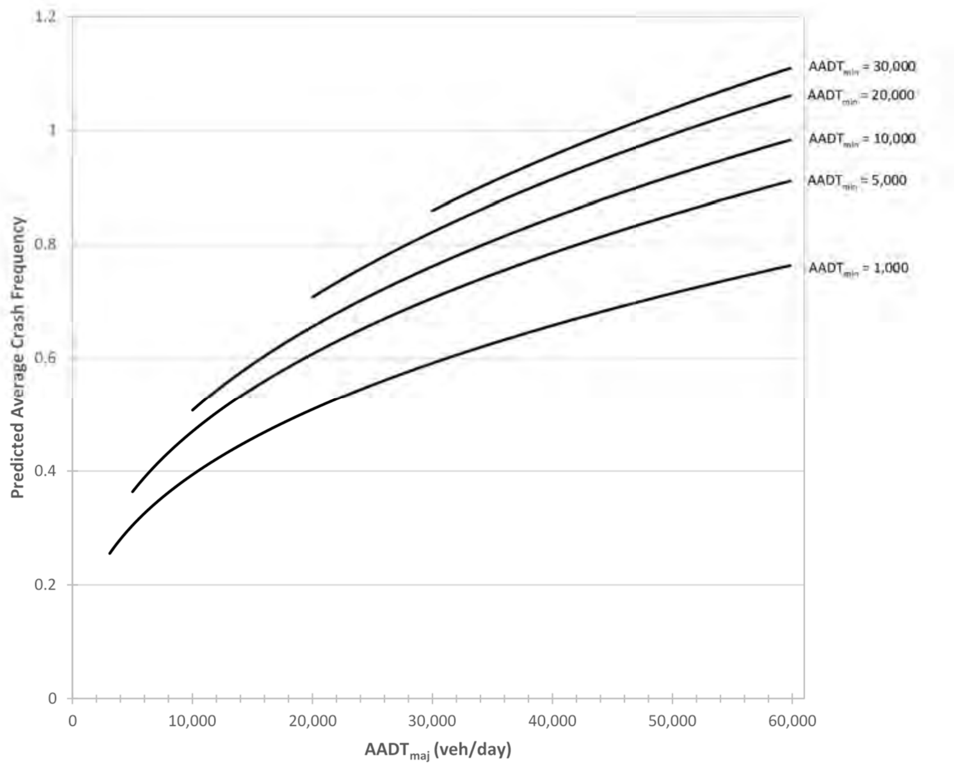


Figure 44. Graphical representation of the SPF for SV PDO crashes at four-leg signalized intersections on high-speed urban and suburban arterials

Similar to Tables 12-11 (MV crashes) and 12-13 (SV crashes) in the HSM, Table 47 (MV crashes) and Table 48 (SV crashes) provide percentages of FI and PDO crash severities by collision types, separately for each intersection type. These percentages were calculated based on all multiple- and SV crash counts at all intersections for all states combined.

Table 47. Distribution of MV crashes for intersections on high-speed urban and suburban arterials

Manner of Collision	Percentage of Multiple-Vehicle Crashes							
	Three-Leg Stop-Controlled Intersections (3ST)		Three-Leg Signalized Intersections (3SG)		Four-Leg Stop-Controlled Intersections (4ST)		Four-Leg Signalized Intersections (4SG)	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Rear-end collision	36.0	48.5	63.4	64.8	16.7	31.5	56.0	65.7
Head-on collision	2.3	2.1	2.9	0.4	5.1	1.9	2.8	0.9
Angle collision	47.3	31.7	23.2	16.5	65.3	42.6	30.3	17.6
Sideswipe collision	7.7	11.3	3.6	11.4	2.8	14.2	2.6	9.4
Other MV collisions	6.8	6.4	6.9	6.9	10.2	9.8	8.3	6.4
Total MV crashes	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**

Table 48. Distribution of SV crashes for intersections on high-speed urban and suburban arterials

Manner of Collision	Percentage of SV Crashes							
	Three-Leg Stop-Controlled Intersections (3ST)		Three-Leg Signalized Intersections (3SG)		Four-Leg Stop-Controlled Intersections (4ST)		Four-Leg Signalized Intersections (4SG)	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Collision with parked vehicle	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Collision with animal	0.0	0.9	0.0	0.0	0.0	1.9	0.0	0.3
Collision with fixed object	18.6	25.7	55.2	41.1	10.9	24.3	24.0	42.2
Collision with other object	0.0	0.0	0.0	0.0	0.0	1.9	0.0	0.6
Other SV collision	67.4	68.1	31.0	53.4	73.9	70.1	62.0	52.0
Noncollision	14.0	5.3	13.8	5.5	15.2	1.9	14.0	4.9
Total SV crashes	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**

Table 49 provides the distribution of pedestrian crashes by total crashes for intersections on high-speed urban and suburban arterials. The proportion of pedestrian crashes is used to estimate the number of pedestrian crashes at intersections on high-speed urban and suburban arterials.

Table 49. Distribution of pedestrian crash counts and percentages for intersections on high-speed urban and suburban arterials

Intersection Type	Number of Sites	Number of Pedestrian Crashes	Number of Total Crashes	Percentage of Pedestrian Crashes
Three-Leg Stop-Controlled Intersections (3ST)	121	3	706	**0.42**
Three-Leg Signalized Intersections (3SG)	50	2	870	**0.23**
Four-Leg Stop-Controlled Intersections (4ST)	125	4	986	**0.41**
Four-Leg Signalized Intersections (4SG)	208	18	6,724	**0.27**

Table 50 provides the distribution of bicycle crashes by total crashes for intersections on high-speed urban and suburban arterials. The proportion of bicycle crashes is used to estimate the number of bicycle crashes at intersections on high-speed urban and suburban arterials.

Table 50. Distribution of bicycle crash counts and percentages for intersections on high-speed urban and suburban arterials

Intersection Type	Number of Sites	Number of Bicycle Crashes	Number of Total Crashes	Percentage of Bicycle Crashes
Three-Leg Stop-Controlled Intersections (3ST)	121	0	706	**0.00**
Three-Leg Signalized Intersections (3SG)	50	1	870	**0.11**
Four-Leg Stop-Controlled Intersections (4ST)	125	0	986	**0.00**
Four-Leg Signalized Intersections (4SG)	208	5	6,724	**0.07**

Following the development of the crash prediction models for intersections on high-speed urban and suburban arterials, compatibility testing of the new models to confirm that the new models provide reasonable results over a broad range of input conditions and that the new models integrate seamlessly with existing intersection crash prediction models in the first edition of the HSM was conducted. The graphical representations of the crash prediction models in Figures 21-44 provide some sense of the reasonableness of the new models for intersections on high-speed urban and suburban arterials. Nothing from these figures suggests that the models provide unreasonable results. In addition, the new models for intersections on high-speed urban and suburban arterials were compared to the corresponding models in Chapter 12 of the HSM.

Figure 45 illustrates a comparison of the predicted average crash frequency for MV total crashes based on the 3ST model for urban and suburban high-speed arterials (Table 41) to the predicted average crash frequency based on the 3ST model in Chapter 12 of the HSM. The dashed lines in the figure represent the predicted average crash frequency for the new model (i.e., 3ST model for urban and suburban high-speed arterials), and the solid lines represent the predicted average crash frequency for the 3ST model in the HSM. Similarly, Figure 46 illustrates a comparison of the predicted average crash frequency for MV FI crashes based on the 4ST model for urban and suburban high-speed arterials (Table 42) to the predicted average crash frequency based on the 4ST model in Chapter 12 of the HSM, and Figure 47 illustrates a comparison of the predicted average crash frequency for MV FI crashes based on the 4SG model for urban and suburban high-speed arterials (Table 42) to the predicted average crash frequency based on the 4SG model in Chapter 12 of the HSM. As illustrated in Figures 45-47 and consistent with most of the other compatibility testing for intersections on high-speed urban and suburban arterials, the new crash prediction models for intersections on high-speed urban and suburban arterials predicted slightly higher crash frequencies for the same traffic conditions as the corresponding models in HSM Chapter 12. This seems reasonable as higher speeds will require quicker reaction times to avoid potential conflicts. Similarly, it is not surprising that more MV FI crashes are predicted on high-speed urban and suburban arterials compared to the predictions for lower speed urban and suburban arterials, given the correlation between vehicle speed and crash severity.

In summary, the models for intersections on high-speed urban and suburban arterials appear to provide reasonable results over a broad range of input conditions and can be integrated seamlessly with existing intersection crash prediction models in the first edition of the HSM.

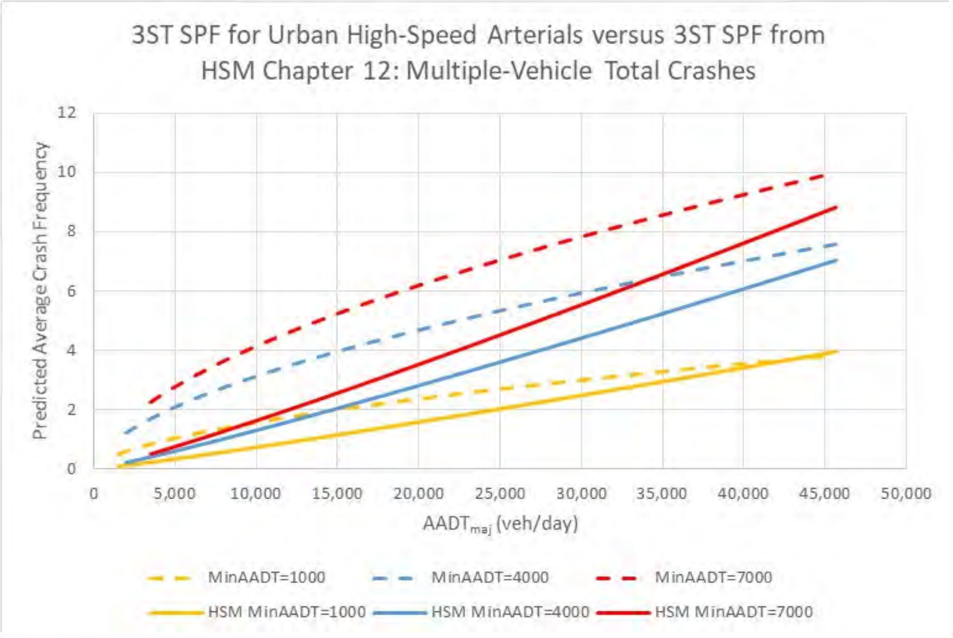


Figure 45. Comparison of new crash prediction model to existing model in HSM: 3ST for MV crashes for urban and suburban high-speed arterials vs 3ST for MV crashes from HSM Chapter 12 (total crashes)

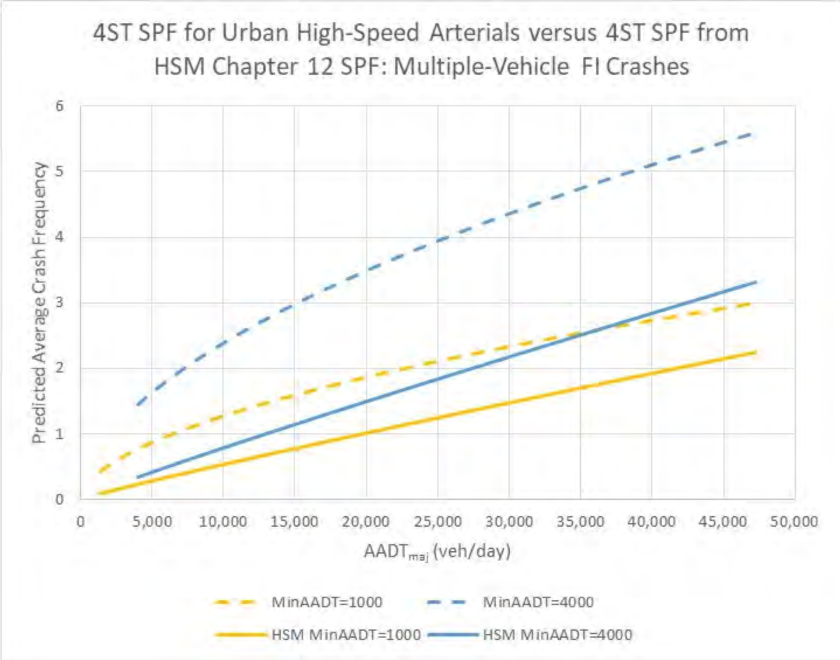


Figure 46. Comparison of new crash prediction model to existing model in HSM: 4ST for MV crashes for urban and suburban high-speed arterials vs 4ST for MV crashes from HSM Chapter 12 (FI crashes)

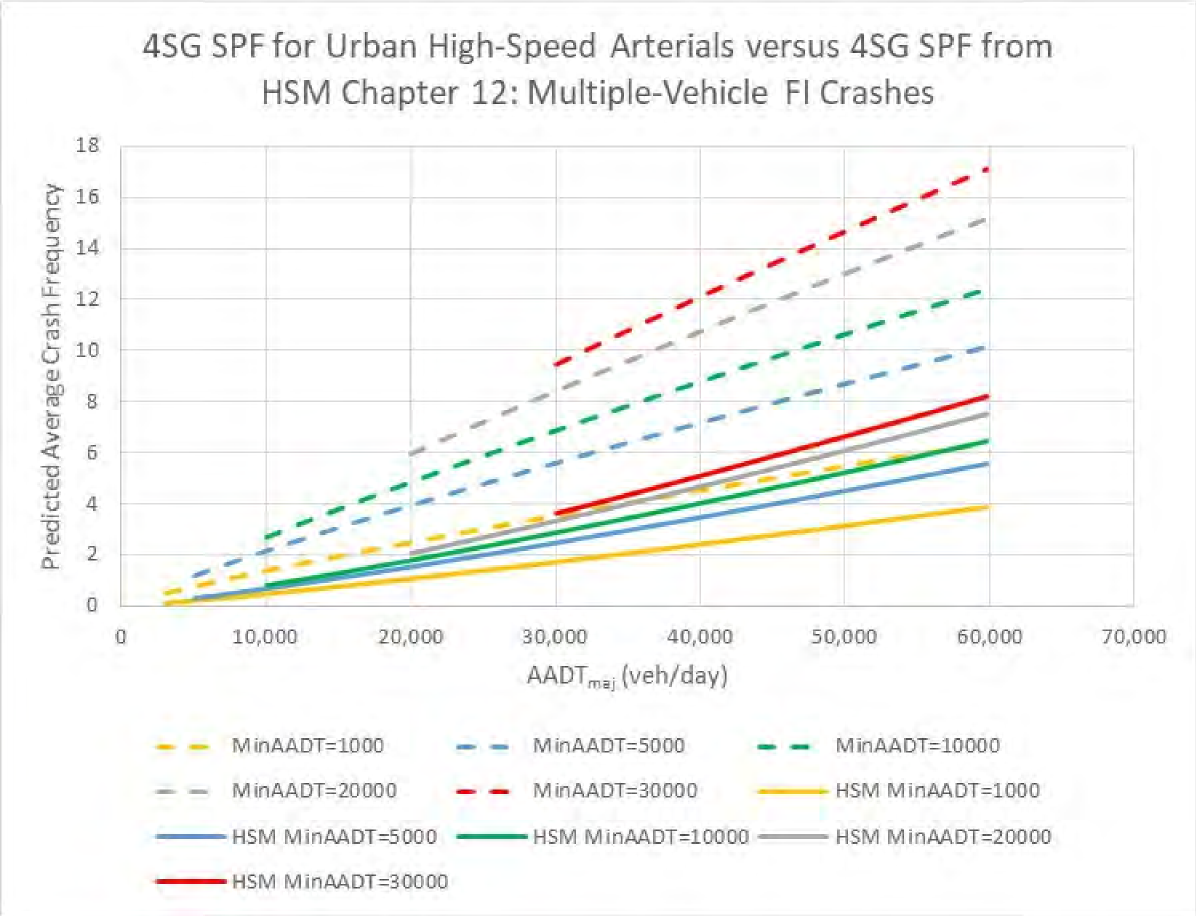


Figure 47. Comparison of new crash prediction model to existing model in HSM: 4SG for MV crashes for urban and suburban high-speed arterials vs 4SG for multiple vehicle crashes from HSM Chapter 12 (FI crashes)

5.4 Crash Modification Factors

During the development of the crash prediction models for intersections on high-speed urban and suburban arterials, three potential sources of CMFs for use with the SPFs were considered:

- CMFs developed as part of this research based on a cross-sectional study design and regression modeling
- CMFs already incorporated into the first edition of the HSM and applicable to intersections on high-speed urban and suburban arterials
- High-quality CMFs applicable to intersections on high-speed urban and suburban arterials developed using defensible study designs (e.g., observational before-after evaluation studies using SPFs—the EB method), as referenced in FHWA's CMF Clearinghouse with four or five-star quality ratings or based on a review of relevant intersection safety literature

After considering developing CMFs through regression modeling as part of this research and based on a review of the CMFs already incorporated in the first edition of the HSM and other potential high-quality CMFs developed using defensible study designs, three CMFs were identified for potential use with the crash prediction models for intersections on high-speed urban and suburban arterials, including:

- The CMF for intersection lighting based on the work by Elvik and Vaa (2004), which is identified for use with the intersection crash prediction models in Chapter 12 of the first edition of the HSM.
- The CMFs for providing a left-turn lane on one or more intersection approaches at an urban or suburban intersection based on the work by Harwood et al. (2002), which is identified for use with the intersection crash prediction models in Chapter 12 of the first edition of the HSM.
- The CMFs for providing a right-turn lane on one or more intersection approaches at an urban or suburban intersection based on the work by Harwood et al. (2002), which is identified for use with the intersection crash prediction models in Chapter 12 of the first edition of the HSM.

The CMFs recommended for use with the SPFs for intersections on high-speed urban and suburban arterials are presented below.

5.4.1 Lighting CMF

With the CMF for intersection lighting based on the work by Elvik and Vaa (2004), the base condition is the absence of intersection lighting. The CMF for lighted intersections is similar to the CMF in Equation 12-36 in the HSM and has the form:

$$CMF_i = 1 - 0.38 \times p_{ni} \quad \text{(Eq. 39)}$$

Where:

CMF_i = crash modification factor for the effect of lighting on total crashes; and
p_{ni} = proportion of total crashes for unlighted intersections that occur at night.

This CMF applies to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle crashes). Table 51 (similar to Table 12-27 in the HSM) presents default values for the nighttime crash proportion, p_{ni}, by roadway type.

Table 51. Nighttime crash proportions for unlighted intersections on high-speed urban and suburban arterials

Intersection Type	Proportion of Crashes that Occur at Night p_{ni}
Three-Leg Stop-Controlled	**0.291**
Three-Leg Signalized	**0.206**
Four-Leg Stop-Controlled	**0.256**
Four-Leg Signalized	**0.245**

5.4.2 Intersection Approaches with Left-Turn Lanes CMF

With the CMFs for providing a left-turn lane on one or more intersection approaches at an intersection on a high-speed urban and suburban arterial based on the work by Harwood et al. (2002), the base condition is the absence of left-turn lanes on intersection approaches. The CMFs for providing a left-turn lane on one or more intersection approaches are presented in Table 52. Table 52 is presented in the same format as Table 12-24 in the HSM Part C (AASHTO, 2010). These CMFs apply to all severity levels.

Table 52. CMF_i for installation of left-turn lanes on intersection approaches (Harwood et al., 2002; AASHTO, 2010)

Intersection Type	Intersection Traffic Control	Number of Approaches with Left-Turn Lanes[a]			
		One Approach	Two Approaches	Three Approaches	Four Approaches
Three-Leg	Minor road stop control[b]	0.67	0.45	-	-
	Traffic signal	0.93	0.86	0.80	-
Four-Leg	Minor road stop control[b]	0.73	0.53	-	-
	Traffic signal	0.90	0.81	0.73	0.66

[a] Stop-controlled approaches are not considered in determining the number of approaches with left-turn lanes.
[b] Stop signs present on minor road approaches only.

5.4.3 Intersection Approaches with Right-Turn Lanes CMF

With the CMFs for providing a right-turn lane on one or more intersection approaches at an intersection on a high-speed urban and suburban arterial based on the work by Harwood et al. (2002), the base condition is the absence of right-turn lanes on intersection approaches. The CMFs for providing a right-turn lane on one or more intersection approaches are presented in Table 53. Table 53 is presented in the same format as Table 12-26 in the HSM Part C (AASHTO, 2010). These CMFs apply to all severity levels.

Table 53. CMF_i for installation of right-turn lanes on intersection approaches (Harwood et al., 2002; AASHTO, 2010)

Intersection Type	Intersection Traffic Control	Number of Approaches with Right-Turn Lanes[a]			
		One Approach	Two Approaches	Three Approaches	Four Approaches
Three-leg	Minor- road stop control[b]	0.86	0.74	-	-
	Traffic signal	0.96	0.92	-	-
Four-leg	Minor road stop control[b]	0.86	0.74	-	-
	Traffic signal	0.96	0.92	0.88	0.85

[a] Stop-controlled approaches are not considered in determining the number of approaches with right-turn lanes.
[b] Stop signs present on minor road approaches only.

5.5 Severity Distribution Functions

The development of SDFs was explored for intersections on high-speed urban and suburban arterials using methods outlined in Section 2.2.3 of this report. SDFs were not used in the development of crash prediction methods in the first edition of the HSM but were subsequently used in the Supplement to the HSM for freeways and ramps (AASHTO, 2014). The database used to explore SDFs for intersections on high-speed urban and suburban arterials consisted of the same crashes and intersections as the databases used to estimate the SPFs, but restructured so that the basic observation unit (i.e., database row) is a crash instead of an intersection.

No traffic or geometric variables showed statistically significant effects in the SDFs for three-leg intersections with stop control (3ST) or signal control (3SG) on high-speed urban and suburban arterials. For four-leg intersections with stop control (4ST) and signal control (4SG) on high-speed urban and suburban arterials, the SDF takes the following form:

$$P_{4x,at,K} = \frac{\exp(V_{KA})}{1+\exp(V_{KA})} \times P_{K|KA,\mathrm{4x,at}} \quad \text{(Eq. 45)}$$

$$P_{4x,at,A} = \frac{\exp(V_{KA})}{1+\exp(V_{KA})} \times P_{A|KA,\mathrm{4x,at}} \quad \text{(Eq. 46)}$$

$$P_{4x,at,B} = (1 - P_{4x,at,K} - P_{4x,at,A}) \times P_{B|BC,\mathrm{4x,at}} \quad \text{(Eq. 47)}$$

$$P_{4x,at,C} = (1 - P_{4x,at,K} - P_{4x,at,A}) \times P_{C|BC,\mathrm{4x,at}} \quad \text{(Eq. 48)}$$

Where:

$P_{4x,at,K}$ = probability of a fatal crash (given that a fatal or injury crash occurred) for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control);

$P_{4x,at,A}$ = probability of an incapacitating injury crash (given that a fatal or injury crash occurred) for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control);

$P_{4x,at,B}$ = probability of a non-incapacitating injury crash (given that a fatal or injury crash occurred) for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control);

$P_{4x,at,C}$ = probability of a possible injury crash (given that a fatal or injury crash occurred) for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x*
(*x* = ST: minor road stop control; SG: signal control);

V_{KA} = systematic component of crash severity likelihood for severity *KA*;

$P_{K|KA,4x,at}$ = probability of a fatal crash given that the crash has a severity of either fatal or incapacitating injury for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control); and

$P_{A|KA,4x,at}$ = probability of an incapacitating injury crash given that the crash has a severity of either fatal or incapacitating injury for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control).

The basic model form for the systematic components of crash severity likelihood at 4-leg intersections on high-speed urban and suburban arterials is illustrated by Equation 49.

$$V_{KA} = a + (b \times 0.001 \times AADT_{maj}) + (c \times 0.001 \times AADT_{min}) + (d \times n_{majLTL}) + (e \times n_{majRTL}) + (f \times n_{majthru}) \quad \text{(Eq. 49)}$$

Where:

$AADT_{maj}$ = AADT on the major road (veh/day)
$AADT_{min}$ = AADT on the minor road (veh/day)
n_{majLTL} = total number of left-turn lanes on both major road approaches (0, 1, or 2)
n_{majRTL} = total number of right-turn lanes on both major road approaches (0, 1, or 2)
$n_{majthru}$ = total number of through lanes on the major road
$a, b, c, d, e,$ and f = estimated SDF coefficients

The SDF coefficients for 4-leg intersections on high-speed urban and suburban arterials are provided in Table 54.

Table 54. SDF coefficients for four-leg intersections on high-speed urban and suburban arterials

Control Type (x)	Severity (z)	Variable	a	b	c	d	e	f
Minor road stop control (ST)	Fatal or incapacitating injury (KA)	V_{KA}	-1.932	-0.0741	0.000	0.000	-0.338	0.383
Signal control, (SG)	Fatal or incapacitating injury (KA)	V_{KA}	-1.971	-0.0598	-0.0373	-0.178	-0.182	0.479

For four-leg intersections with stop control on high-speed urban and suburban arterials, values of 0.18 and 0.82 are used for $P_{K|KA}$ and $P_{A|KA}$, respectively.

For four-leg intersections with signal control on high-speed urban and suburban arterials, values of 0.18 and 0.82 are also used for $P_{K|KA}$ and $P_{A|KA}$, respectively.

5.6 Summary of Recommended Models for Incorporation in the HSM

In summary, several crash prediction models were developed for three- and four-leg intersections with stop control and signal control on high-speed urban and suburban arterials for consideration in the second edition of the HSM, including models for:

- Three-leg intersections with minor road stop control (3ST) on high-speed urban and suburban arterials
- Three-leg intersections with signal control (3SG) on high-speed urban and suburban arterials
- Four-leg intersections with minor road stop control (4ST) on high-speed urban and suburban arterials

- Four-leg intersections with signal control (4SG) on high-speed urban and suburban arterials

The final models presented in Tables 41-46 are recommended for inclusion in the second edition of the HSM. As noted, several of the models for SV crashes include major- and minor road AADT coefficients that were not significant. These models are still considered the most reasonable models for estimating SV crashes at intersections on high-speed urban and suburban arterials. Having models with coefficients for major- and minor road AADTs that are not significant is not a major concern because SV crashes at intersections do not occur often and is not a crash type of interest that agencies often consider to remedy. MV crashes are the major concern at intersections, and all of the MV models include coefficients for major- and minor road AADTs that are statistically significant. In addition, the final models for intersections on high-speed urban and suburban arterials recommended for inclusion in the HSM are not intended to replace the existing models in the HSM for the corresponding intersection configurations and traffic control types. Rather it is recommended the second edition of the HSM state that the intersection SPFs not designated specifically for high-speed arterials can be used to predict crash frequencies at intersections located on high-speed arterials, but use of models that have been developed specifically for intersections located on high-speed arterials is recommended when analyzing intersections located on urban and suburban arterials with posted speed limits of 50 mph or greater.

SDFs for intersections on high-speed urban and suburban arterials are reported in Section 5.5. However, for the reasons provided in Section 4.6, it is recommended for the second edition of the HSM that crash severity for intersections on high-speed urban and suburban arterials be addressed in a manner consistent with existing methods in Chapter 12 of the HSM, without use of SDFs.

Appendix A presents recommended text for incorporating the final recommended models for intersections on high-speed urban and suburban arterials into Chapter 12 of the HSM.

Chapter 6.
Development of Models for Use in HSM Crash Prediction Methods: Five-Leg Intersections

This section describes the development of crash predictive methods for five-leg intersections and presents the final models recommended for incorporation in the second edition of the HSM. None of the HSM Part C chapters in the first edition of the HSM include crash prediction models for five-leg intersections. A five-leg intersection is a junction of five roadway segments that intersect at a common paved area. Five-leg intersections can be stop-controlled or signal-controlled. Data collection and analysis focused on five-leg intersections with signal control on urban and suburban arterials (5SG) due to limitations in sample sizes and data availability for other area and traffic control types.

6.1 Site Selection and Data Collection

A list of potential five-leg intersections for model development was derived using databases obtained from state DOTs, HSIS, or Safety Analyst. The five-leg intersections ultimately selected for model development were in four states:

- Ohio (OH)
- Illinois (IL)
- Massachusetts (MA)
- Minnesota (MN)

Intersections in Michigan and California were also initially considered, but were not included in the databases. Data obtained from Michigan had limited information on approach-level traffic volumes for the five approach legs. HSIS data from California generally contained AADTs for major- and minor- approaches that were part of the state highway system but did not contain traffic volume information for the fifth leg, which was usually a local road.

Each potential intersection was visually investigated using Google Earth® to verify the intersection had five approaches and to remove from consideration intersections where there were noticeable changes in geometry, traffic control, or access points in close proximity to the intersection during the study period.

Lists of potential five-leg intersections and their coordinates were provided by Ohio, Illinois, and Massachusetts DOTs. Potential intersections in Minnesota were obtained from HSIS. Intersections from HSIS were located using the state's linear referencing system (LRS), and it was challenging to position them accurately on a map for further processing. The Network Explorer for Traffic Analysis (NeXTA) and Minnesota's roadway network shapefile were ultimately used as an alternative to identify potential intersections as nodes with five links. With this approach, nodes and links were created from the roadway network using the network segment endpoints to identify common intersection nodes. Five-leg intersections were then identified as node locations with five or more links connected. Finally, the coordinates of selected nodes were imported into ArcGIS to generate a KML file, which was then used for visual verifications in Google Earth®.

Visual investigation was essential to correct misclassifications of four-leg and six-leg intersections as five-leg intersections in the initial dataset. This was in part due to link-node roadway representations that showed five links approaching a given node. Figure 48 shows an example of a location identified as a five-leg intersection in the initial list, but later identified as a four-leg intersection during visual verification and not included in model development.

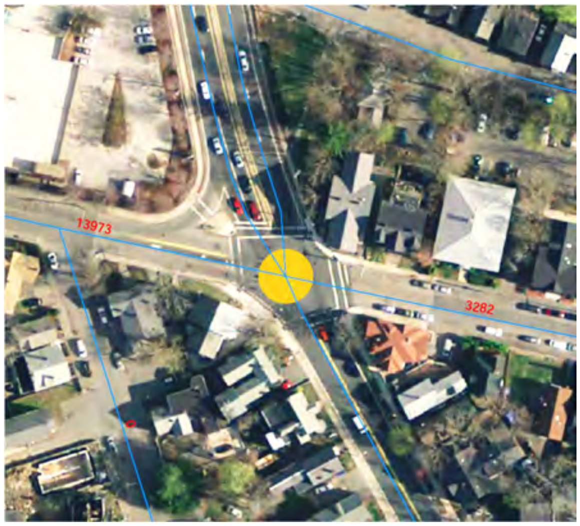


Figure 48. Sample intersection excluded from the five-leg analysis after visual inspection (Source: Massachusetts DOT / ArcMap)

As shown in Table 55, the initial set of 446 potential five-leg intersections in Ohio, Illinois, Massachusetts, and Minnesota resulted in a total of 177 verified five-leg intersections: 93 signalized in urban and suburban locations, 57 stop-controlled in urban and suburban locations, and 27 stop-controlled in rural locations. Difficulties in practically obtaining detailed traffic volume and crash data for five-leg intersections with stop control prevented further consideration of such locations for model development. Data collection continued for the 93 remaining five-leg intersections with signal control in urban and suburban areas.

Table 55. Potential and verified five-leg intersections

State	Potential Intersections	Verified Intersections		
		Rural	Urban and Suburban	
		Stop-Controlled	Stop-Controlled	Signal Control
OH	183	22	13	39
MA	18	0	0	18
IL	107 [a]	2	4	25
MN	138	3	40	11
Total	446	27	57	93

[a] The list of 86 potential five-leg intersections provided by Illinois DOT was expanded to 107 intersections during visual inspection.

Table 56 lists the intersection attributes collected (and respective definitions and permitted values) for five-leg intersections.

Table 56. Site characteristic variables collected for five-leg intersections

Variable	Definition	Range or Permitted Values
General Intersection Attributes		
Intersection configuration (i.e., number of legs and type of traffic control)	Indicates the number of legs and type of traffic control	5ST, 5SG
Area type	Indicates whether the intersection is in a rural or urban area	Rural, urban
Presence of intersection lighting	Indicates if overhead lighting is present at the intersection proper	Yes, no
Approach Specific Attributes		
Route name or number	Specifies the route name or number of the approach	
Location at intersection	Side/quadrant of the intersection the approach is located	N, S, E, W, NE, NW, SE, SW
Presence of left-turn lanes	The number of approaches with one or more left-turn lanes	0,1,2,3,4,5
Left-turn protected only	Number of approaches with protected only left-turn operations	0,1,2,3,4,5
Left-turn permitted only	Number of approaches with permitted only left-turn operations	0,1,2,3,4,5
Left-turn protected and permitted	Number of approaches with protected and permitted left-turn operations	0,1,2,3,4,5
Two-way no left-turn	Number of approaches with two-way operation and no left turns	0,1,2,3,4,5
Presence of right-turn lane	Number of approaches with one or more right-turn lanes	0,1,2,3,4,5
No turn on red	Number of approaches with no turn on red	0,1,2,3,4,5
Two-way no turn restrictions	Number of approaches with two-way operation and no turn restrictions	0,1,2,3,4,5
One-way	Number of approaches with one-way operation (traffic approaching intersection)	0,1,2,3,4,5
One-way receiving	Number of approaches with one-way operation (receiving traffic from intersection)	0,1,2,3,4,5
Red light camera	Indicates presence of red light cameras	Yes, no

In general, the goal of data collection was to obtain the most recent four to six years of crash and traffic volume data for each site for model development. After gathering all available information, a continuous five-year period from 2009 to 2013 was common to all four states and was therefore selected for model development. All data (i.e., site characteristics, crash, and traffic volume) were assembled into one database for the purposes of model development.

Traffic volumes for the 57 urban, five-leg signalized intersections in Ohio and Massachusetts were readily available from the respective five-leg intersection databases provided to the research team by each state. Additional sources were required to complete traffic volume data collection for intersections in Illinois and Minnesota.

Crash data were obtained directly from the state DOTs. All verified intersections with available traffic volumes also had available crash data except for five intersections in Massachusetts.

Therefore, the total number of intersections available for model development was 76, including 39 intersections from Ohio, 13 from Illinois, 13 from Massachusetts, and 11 from Minnesota. Definitions of intersection and intersection-related crashes from existing HSM intersection predictive methods were used for this study.

6.2 Descriptive Statistics of Database

A total of 76 five-leg intersections with signal control on urban and suburban arterials were available for development of crash prediction models. The data collections sites were located in four states—Illinois, Massachusetts, Minnesota, and Ohio. To remain consistent with the standards for development of the intersection predictive models in the first edition of the HSM, the goal of this research was to develop crash prediction models with a minimum of 200 site-years of data, and preferably 450 site-years of data or more.

6.2.1 Traffic Volumes and Site Characteristics

Table 57 summarizes the number of five-leg intersections with respect to lighting and red light camera presence, as well as selected operational characteristics by approach. Traffic volume and crash data were available for the years 2009 through 2013. Table 58 shows the summary statistics for traffic volumes at all 76 study sites used for model development, including the study period (date range), number of sites and site-years, and basic traffic volume statistics by state.

Table 57. Number of intersections with attributes present by approach

Approach Attribute Variable	Number of Approaches with Attribute Present						Total
	0	1	2	3	4	5	
Two-way no turn restrictions	1	0	3	8	14	50	76
Two-way no left-turn	62	8	5	0	0	1	76
One-way approaching	73	3	0	0	0	0	76
One-way receiving	61	12	3	0	0	0	76
Left-turn protected only	14	40	7	11	3	1	76
Left-turn permitted only	15	17	14	8	22	0	76
Left-turn protected and permitted	40	14	12	4	6	0	76
No turn on red	14	8	12	12	14	16	76
Presence of left-turn lane	14	11	24	8	15	4	76
Presence of right-turn lane	43	24	8	1	0	0	76
Intersection Attribute Variable	**Present**			**Not Present**			**Total**
Intersection lighting	71			5			76
Red light camera	0			76			76

Table 58. Major-, minor-, and fifth-road AADT statistics at urban, five-leg signalized intersections

State	Date Range	Number of Sites	Number of Site-Years	Major Road AADT (veh/day)				Minor Road AADT (veh/day)				Fifth-Road AADT (veh/day)			
				Min	Max	Mean	Median	Min	Max	Mean	Median	Min	Max	Mean	Median
OH	2009-2013	39	195	3,020	23,506	13,596	13,470	454	17,445	6,405	4,298	251	16,448	3,854	1,925
MA	2009-2013	13	65	5,425	28,208	13,576	13,679	2,782	14,489	6,608	6,704	2,479	15,421	6,252	3,325
IL	2009-2013	13	65	6,270	25,525	18,904	18,700	800	21,865	10,916	10,400	2,210	24,340	9,542	8,140
MN	2009-2013	11	55	7,270	29,630	15,503	15,330	2,190	10,650	5,358	4,870	247	11,493	5,412	5,230
All states	**2009-2013**	**76**	**380**	**3,020**	**29,630**	**14,776**	**14,276**	**454**	**21,865**	**7,060**	**6,162**	**247**	**24,340**	**5,463**	**3,319**

6.2.2 Crash Counts

All 76 intersections experienced crashes during the study period. The average number of single- and MV crashes per intersection over the 5-year study period was 35.2 crashes, and the average number of nonmotorized (i.e., vehicle-pedestrian plus vehicle-bicycle) crashes per intersection over the 5-year study period was 2.3 crashes. Intersection crashes were defined as those crashes that occurred within 250 ft of the intersection and were classified as at intersection or intersection-related, consistent with recommended practice in the HSM for assigning crashes to an intersection.

Table 59 shows all crashes combined, single- and MV crashes, and pedestrian and bicycle crash counts by crash severity and time of day for each state over the entire 5-year study period. Crash counts are aggregated by collision type and manner of collision across all states in Table 60.

Table 59. All crashes combined, single- and MV, and pedestrian and bicycle crash counts by crash severity—urban, five-leg signalized intersections

State	Date Range	Number of Sites	Number of Site-Years	Time of Day	All Crashes Combined			SV Crashes			Multiple-Vehicle Crashes			Pedestrian Crashes	Bicycle Crashes
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO	FI	FI
OH	2009-2013	39	195	All	1,434	428	1,006	37	10	27	1,351	372	979	27	19
				Night	322	109	213	15	4	11	294	92	202	8	5
MA	2009-2013	13	65	All	327	99	228	21	5	16	278	66	212	15	13
				Night	88	30	58	7	4	3	72	17	55	7	2
IL	2009-2013	13	65	All	867	265	602	33	11	22	745	165	580	42	47
				Night	222	71	151	11	7	4	190	43	147	12	9
MN	2009-2013	11	55	All	222	61	161	11	3	8	197	44	153	5	9
				Night	50	13	37	6	1	5	40	8	32	0	4
All states	**2009-2013**	**76**	**380**	**All**	**2,850**	**853**	**1,997**	**102**	**29**	**73**	**2,571**	**647**	**1,924**	**89**	**88**
				Night	**682**	**223**	**459**	**39**	**16**	**23**	**596**	**160**	**436**	**27**	**20**

Table 60. Crash counts by collision type and manner of collision and crash severity at urban, five-leg signalized intersections

Collision Type	Total	FI	PDO
Single-Vehicle Crashes			
Collision with parked vehicle	4	0	4
Collision with animal	0	0	0
Collision with fixed object	24	9	15
Collision with other object	2	0	2
Other SV collision	70	18	52
Noncollision	2	2	0
All SV crashes[a]	**102**	**29**	**73**
Multiple-Vehicle Crashes			
Rear-end collision	1,104	275	829
Head-on collision	88	42	46
Angle collision	665	208	457
Sideswipe collision	357	32	325
Other multiple-vehicle collisions	357	90	267
Total MV crashes[a]	2,571	647	1,924
Total Crashes[a]	2,673	676	1,997

[a] Note crash counts do not include pedestrian and bicycle crashes

6.3 Safety Performance Functions—Model Development

Intersection SPFs were developed in the forms illustrated by Equations 50 through 52:

$$N_{spf\ int} = exp\left[a + b \times \ln\left(AADT_{maj}\right) + c \times \ln(AADT_{min}) + d \times \ln\left(AADT_{fif}\right)\right] \quad \text{(Eq. 50)}$$

$$N_{spf\ int} = exp\left[a + b \times \ln\left(AADT_{maj}\right) + e \times \ln\left(AADT_{min+fif}\right)\right] \quad \text{(Eq. 51)}$$

$$N_{spf\ int} = exp[a + f \times \ln(AADT_{total})] \quad \text{(Eq. 52)}$$

Where:

$N_{spf\ int}$ = predicted average crash frequency for an intersection with base conditions (crashes/year)
$AADT_{maj}$ = AADT on the major road (veh/day)
$AADT_{min}$ = AADT on the minor road (veh/day)
$AADT_{fif}$ = AADT on the fifth leg (veh/day)
$AADT_{min+fif}$ = sum of $AADT_{min}$ and $AADT_{fif}$ (veh/day)
$AADT_{total}$ = sum of $AADT_{maj}$, $AADT_{min}$, and $AADT_{fif}$ (veh/day)
a, b, c, d, e, and *f* = estimated regression coefficients

For five-leg signalized intersections on urban and suburban arterials, the SPFs were developed in a manner consistent with the methodology used in Chapter 12 of the HSM for predicting intersections crashes in urban and suburban areas. This methodology is illustrated in Equation 4 and Equation 5.

$$N_{predicted\ int} = \left(N_{bi} + N_{pedi} + N_{bikei}\right) \times C_i \quad \text{(Eq. 4)}$$

$$N_{bi} = N_{spf\ int} \times \left(CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{yi}\right) \quad \text{(Eq. 5)}$$

Where:

$N_{predicted\ int}$ = predicted average crash frequency for an individual intersection for the selected year (crashes/year)
N_{bi} = predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle crashes) (crashes/year)
N_{pedi} = predicted average crash frequency of vehicle-pedestrian crashes of an intersection (crashes/year)
N_{bikei} = predicted average crash frequency of vehicle-bicycle crashes of an intersection (crashes/year)
$N_{spf\ int}$ = predicted total average crash frequency of intersection-related crashes for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions) (crashes/year)
$CMF_{1i} \ldots CMF_{yi}$ = crash modification factors specific to intersection type *i* and specific geometric design and traffic control features *y*
C_i = calibration factor to adjust the SPF for intersection type *i* to local conditions

The SPF portion of N_{bi}, $N_{spf\ int}$, is the sum of two more disaggregate predictions by collision type, as shown in Equation 6.

$$N_{spf\ int} = N_{bimv} + N_{bisv} \quad \text{(Eq. 6)}$$

Where:

N_{bimv} = predicted average crash frequency of MV crashes of an intersection for base conditions (crashes/year)
N_{bisv} = predicted average crash frequency of SV crashes of an intersection for base conditions (crashes/year)

Separate model structures are used to estimate the yearly number of vehicle-pedestrian (N_{pedi}) and vehicle-bicycle (N_{bikei}) crashes at five-leg signalized intersections on urban and suburban arterials. The average number of annual vehicle-pedestrian and vehicle-bicycle crashes are estimated with Equations 9 and 12, respectively.

$$N_{pedi} = N_{bi} \times f_{pedi} \quad \text{(Eq. 9)}$$

Where:

f_{pedi} = pedestrian crash adjustment factor for intersection type *i*

$$N_{bikei} = N_{bi} \times f_{bikei} \quad \text{(Eq. 12)}$$

Where:

f_{bikei} = bicycle crash adjustment factor for intersection type *i*

All of the vehicle-pedestrian and vehicle-bicycle crashes predicted with Equations 9 and 12 are assumed to be FI crashes (none as PDO).

All SPFs were developed using a NB regression model based on all sites combined. Based on a review of the number of states, sites, site-years, and crashes for the database assembled, data for all sites were used for model development to maximize the sample size rather than using a portion of the data for model development and a portion for model validation. A significance level of 0.2 was used to assess the individual, estimated regression parameters. During model development, several intersection characteristics were initially tested in the models to develop CMFs for use with the SPFs. However, the intersection characteristics showed no consistent or statistically significant relationships to expected crash frequency. Therefore, no CMFs for use with the SPFs were developed. Additionally, existing CMFs for other intersection forms (e.g., four-leg signalized intersections) were not adapted to five-leg signalized intersections due to the different operational characteristics inherent to a five-leg intersection. Therefore, AADT-only models were developed with no base conditions for five-leg intersections with signal control on urban and suburban arterials. STATA 13 was used for all modeling.

The final SPFs for five-leg intersections with signal control on urban and suburban arterials are provided in the following tables:

- Table 61: MV total, FI, and PDO crashes using Equation 50
- Table 62: MV total, FI, and PDO crashes using Equation 51
- Table 63: MV total, FI, and PDO crashes using Equation 52
- Table 64: SV total, FI, and PDO crashes using Equation 50
- Table 65: SV total, FI, and PDO crashes using Equation 51
- Table 66: SV total, FI, and PDO crashes using Equation 52

Each table shows the estimated model coefficients and overdispersion parameter (estimate), their standard errors, and associated p-values (or significance level) for each severity level.
Figures 49-54 graphically present the SPFs shown in Tables 61-66 for various major-, minor-, and fifth-approach AADTs.

SPFs for vehicle-pedestrian and vehicle-bicycle crashes at five-leg intersections with signal control on urban and suburban arterials could not be developed as pedestrian and bicycle volumes were not available.

Table 61. SPF coefficients for five-leg intersections with signal control on urban and suburban arterials-MV crashes (AADTs separate for major-, minor-, and fifth-roads)

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
MULTIPLE-VEHICLE CRASHES					
Total Crashes	Intercept	-11.23	1.81	--	--
	$\ln(AADT_{maj})$	0.87	0.21	0.00	Significant at 99% level
	$\ln(AADT_{min})$	0.36	0.10	0.00	Significant at 99% level
	$\ln(AADT_{fif})$	0.19	0.08	0.02	Significant at 95% level
	Overdispersion	0.46	0.08	--	--
FI Crashes	Intercept	-15.00	2.64	--	--
	$\ln(AADT_{maj})$	1.30	0.30	0.00	Significant at 99% level
	$\ln(AADT_{min})$	0.27	0.13	0.04	Significant at 95% level
	$\ln(AADT_{fif})$	0.08	0.10	0.44	Not significant
	Overdispersion	0.64	0.13	--	--
PDO Crashes	Intercept	-10.92	1.83	--	--
	$\ln(AADT_{maj})$	0.75	0.21	0.00	Significant at 99% level
	$\ln(AADT_{min})$	0.39	0.10	0.00	Significant at 99% level
	$\ln(AADT_{fif})$	0.23	0.09	0.01	Significant at 99% level
	Overdispersion	0.48	0.09	--	--

No base conditions

Table 62. SPF coefficients for five-leg intersections with signal control on urban and suburban arterials-MV crashes (AADTs combined for minor- and fifth-roads)

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
MULTIPLE-VEHICLE CRASHES					
Total Crashes	Intercept	-11.42	1.82	--	--
	$\ln(AADT_{maj})$	0.85	0.22	0.00	Significant at 99% level
	$\ln(AADT_{min+fif})$	0.55	0.13	0.00	Significant at 99% level
	Overdispersion	0.47	0.08	--	--
FI Crashes	Intercept	-15.22	2.63	--	--
	$\ln(AADT_{maj})$	1.24	0.31	0.00	Significant at 99% level
	$\ln(AADT_{min+fif})$	0.40	0.17	0.02	Significant at 95% level
	Overdispersion	0.63	0.13	--	--
PDO Crashes	Intercept	-11.07	1.85	--	--
	$\ln(AADT_{maj})$	0.73	0.22	0.00	Significant at 99% level
	$\ln(AADT_{min+fif})$	0.60	0.14	0.00	Significant at 99% level
	Overdispersion	0.50	0.09	--	--

No base conditions

Table 63. SPF coefficients for five-leg intersections with signal control on urban and suburban arterials-MV crashes

(AADTs combined for major-, minor-, and fifth-roads)

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
MULTIPLE-VEHICLE CRASHES					
Total Crashes	Intercept	-12.83	1.82	--	--
	$\ln(AADT_{total})$	1.44	0.18	0.00	Significant at 99% level
	Overdispersion	0.47	0.08	---	--
FI Crashes	Intercept	-14.96	2.57	--	--
	$\ln(AADT_{total})$	1.51	0.25	0.00	Significant at 99% level
	Overdispersion	0.65	0.13	--	--
PDO Crashes	Intercept	-12.87	1.84	--	--
	$\ln(AADT_{total})$	1.41	0.18	0.00	Significant at 99% level
	Overdispersion	0.49	0.09	--	--

No base conditions

Table 64. SPF coefficients for five-leg intersections with signal control on urban and suburban arterials-SV crashes (AADTs separate for major-, minor-, and fifth-roads)

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
SV CRASHES					
Total Crashes	Intercept	-11.23	3.08	--	--
	$\ln(AADT_{maj})$	0.70	0.35	0.05	Significant at 95% level
	$\ln(AADT_{min})$	0.09	0.15	0.54	Not significant
	$\ln(AADT_{fif})$	0.28	0.14	0.05	Significant at 95% level
	Overdispersion	0.36	0.21	--	--
FI Crashes	Intercept	-15.54	4.89	0.00	--
	$\ln(AADT_{maj})$	0.54	0.59	0.36	Not significant
	$\ln(AADT_{min})$	0.62	0.30	0.04	Significant at 95% level
	$\ln(AADT_{fif})$	0.28	0.23	0.21	Not significant
	Overdispersion	0.19	0.39	--	--
PDO Crashes	Intercept	-10.13	3.27	0.00	--
	$\ln(AADT_{maj})$	0.68	0.37	0.07	Significant at 90% level
	$\ln(AADT_{min})$	-0.09	0.15	0.54	Not significant
	$\ln(AADT_{fif})$	0.32	0.15	0.04	Significant at 95% level
	Overdispersion	0.19	0.22	--	--

No base conditions

Table 65. SPF coefficients for five-leg intersections with signal control on urban and suburban arterials-SV crashes (AADTs combined for minor- and fifth-roads)

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
SV CRASHES					
Total Crashes	Intercept	-12.01	3.06	--	--
	$\ln(AADT_{maj})$	0.56	0.36	0.12	Significant at 85% level
	$\ln(AADT_{min+fif})$	0.56	0.23	0.01	Significant at 99% level
	Overdispersion	0.34	0.21	--	--
FI Crashes	Intercept	-17.13	4.95	0.00	--
	$\ln(AADT_{maj})$	0.21	0.60	0.73	Not significant
	$\ln(AADT_{min+fif})$	1.32	0.45	0.00	Significant at 99% level
	Overdispersion	0.14	0.35	--	--
PDO Crashes	Intercept	-11.14	3.40	--	--
	$\ln(AADT_{maj})$	0.65	0.40	0.11	Significant at 85% level
	$\ln(AADT_{min+fif})$	0.34	0.24	0.15	Significant at 85% level
	Overdispersion	0.28	0.24	--	--

No base conditions

Table 66. SPF coefficients for five-leg intersections with signal control on urban and suburban arterials-SV crashes (AADTs combined for major-, minor-, and fifth-roads)

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
SV CRASHES					
Total Crashes	Intercept	-13.94	3.10	--	--
	$\ln(AADT_{total})$	1.23	0.30	0.00	Significant at 99% level
	Overdispersion	0.34	0.20	--	--
FI Crashes	Intercept	-20.72	5.20	--	--
	$\ln(AADT_{total})$	1.76	0.50	0.00	Significant at 99% level
	Overdispersion	0.15	0.36	--	--
PDO Crashes	Intercept	-12.25	3.36	--	--
	$\ln(AADT_{total})$	1.03	0.33	0.00	Significant at 99% level
	Overdispersion	0.27	0.23	--	--

No base conditions

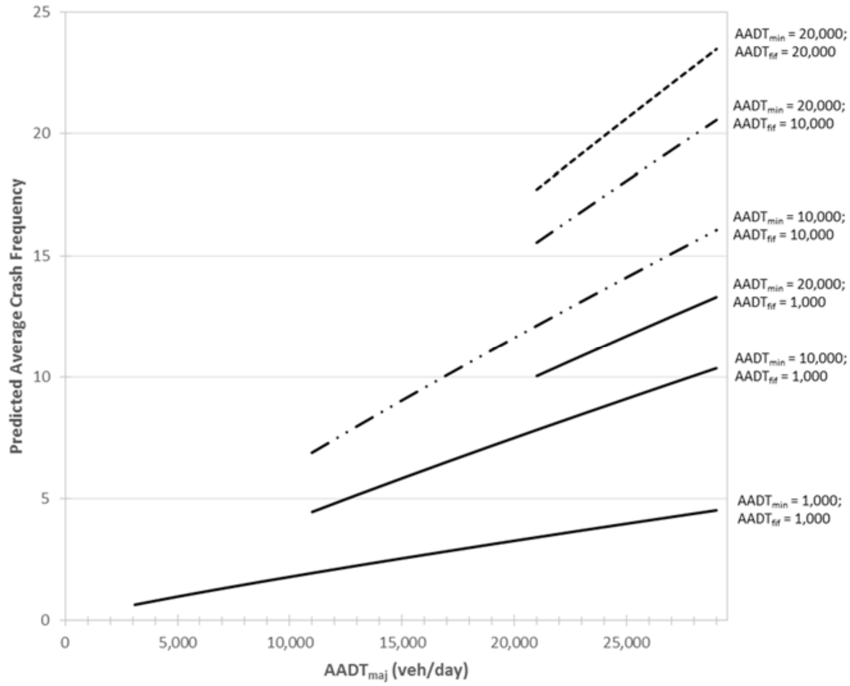


Figure 49. Graphical representation of the SPF for MV total crashes at five-leg intersections with signal control on urban and suburban arterials (based on model for MV total crashes in Table 61)

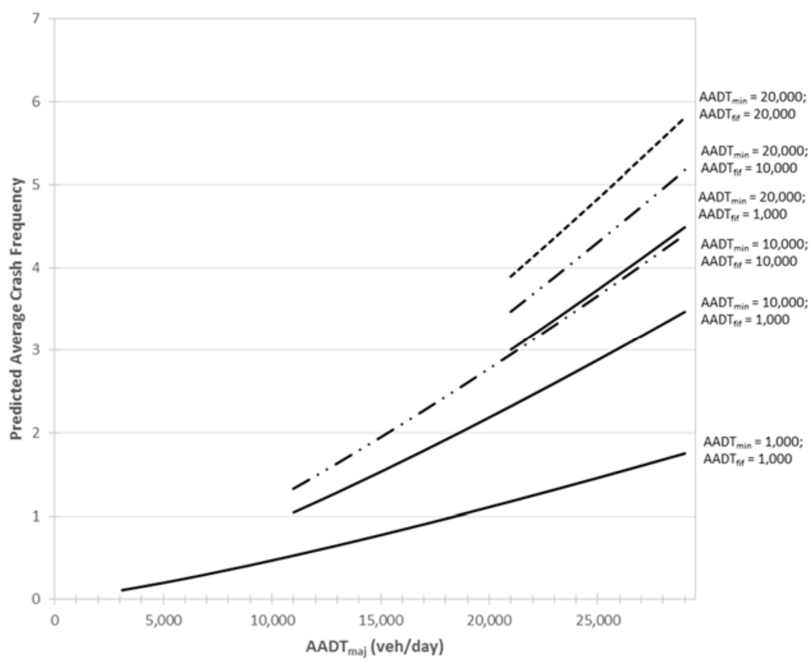


Figure 50. Graphical representation of the SPF for MV FI crashes at five-leg intersections with signal control on urban and suburban arterials (model for multiple-vehicle FI crashes in Table 62)

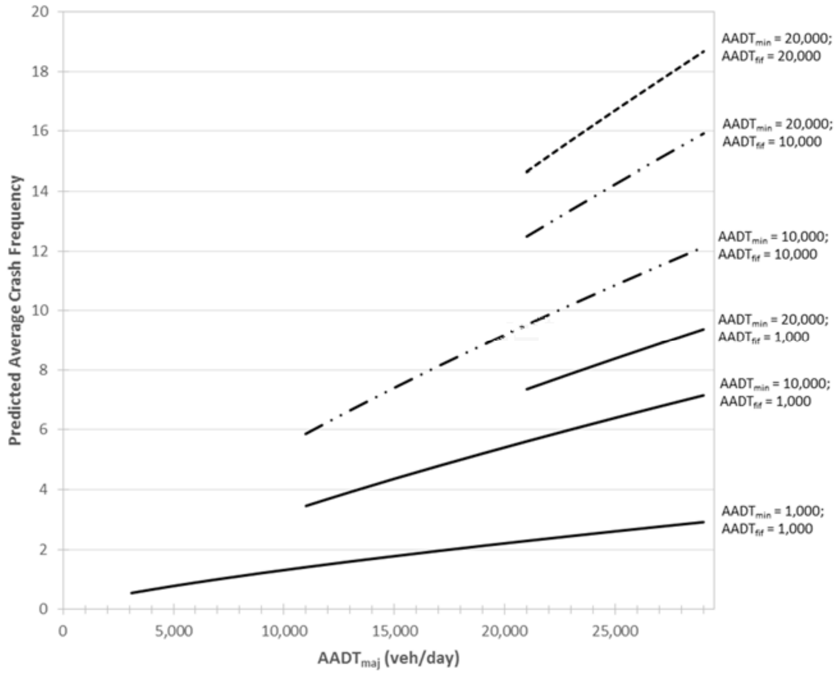


Figure 51. Graphical representation of the SPF for MV PDO crashes at five-leg intersections with signal control on urban and suburban arterials (based on model for MV PDO crashes in Table 61)

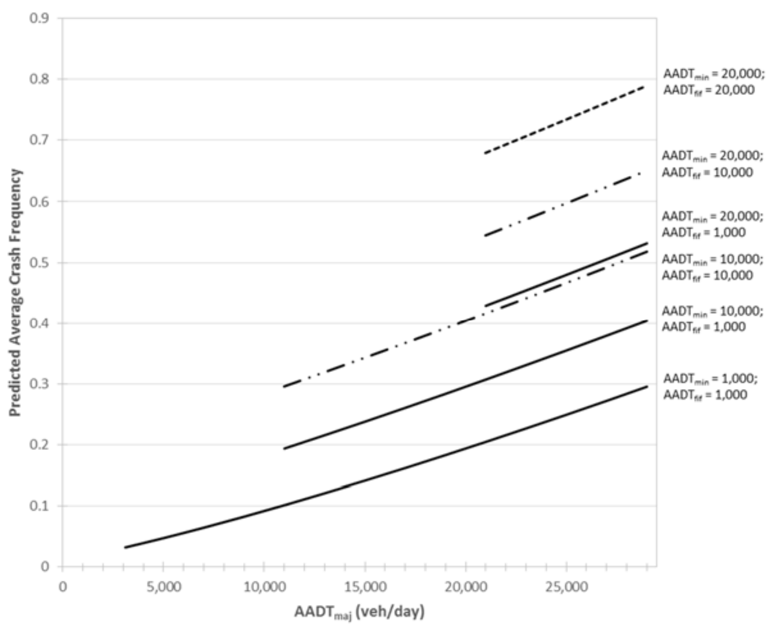


Figure 52. Graphical representation of the SPF for SV total crashes at five-leg intersections with signal control on urban and suburban arterials (based on model for SV total crashes in Table 66)

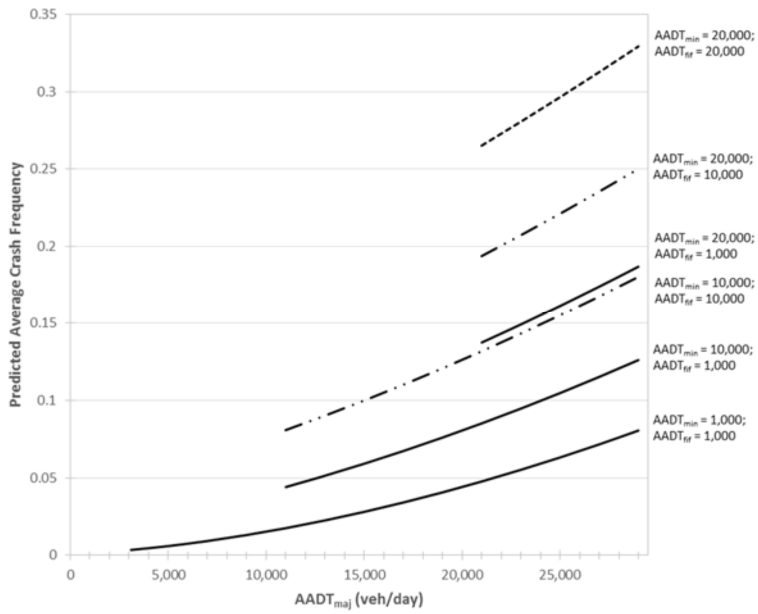


Figure 53. Graphical representation of the SPF for SV FI crashes at five-leg intersections with signal control on urban and suburban arterials (based on model for SV FI crashes in Table 66)

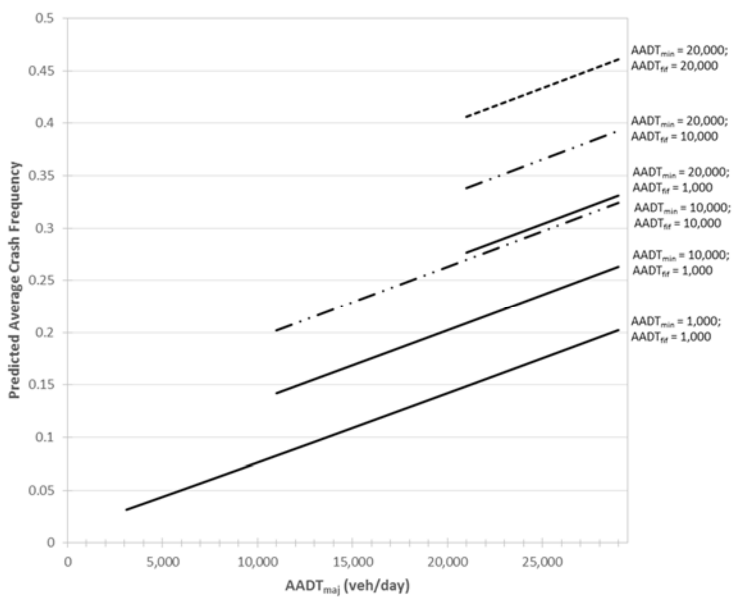


Figure 54. Graphical representation of the SPF for SV PDO crashes at five-leg intersections with signal control on urban and suburban arterials (based on model for SV PDO crashes in Table 66)

Tables 67 (MV crashes) and 68 (SV crashes) provide percentages to break down FI and PDO crash frequencies into collision types for five-leg intersections with signal control on urban and suburban arterials. These percentages were calculated based on all multiple- and SV crash counts at all intersections in all states combined. Tables 69 and 70 provide the distribution of pedestrian and bicycle crashes, respectively, for five-leg intersections with signal control on urban and suburban arterials.

Table 67. Distribution of MV crashes for five-leg intersections with signal control on urban and suburban arterials

Manner of Collision	Percentage of Multiple-Vehicle Crashes Five-Leg Signalized Intersections (5SG)	
	FI	PDO
Rear-end collision	42.5	43.1
Head-on collision	6.5	2.4
Angle collision	32.1	23.8
Sideswipe collision	4.9	16.9
Other MV collisions	13.9	13.9
Total MV crashes	**100.0**	**100.0**

Table 68. Distribution of SV crashes for five-leg intersections with signal control on urban and suburban arterials

Manner of Collision	Percentage of SV Crashes	
	Five-Leg Signalized Intersections (5SG)	
	FI	PDO
Collision with parked vehicle	0.0	5.5
Collision with animal	0.0	0.0
Collision with fixed object	31.0	20.5
Collision with other object	0.0	2.7
Other SV collision	62.1	71.2
Noncollision	6.9	0.0
Total SV crashes	**100.0**	**100.0**

Table 69. Distribution of pedestrian crash counts and percentage for five-leg intersections with signal control on urban and suburban arterials

Intersection Type	Number of Sites	Number of Pedestrian Crashes	Number of Total Crashes	Percentage of Pedestrian Crashes
Five-leg Signalized Intersections (5SG)	76	89	2850	**3.1**

Table 70. Distribution of bicycle crash counts and percentage for five-leg intersections with signal control on urban and suburban arterials

Intersection Type	Number of Sites	Number of Bicycle Crashes	Number of Total Crashes	Percentage of Bicycle Crashes
Five-leg Signalized Intersections (5SG)	76	88	2850	**3.1**

Following the development of the crash prediction models for five-leg intersections with signal control on urban and suburban arterials, the research team conducted compatibility testing of the new models to confirm that the new models provide reasonable results over a broad range of input conditions and that the new models integrate seamlessly with existing intersection crash prediction models in the first edition of the HSM. The graphical representations of the crash prediction models in Figures 49-54 provide some sense of the reasonableness of the new models for five-leg intersections with signal control on urban and suburban arterials. Nothing from these figures suggests that the models provide unreasonable results. In addition, several of the crash prediction models for five-leg intersections with signal control on urban and suburban arterials were compared to models for four-leg intersections with signal control on urban and suburban arterials in Chapter 12 of the HSM.

Figure 55 illustrates a comparison of the predicted average crash frequency for MV total crashes based on the five-leg intersection with signal control on urban and suburban arterials model in Table 61 to the corresponding predicted average crash frequency based on the 4SG model in Chapter 12 of the HSM. The dashed lines in the figure represent the predicted average crash frequency for the 5SG model, and the solid lines represent the predicted average crash frequency for the 4SG model in the HSM. For the comparisons, the traffic volumes used for the minor road and fifth-road in the 5SG model were combined and used for the traffic volume of the minor road for the 4SG model. As Figure 55 illustrates, for very low minor- and fifth-road volumes, fewer MV crashes are predicted for five-leg signalized intersections compared to four-leg signalized intersections. This seems reasonable as the right of way is more clearly defined for vehicles traveling through five-leg intersections due to the need for more signal phases. Then, as the minor- and fifth-road volumes increase, the predicted crashes for five-leg signalized intersections exceed the predicted crashes for four-leg signalized intersections. This seems reasonable as the signal phasing and operations for five-leg intersections with increasing minor- and fifth-road volumes would become more and more complex and would likely lead to more potential conflicts and a higher potential for crashes than similar volumes as four-leg signalized intersections.

In summary, the models for five-leg intersections with signal control on urban and suburban arterials appear to provide reasonable results over a broad range of input conditions and can be integrated seamlessly with existing intersection crash prediction models in the first edition of the HSM.

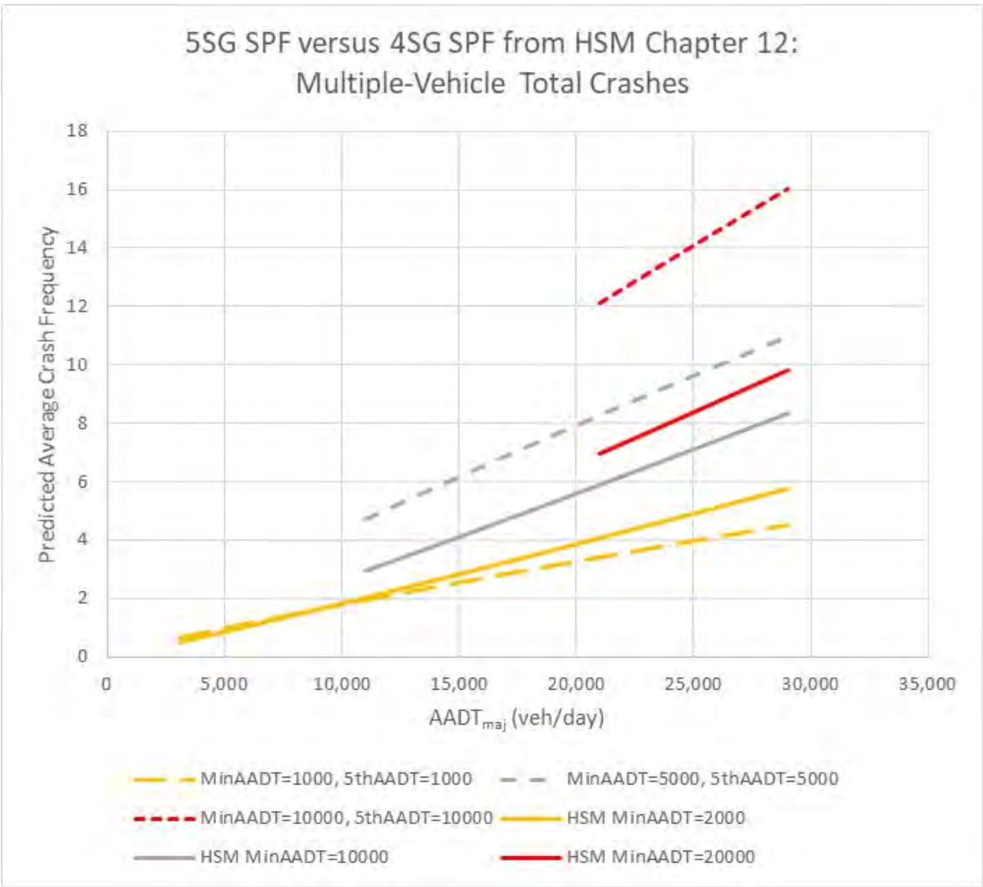


Figure 55. Comparison of new crash prediction model to existing model in HSM: 5SG for MV crashes for urban and suburban arterials vs 4SG for multiple-vehicle crashes from HSM Chapter 12 (total crashes)

6.4 Crash Modification Factors

During the development of the crash prediction models for urban five-leg signalized intersections, three potential sources of CMFs for use with the SPFs were considered:

- CMFs developed as part of this research based on a cross-sectional study design and regression modeling;
- CMFs already incorporated into the first edition of the HSM and applicable to urban five-leg signalized intersections; and
- High-quality CMFs applicable to urban five-leg signalized intersections developed using defensible study designs (e.g., observational before-after evaluation studies using SPFs – the EB method), as referenced in FHWA's CMF Clearinghouse with four or five-star quality ratings or based on a review of relevant intersection safety literature.

Based on the regression modeling as part of this research, no geometric features or traffic control devices were identified for CMF development. Also, based on a review of the CMFs already incorporated in the first edition of the HSM and other potential high-quality CMFs developed using defensible study designs, no CMFs were identified as applicable to urban five-leg signalized intersections and were considered of sufficient quality for use with the urban five-leg signalized intersection SPFs. Therefore, the SPFs for five-leg intersections with signal control on urban and suburban arterials were developed as AADT only models with no base conditions, and no CMFs are recommended for use with the SPFs provided in Section 6.4.

6.5 Severity Distribution Functions

Development of SDFs was explored for five-leg intersections with signal control on urban and suburban arterials using methods outlined in Section 2.2.3 of this report. SDFs were not used in the development of crash prediction methods in the first edition of the HSM but were subsequently used in the Supplement to the HSM for freeways and ramps (AASHTO, 2014). The database used to explore SDFs for five-leg intersections with signal control on urban and suburban arterials consisted of the same crashes and intersections as the database used to estimate the SPFs for five-leg intersections with signal control on urban and suburban arterials, but restructured so that the basic observation unit (i.e., database row) is a crash instead of an intersection. No traffic or geometric variables showed consistent and statistically significant effects in the SDFs for five-leg intersections with signal control on urban and suburban arterials.

6.6 Summary of Recommended Models for Incorporation in the HSM

In summary, several crash prediction models were developed for five-leg intersections with signal control on urban and suburban arterials for consideration in the second edition of the HSM, including models where:

- The fifth-road AADT was included separately as a predictor variable in the models
- The minor road and fifth-road AADTs were combined together as a predictor variable in the models
- The major road, minor road, and fifth-road AADTs were summed together as a predictor variable in the models

The final models recommended for inclusion in the second edition of the HSM include:

- The model for MV total crashes in Table 61
- The model for MV FI crashes in Table 62
- The model for MV PDO crashes inTable 61
- The model for SV total crashes in Table 66
- The model for SV FI crashes in Table 66
- The model for SV PDO crashes in Table 66

Attempts to develop SDFs for five-leg intersections with signal control on urban and suburban arterials proved unsuccessful for the reasons explained in Section 4.6. Therefore, it is recommended for the second edition of the HSM that crash severity for five-leg intersections be addressed in a manner consistent with existing methods in Chapter 12 of the HSM, without use of SDFs.

Appendix A presents recommended text for incorporating the final recommended models for five-leg intersections with signal control on urban and suburban arterials into Chapter 12 of the HSM.

Chapter 7.
Development of Models for Use in HSM Crash Prediction Methods: Three-Leg Intersections Where the Through Movements Make Turning Maneuvers at the Intersections

This section describes the development of crash prediction models for three-leg intersections, where the through movement makes a turning maneuver at the intersection, and presents the final models recommended for incorporation in the second edition of the HSM. Three-leg turning intersections (3STT) are implemented in both rural and urban areas; and due to the characteristics of this intersection type, they are almost always located on two-lane undivided roadways. Stop control can be used on only the minor road approach, or also on one of the major road approaches with a "Stop Except Right Turn" sign. These two configurations are shown in Figure 56.

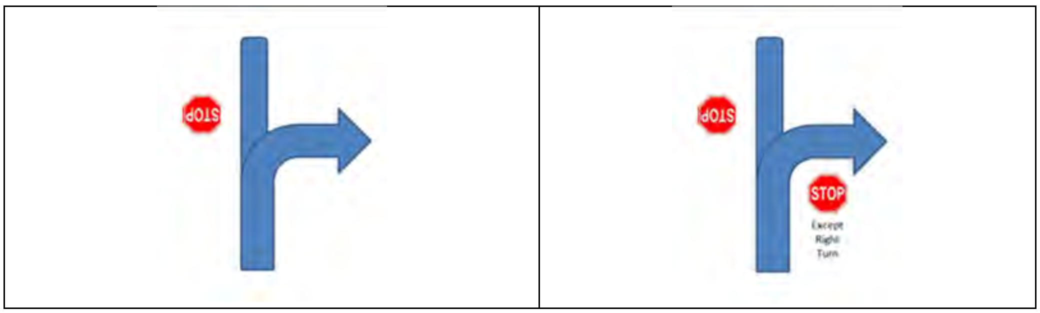


Figure 56. Three-leg turning intersection traffic control configurations

Section 7.1 describes the site selection and data collection processes for developing crash prediction models for three-leg turning intersections. Section 7.2 provides descriptive statistics of the databases used for model development. Section 7.3 presents the statistical analysis and resulting SPFs for three-leg turning intersections. Section 7.4 discusses the CMFs recommended for use with the SPFs. Section 7.5 addresses SDFs for three-leg turning intersections, and Section 7.6 provides recommendations for incorporating the new crash prediction models for three-leg turning intersections in the second edition of the HSM.

7.1 Site Selection and Data Collection

A list of potential three-leg intersections, where the through movement makes a turning maneuver at the intersection, was developed by searching databases and satellite imagery in three states:

- Kentucky (KY)
- Ohio (OH)
- Pennsylvania (PA)

Each intersection in the list was initially screened using Google Earth® to determine if the site was suitable for inclusion in model development. Several reasons a site could be deemed inappropriate for use in model development were:

- The traffic control at the intersection was something other than stop control or stop except right turn
- The number of intersection legs was not three
- A private driveway was located at the intersection
- One or more of the approaches to the intersection was a private/commercial access
- One or more of the intersection legs was a one-way street

Each intersection that was initially deemed appropriate for inclusion in model development was given a unique identification code and included in a refined database for detailed data collection.

Three types of data were collected for each intersection during detailed data collection: site characteristic, crash, and traffic volume data. Google Earth® was used to collect detailed site characteristics of the intersections. To reduce potential errors during data collection and to streamline data entry, a data collection tool was created using Visual Basic for Applications, very similar to the tool shown in Figure 4. The data collection tool was suited to only collect data relevant to three-leg intersections where the through movements make turning maneuvers at the intersections. Table 71 lists all of the intersection attributes collected (and respective definitions and permitted values) for three-leg intersections using the data collection tool. Once all necessary data were entered into the data collection tool and saved for a given intersection, the data collection tool was used to validate the inputs for that particular intersection, consistent with the range and/or permitted values for the respective variables/parameters.

Table 71. Site characteristic variables collected for three-leg intersections where the through movements make turning maneuvers at the intersections

Variable	Definition	Range or Permitted Values
General Intersection Attributes		
Area type (urban/rural)	Indicates whether the intersection is in a rural or urban area	Rural, urban
Presence of intersection lighting	Indicates if overhead lighting is present at the intersection proper	Yes, no
Presence of flashing beacons	Indicates if overhead flashing beacons are present at the intersection proper	Yes, no
Curve length	The horizontal curve length of the through movement at the intersection	Range: 40 to 383 ft
Curve radius	The horizontal curve radius of the through movement at the intersection	Range: 25 to 438 ft
Specific Approach Attributes		
Route name or number	Specify the route name or number of the approach	
Traffic control	The type of traffic control used on the approach	Uncontrolled, stop, stop except right turn, other
Number of through lanes	This includes dedicated through lanes and any lanes with shared movements. On the minor approach of a 3-leg intersection, if there is only one lane, then it should be classified as a through lane	0, 1, 2, 3
Presence/number of left-turn lanes	The number of lanes in which only a left-turn movement can be made	0, 1, 2
Left-turn channelization	Type of left-turn channelization used on the intersection approach	Raised or depressed island, painted, none
Presence/number of right-turn lanes	The number of lanes in which only a right-turn movement can be made	0, 1, 2

Table 71. Site characteristic variables collected for three-leg intersections where the through movements make turning maneuvers at the intersections (Continued)

Variable	Definition	Range or Permitted Values
Right-turn channelization	Type of right-turn channelization used on the intersection approach	Raised or depressed island, painted, none
Presence of transverse rumble strips	Indicates the presence of transverse rumble strips on the intersection approach	Yes, no, unknown
Presence/type of supplementary pavement markings	Indicates the presence of supplementary pavement markings on the intersection approach	Yes, no, unknown
Presence of stop ahead warning signs	Indicates the presence of stop ahead warning signs on the intersection approach	Yes, no, unknown
Presence of advance warning flashers	Indicates the presence of advance warning flashers on the intersection approach	Yes, no, unknown
Posted speed limit	Posted speed limit on the intersection approach	15, 20, 25, 30, 35, 40, 45, 50, 55, unknown
Presence of crosswalk	Indicates the presence of a crosswalk perpendicular to the intersection approach	Yes, no, unknown
Presence of bike lane	Indicates the presence of a marked bike lane parallel to the intersection approach	Yes, no, unknown
Presence of railroad crossing	Indicates the presence of a railroad crossing on the intersection approach within 250 ft of the intersection	Yes, no, unknown
Approach heading	Heading of the approach in the direction towards the intersection	0 to 359.99 degrees

During detailed data collection, to the extent possible, the research team reviewed historical aerial images to determine if a site had recently been reconstructed or improved to determine which years of data should be used in model development.

Crash and traffic volume data were obtained from the Kentucky Transportation Cabinet, the Ohio DOT, and the Pennsylvania DOT. The goal was to obtain the most recent four to six years of crash and traffic volume data for each site for model development. All of the data (i.e., site characteristics, crash, and traffic volume) were assembled into one database for the purposes of model development.

7.2 Descriptive Statistics of Database

Data for 242 sites—195 rural and 47 urban three-leg intersections—were initially available for development of crash prediction models for three-leg turning intersections. The data collection sites were located in three states—Kentucky, Ohio, and Pennsylvania. To remain consistent with the standards for development of the intersection predictive models in the first edition of the HSM, the goal of this research was to develop crash prediction models with a minimum of 200 site-years of data, and preferably 450 site-years of data or more.

7.2.1 Traffic Volumes and Site Characteristics

Of the intersection characteristics collected in Google Earth® (see Table 71), many showed no or very little variability across sites within a category (i.e., most intersections were predominantly of one type for a specific variable) and were thus excluded from modeling. The remaining variables (percent of "Yes" by area type indicated in parentheses) of potential interest in modeling were:

- Presence of intersection lighting (rural: 16%; urban: 51%)
- Presence of stop ahead warning signs (rural: 45%; urban: 36%)

Curve length and radius were also of potential interest for model development. The use of some of these site characteristics is discussed later in the SPF model development section (Section 7.3).

As explained in more detail in Section 7.3, a decision was made to use only unlighted intersections in rural areas and use both unlighted and lighted intersections in urban areas. Therefore, the summary statistics throughout this section are based on the 164 unlighted intersections in rural areas and the 47 unlighted and lighted intersections in urban areas.

Traffic volume and crash data were available for varying periods but were typically collected over a five- to ten-year period. Table 72 shows the breakdown of all sites by area type. Study period (date range), number of sites and site-years, and basic traffic volume statistics are shown by state in each category and across all states within a category.

Table 72. Major- and minor road AADT and total entering volume statistics for three-leg intersections where the through movements make turning maneuvers at the intersections

				Major Road AADT (veh/day)				Minor Road AADT (veh/day)				Total Entering Volume (veh/day)			
State	Date Range	Number of Sites	Number of Site-Years	Min	Max	Mean	Median	Min	Max	Mean	Median	Min	Max	Mean	Median
Rural[a]															
KY	2014-2018	41	205	46.0	7,663.0	890.6	391.0	50.0	1,362.0	161.2	92.0	71.0	8,344.0	971.2	508.5
OH	2008-2017	56	560	90.0	5,700.0	1,377.5	1,090.0	45.0	4,020.0	424.6	280.0	112.5	7,710.0	1,589.8	1,326.0
PA	2013-2017	67	335	95.5	2,727.0	636.8	402.0	16.0	2,797.0	390.6	273.0	118.0	3,042.0	832.1	539.0
All states	**2008-2018**	**164**	**1100**	**46.0**	**7,663.0**	**1,061.2**	**680.1**	**16.0**	**4,020.0**	**365.1**	**255.0**	**71.0**	**8,344.0**	**1,243.8**	**873.6**
Urban															
KY	2014-2018	19	95	642.0	17,688.0	4,587.8	2,610.0	50.0	4,181.0	506.7	129.0	673.0	17,752.5	4,841.2	2,663.5
OH	2008-2017	7	70	492.0	6,840.0	2,616.5	2,020.0	226.0	3,503.0	1,042.1	535.0	615.0	8,591.5	3,137.6	2,389.5
PA	2013-2017	21	105	795.0	11,431.5	4,120.8	3,468.5	93.0	5,787.0	2,535.3	2,281.0	1,086.0	14,325.0	5,388.5	4,671.0
All states	**2008-2018**	**47**	**270**	**492.0**	**17,688.0**	**3,895.1**	**2,976.0**	**50.0**	**5,787.0**	**1,434.4**	**636.0**	**615.0**	**17,752.5**	**4,612.3**	**3,151.5**

[a] Unlighted intersections only

7.2.2 Crash Counts

Intersection crashes were defined as those crashes that occurred within 250 ft of the intersection and were classified as "at intersection" or intersection-related, consistent with recommended practice in the HSM for assigning crashes to an intersection.

Of the 211 intersections used in model development, 87 intersections (41.2%) experienced no crashes over the entire study period.

Tables 73 (rural intersections) and 74 (urban intersections) show total, FI, and PDO crash counts by crash severity for each state over the entire study period. Counts are also shown for nighttime crashes only.

Table 73. Crash counts by crash severity for unlighted rural three-leg intersections where the through movements make turning maneuvers at the intersections

State	Date Range	Number of Sites	Number of Site-Years	Time of Day	All Crashes Combined			SV Crashes[a]			Multiple-Vehicle Crashes		
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO
KY	2014-2018	41	205	All	29	5	24	18	4	14	11	1	10
				Night	16	3	13	14	3	11	2	0	2
OH	2008-2017	56	560	All	301	116	185	214	82	132	87	34	53
				Night	147	54	93	129	48	81	18	6	12
PA	2013-2017	67	335	All	34	10	24	30	8	22	4	2	2
				Night	20	4	16	18	4	14	2	0	2
All states	**2008-2018**	**164**	**1100**	**All**	364	131	233	262	94	168	102	37	65
				Night	183	61	122	161	55	106	22	6	16

[a] Total and FI SV crashes include pedestrian and bicycle crashes.

Table 74. Crash counts by crash severity for urban three-leg intersections where the through movements make turning maneuvers at the intersections

State	Date Range	Number of Sites	Number of Site Years	Time of Day	All Crashes Combined			SV Crashes			Multiple-Vehicle Crashes			Pedestrian Crashes	Bicycle Crashes
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO	FI	FI
KY	2014-2018	19	95	All	49	5	44	15	4	11	34	1	33	0	0
				Night	11	3	8	7	3	4	4	0	4	0	0
OH	2008-2017	7	70	All	67	21	46	34	11	23	33	10	23	0	0
				Night	17	4	13	15	4	11	2	0	2	0	0
PA	2013-2017	21	105	All	61	26	35	29	11	18	32	15	17	2	0
				Night	32	13	19	19	8	11	13	5	8	2	0
All States	**2008-2018**	**47**	**270**	**All**	**177**	**52**	**125**	**78**	**26**	**52**	**99**	**26**	**73**	**2**	**0**
				Night	**60**	**20**	**40**	**41**	**15**	**26**	**19**	**5**	**14**	**2**	**0**

Crash counts are tallied by collision type and manner of collision across all states in Table 75 for rural three-leg turning intersections and in Table 76 for urban three-leg turning intersections.

Table 75. Crash counts by collision type and manner of collision and crash severity at unlighted rural three-leg intersections where the through movements make turning maneuvers at the intersections

Collision Type	Total	FI	PDO
SINGLE-VEHICLE CRASHES			
Collision with animal	26	0	26
Collision with bicycle	0	0	0
Collision with pedestrian	0	0	0
Overturned	14	9	5
Ran off road	208	80	128
Other SV crash	14	5	9
Total SV crashes	**262**	**94**	**168**
MULTIPLE-VEHICLE CRASHES			
Angle collision	66	26	40
Head-on collision	10	5	5
Rear-end collision	8	2	6
Sideswipe collision	14	3	11
Other MV collision	4	1	3
Total MV crashes	**102**	**37**	**65**
Total crashes	**364**	**131**	**233**

Table 76. Crash counts by collision type and manner of collision and crash severity at urban three-leg intersections where the through movements make turning maneuvers at the intersections

Collision Type	Total	FI	PDO
SINGLE-VEHICLE CRASHES			
Collision with parked vehicle	0	0	0
Collision with animal	2	0	2
Collision with fixed object	62	18	44
Collision with other object	3	0	3
Collision with pedestrian	2	2	0
Collision with bicycle	0	0	0
Other SV crash	7	4	3
Noncollision	2	2	0
Total SV crashes	**78**	**26**	**52**
MULTIPLE-VEHICLE CRASHES			
Angle collision	45	15	30
Head-on collision	8	3	5
Rear-end collision	16	3	13
Sideswipe collision	24	3	21
Other MV collision	6	2	4
Total MV crashes	**99**	**26**	**73**
Total crashes	**177**	**52**	**125**

7.3 Safety Performance Functions—Model Development

Intersection SPFs were developed in the forms illustrated by Equations 2, 53, and 54.

$$N_{spf\ int} = exp\left[a + b \times \ln\left(AADT_{maj}\right) + c \times \ln(AADT_{min})\right] \qquad \text{(Eq. 2)}$$

$$N_{spf\ int} = exp[a + d \times \ln(TEV)] \qquad \text{(Eq. 53)}$$

$$TEV = 0.5 \times \left[AADT_{maj,1} + AADT_{maj,2} + AADT_{min}\right] \qquad \text{(Eq. 54)}$$

Where:

$N_{spf\ int}$ = predicted average crash frequency for an intersection with base conditions (crashes/year)
$AADT_{maj}$ = AADT on the major road (veh/day)
$AADT_{min}$ = AADT on the minor road (veh/day)
$AADT_{maj,1}$ = AADT on major road approach 1 (veh/day)
$AADT_{maj,2}$ = AADT on major road approach 2 (veh/day)
TEV = total entering volume at intersection (veh/day)
$a, b, c,$ and d = estimated regression coefficients

For consistency with Chapters 10 and 12 in the HSM, an attempt was made to develop SPFs for the following crash severity levels and collision types:

- Rural three-leg turning intersections: total crashes, including pedestrian and bicycle crashes (similar to Equations 10-8 and 10-9 in the HSM)
- Urban three-leg turning intersections: total, FI, and PDO crashes (excluding pedestrian and bicycle crashes), separately for single- and MV crashes (similar to Equations 12-21 and 12-24 in the HSM)

For three-leg turning intersections on urban and suburban arterials, the SPFs were developed in a manner consistent with the methodology used in Chapter 12 of the HSM for predicting intersection crashes in urban and suburban areas. This methodology is illustrated in Equation 4 and Equation 5.

$$N_{predicted\ int} = \left(N_{bi} + N_{pedi} + N_{bikei}\right) \times\ C_i \qquad \text{(Eq. 4)}$$

$$N_{bi} = N_{spf\ int} \times \left(CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{yi}\right) \qquad \text{(Eq. 5)}$$

Where:

$N_{predicted\ int}$ = predicted average crash frequency for an individual intersection for the selected year (crashes/year)
N_{bi} = predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle crashes) (crashes/year)
N_{pedi} = predicted average crash frequency of vehicle-pedestrian crashes of an intersection (crashes/year)
N_{bikei} = predicted average crash frequency of vehicle-bicycle crashes of an intersection (crashes/year)
$N_{spf\ int}$ = predicted total average crash frequency of intersection-related crashes for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions) (crashes/year)
$CMF_{1i} \ldots CMF_{yi}$ = crash modification factors specific to intersection type *i* and specific geometric design and traffic control features *y*
C_i = calibration factor to adjust the SPF for intersection type *i* to local conditions

The SPF portion of N_{bi}, $N_{spf\ int}$, is the sum of two more disaggregate predictions by collision type, as shown in Equation 6.

$$N_{spf\ int} = N_{bimv} + N_{bisv} \qquad \text{(Eq. 6)}$$

Where:

N_{bimv} = predicted average crash frequency of MV crashes of an intersection for base conditions (crashes/year)
N_{bisv} = predicted average crash frequency of SV crashes of an intersection for base conditions (crashes/year)

Separate model structures are used to estimate the yearly number of vehicle-pedestrian (N_{pedi}) and vehicle-bicycle (N_{bikei}) crashes at three-leg turning intersections on urban and suburban arterials. The average number of annual vehicle-pedestrian and vehicle-bicycle crashes are estimated with Equations 9 and 12, respectively.

$$N_{pedi} = N_{bi} \times f_{pedi} \qquad \text{(Eq. 9)}$$

Where:

f_{pedi} = pedestrian crash adjustment factor for intersection type *i*

$$N_{bikei} = N_{bi} \times f_{bikei} \qquad \text{(Eq. 12)}$$

Where:

f_{bikei} = bicycle crash adjustment factor for intersection type *i*

All of the vehicle-pedestrian and vehicle-bicycle crashes predicted with Equations 9 and 12 are assumed to be FI crashes (none as PDO).

All SPFs were developed using a NB regression model based on all sites combined. Based on a review of the number of states, sites, site-years, and crashes for the database assembled, data for all sites were used for model development to maximize the sample size rather than using a portion of the data for model development and a portion for model validation. A significance level of 0.2 was used to assess the individual, estimated regression parameters. During model development, several intersection characteristics were initially tested in the models to develop CMFs for use with the SPFs. These characteristics included traffic control configuration, presence of intersection lighting, presence of stop ahead warning signs, and curve length and radius of the through movement at the intersection. Presence of stop ahead warning signs and traffic control configuration showed no consistent or statistically significant relationships to expected crash frequency.

For both rural and urban three-leg turning intersections, minor road AADT was not found to be statistically significant. Thus, total entering volume was used as an exposure variable in the models, which was estimated as half of the sum of the AADTs for the three intersection legs.

For the rural three-leg turning intersection model, presence of intersection lighting was found to be the only intersection characteristic that was statistically significant. Since there was an abundance of site-years, it was decided to exclude all intersections with lighting and derive a model based only on unlighted intersections. The CMF for presence of lighting in Chapter 10 of the HSM then could be used to adjust for lighted intersections (see discussion in Section 7.4).

For the urban three-leg turning intersection models, presence of intersection lighting was not found to be a statistically significant predictor of intersection crashes. However, horizontal curve length and radius were found to be statistically significant predictors in the models for total, FI, and PDO severity levels for MV crashes. Curve length was also found to be a statistically significant predictor of SV crashes for total, FI, and PDO severity levels. The effect that curve length and radius has on predicting crashes at urban three-leg turning intersections was converted into CMFs, which are presented in Section 7.4.

The statistical software known as "R" was used for developing models for SV and multi-vehicle FI crashes at three-leg turning intersections on urban and suburban arterials. SAS® Version 9 was used for all other modeling.

The final SPF for three-leg turning intersections in rural areas is provided in Table 77, for total severity using Equation 53. Table 77 shows the estimated model coefficients and overdispersion parameter (estimate), their standard errors, and associated p-values (and significance level). Figure 57 graphically presents the SPF in Table 77 for various major- and minor approach AADTs.

Table 77. SPF coefficients for three-leg turning intersections on rural two-lane roadways

Intersection Type	Parameter	Estimate	Standard Error	Pr > F	Significance Level
		TOTAL CRASHES[a]			
Three-Leg Turning Intersection	Intercept	-6.501	0.782	--	--
	ln(TEV)	0.703	0.099	<.001	Significant at 99% level
	Overdispersion	0.24	0.11	--	--

[a] Includes SV, MV, pedestrian, and bicycle crashes.
Base condition: absence of lighting.

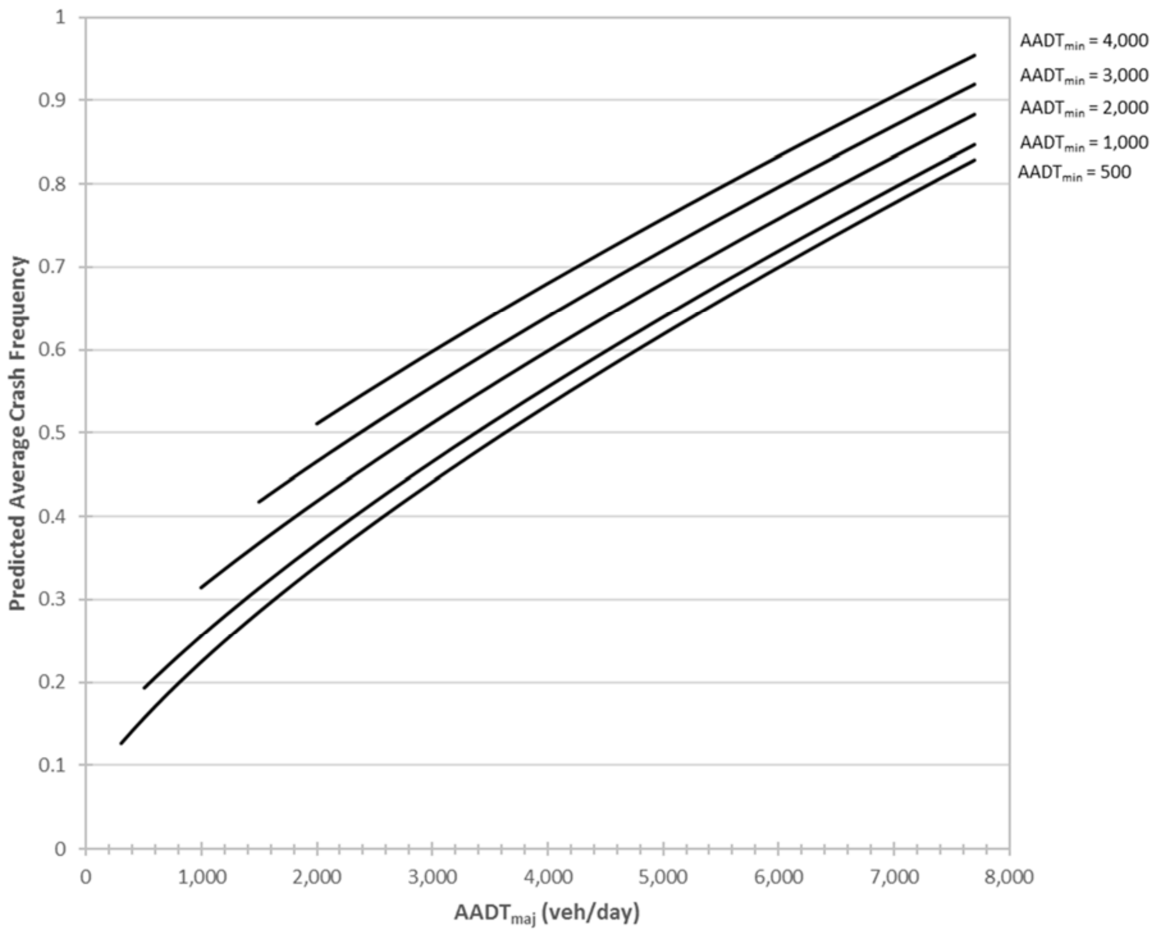


Figure 57. Graphical representation of the SPF for total crashes at three-leg turning intersections on rural two-lane roadways (based on model for total crashes in Equation 53)

Similar to Tables 10-5 and 10-6 in the first edition of the HSM, Tables 78 and 79 provide percentages for crash severity levels and for collision types and manner of collision, respectively, for rural three-leg turning intersections. These percentages were calculated based on all crash counts at all unlighted intersections in all states combined.

Table 78. Distributions for crash severity level at three-leg turning intersections on rural two-lane roadways

Crash Severity Level	Percentage of Total Crashes
Fatal	0.3
Incapacitating injury	6.0
Non-incapacitating injury	17.3
Possible injury	12.4
Total fatal plus injury	36.0
Property-damage-only	64.0
Total	**100.0**

Table 79. Distributions for collision type and manner of collision and crash severity at three-leg turning intersections on rural two-lane roadways

Collision Type	Percentage of Total Crashes by Collision Type		
	Total	FI	PDO
Single-Vehicle Crashes			
Collision with animal	7.1	0.0	11.2
Collision with bicycle	0.0	0.0	0.0
Collision with pedestrian	0.0	0.0	0.0
Overturned	3.8	6.9	2.1
Ran off road	57.1	61.1	54.9
Other SV crash	3.9	3.8	3.9
Total SV crashes	71.9	71.8	72.1
Multiple-Vehicle Crashes			
Angle collision	18.1	19.8	17.2
Head-on collision	2.8	3.8	2.1
Rear-end collision	2.2	1.5	2.6
Sideswipe collision	3.9	2.3	4.7
Other MV collision	1.1	0.8	1.3
Total MV crashes	28.1	28.2	27.9
Total crashes	100.0	100.0	100.0

Table 80 shows the coefficients and associated statistics of the final SPFs for urban three-leg turning intersections. Usable models were developed for multiple- and SV crashes separately for total, FI and PDO severity levels. Figures 58-63 graphically present the SPFs shown in Table 80 for various major- and minor approach AADTs.

SPFs for vehicle-pedestrian and vehicle-bicycle crashes at three-leg turning intersections on urban and suburban arterials could not be developed as pedestrian and bicycle volumes were not available.

Table 80. SPF coefficients for three-leg turning intersections on urban and suburban arterials

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
Multiple-Vehicle Crashes					
Total Crashes	Intercept	-8.49	1.95	--	--
	ln(TEV)	0.87	0.22	< .01	Significant at 99% level
	Overdispersion	0.32	0.22	--	--
FI Crashes	Intercept	-9.53	3.04	--	--
	ln(TEV)	0.81	0.33	0.01	Significant at 99% level
	Overdispersion	0.02	0.01	--	--
PDO Crashes	Intercept	-8.12	2.05	--	--
	ln(TEV)	0.79	0.23	< .01	Significant at 99% level
	Overdispersion	0.14	0.22	--	--
Single-Vehicle Crashes[a]					
Total Crashes	Intercept	-5.40	1.93	--	--
	ln(TEV)	0.46	0.23	0.05	Significant at 95% level
	Overdispersion	0.50	0.29	--	--
FI Crashes	Intercept	-4.69	3.29	--	--
	ln(TEV)	0.19	0.38	0.61	Not significant
	Overdispersion	0.00	0.00	--	--
PDO Crashes	Intercept	-6.68	2.21	--	--
	ln(TEV)	0.57	0.26	0.03	Significant at 95% level
	Overdispersion	0.61	0.44	--	--

[a] (i.e., pedestrian and bicycle crashes are excluded). Base condition is 100-ft long curve with a radius of 84 ft for the through route making a turning maneuver.

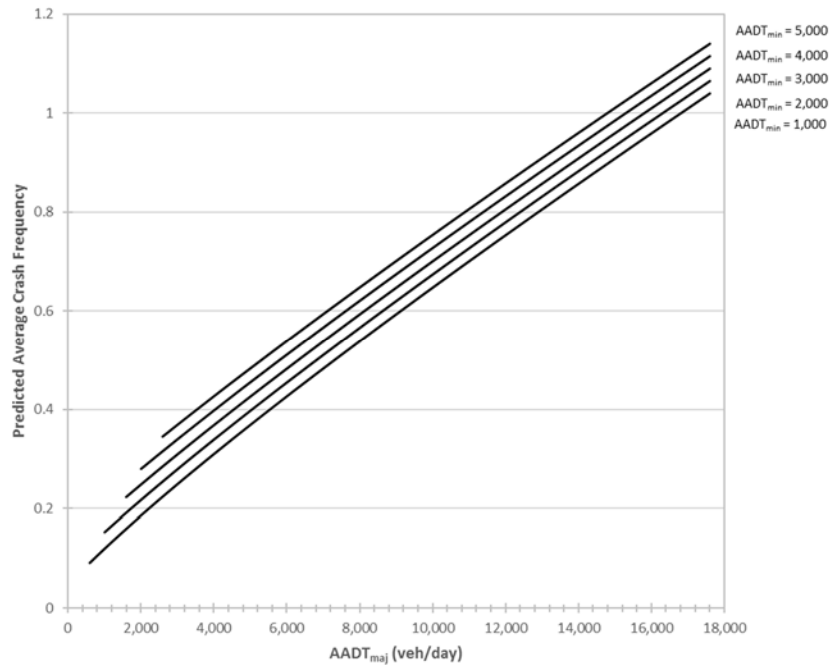


Figure 58. Graphical representation of the SPF for total MV crashes at three-leg turning intersections on urban and suburban arterials (based on model for total crashes in Equation 53)

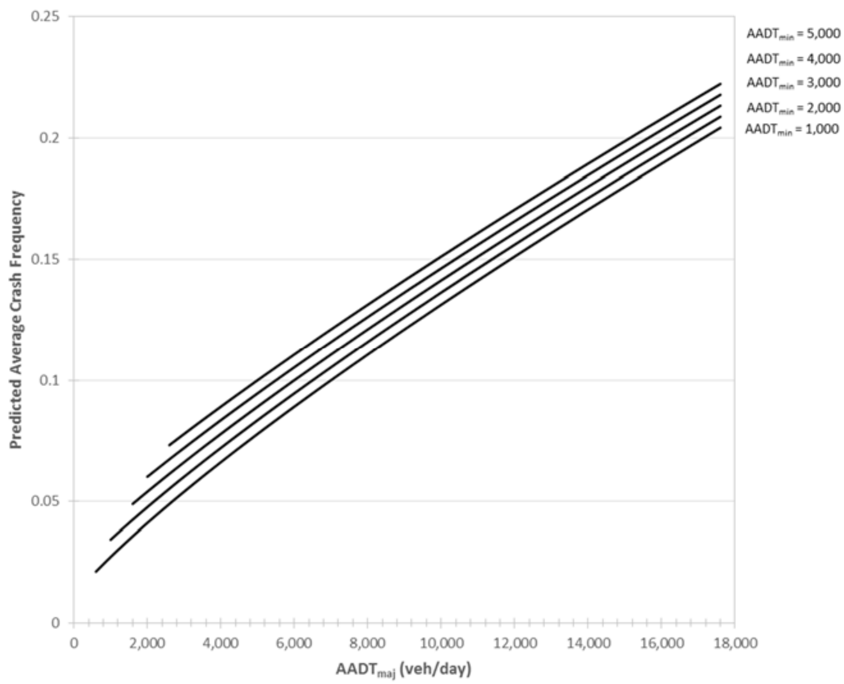


Figure 59. Graphical representation of the SPF for FI multiple-vehicle crashes at three-leg turning intersections on urban and suburban arterials (based on model for total crashes in Equation 53)

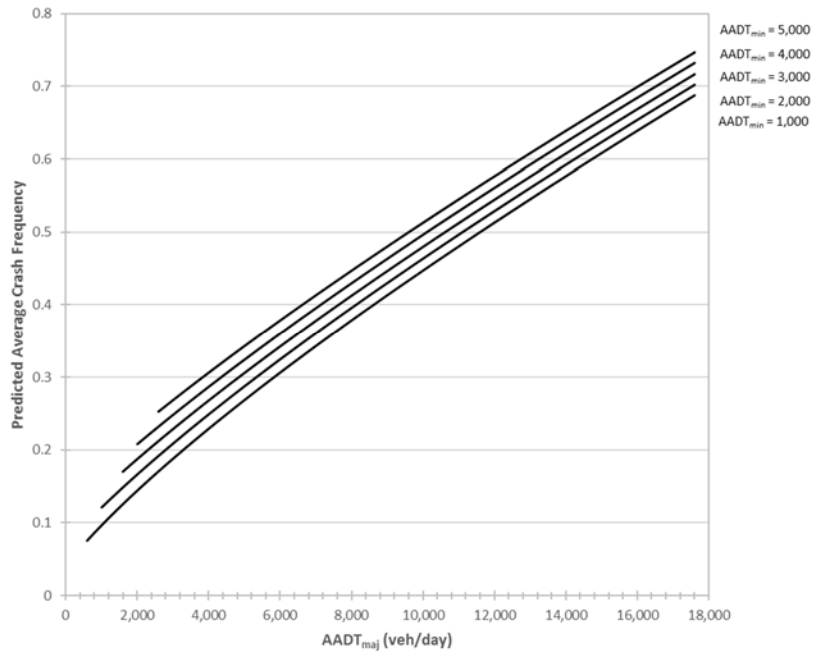


Figure 60. Graphical representation of the SPF for PDO multiple-vehicle crashes at three-leg turning intersections on urban and suburban arterials (based on model for total crashes in Equation 53)

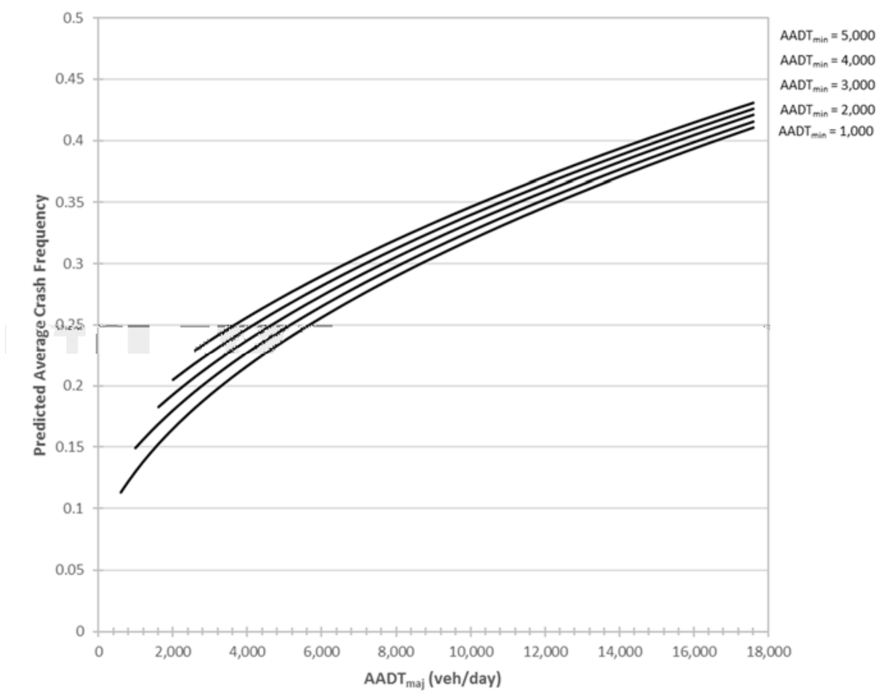


Figure 61. Graphical representation of the SPF for total SV crashes at three-leg turning intersections on urban and suburban arterials (based on model for total crashes in Equation 53)

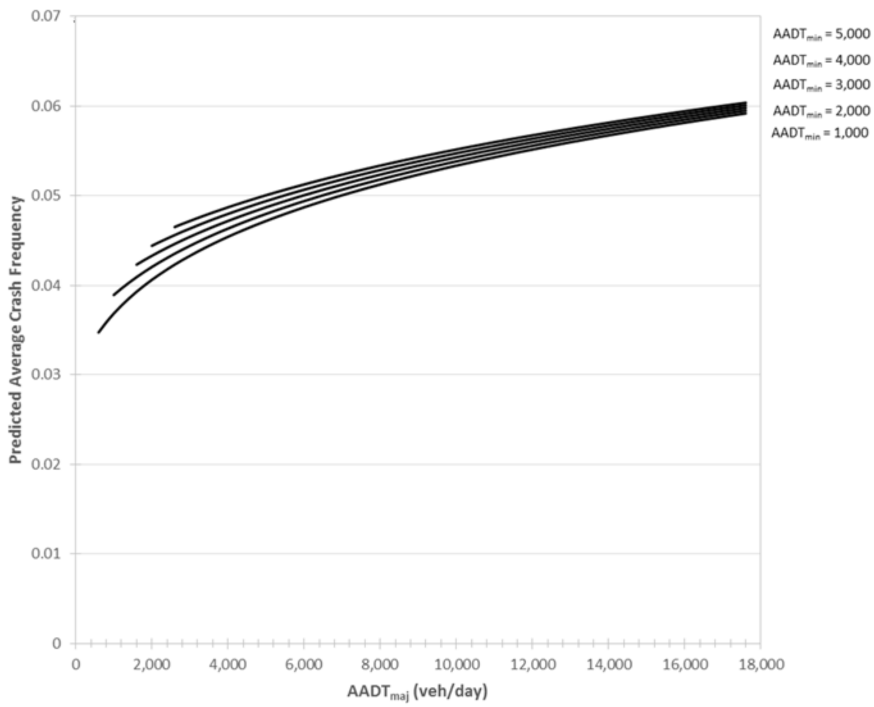


Figure 62. Graphical representation of the SPF for FI SV crashes at three-leg turning intersections on urban and suburban arterials (based on model for total crashes in Equation 53)

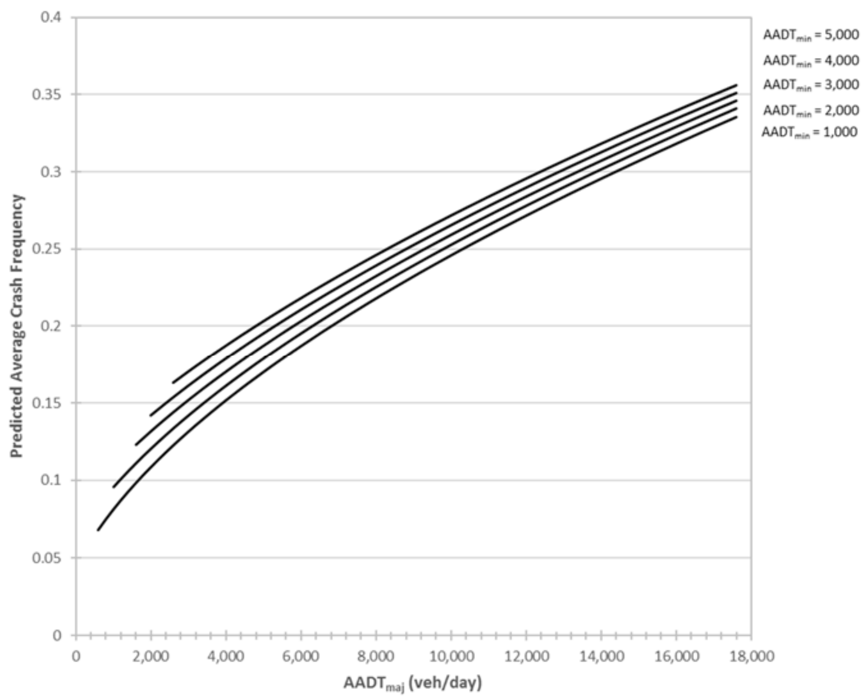


Figure 63. Graphical representation of the SPF for PDO single-vehicle crashes at three-leg turning intersections on urban and suburban arterials (based on model for total crashes in Equation 53)

Table 81 (similar to Table 79 for rural intersections) provides percentages of total crashes by collision type and severity level for urban three-leg turning intersections. These percentages were calculated based on all crash counts at all intersections—lighted and unlighted—in all states combined.

Table 81. Distributions for collision type and manner of collision and crash severity at three-leg turning intersections on urban and suburban arterials

Collision Type	Percentage of Total Crashes by Collision Type		
	Total	FI	PDO
Single-Vehicle Crashes			
Collision with parked vehicle	0.0	0.0	0.0
Collision with animal	1.1	0.0	1.6
Collision with fixed object	35.4	36.0	35.2
Collision with other object	1.7	0.0	2.4
Other SV collision	4.0	8.0	2.4
Noncollision	1.1	4.0	0.0
Total SV crashes	42.4	48.0	41.6
Multiple-Vehicle Crashes			
Angle collision	25.7	30.0	24.0
Head-on collision	4.6	6.0	4.0
Rear-end collision	9.1	6.0	10.4
Sideswipe collision	13.7	6.0	16.8
Other MV collision	3.4	4.0	3.2
Total MV crashes	56.6	52.0	58.4
Total crashes	100.0	100.0	100.0

For urban intersections, the predicted average crash frequency excludes vehicle-pedestrian and vehicle-bicycle crashes. To calculate a predicted average crash frequency of an intersection that includes vehicle-pedestrian and vehicle-bicycle crashes, the predictive model is given by

$$\boldsymbol{N_{predicted\ int}} = \boldsymbol{C_i} \times (\boldsymbol{N_{bi}} + \boldsymbol{N_{pedi}} + \boldsymbol{N_{bikei}}) \qquad \text{(Eq. 4)}$$

Where:

$N_{predicted\ int}$ = predicted average crash frequency for an individual intersection for the selected year (crashes/year)

N_{bi} = predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle crashes) (crashes/year)

N_{pedi} = predicted average crash frequency of vehicle-pedestrian crashes of an intersection (crashes/year)

N_{bikei} = predicted average crash frequency of vehicle-bicycle crashes of an intersection (crashes/year)

C_i = calibration factor to adjust the SPF for intersection type *i* to local conditions

Similar to Table 12-16 in the HSM, Table 82 provide a pedestrian crash adjustment factor for three-leg turning intersections on urban and suburban arterials. The number of vehicle-pedestrian crashes per year for a three-leg turning intersection is estimated as:

$$N_{pedi} = N_{bi} \times f_{pedi} \qquad \text{(Eq. 9)}$$

Where:

f_{pedi} = pedestrian crash adjustment factor for intersection type *i*

Table 82. Pedestrian crash adjustment factor for three-leg turning intersections on urban and suburban arterials

Intersection Type	Pedestrian Crash Adjustment Factor (f_{pedi})
Three-Leg Turning	0.011

Similar to Table 12-17 in the first edition of the HSM, Table 83 provides a bicycle crash adjustment factor for three-leg turning intersections on urban and suburban arterials. The number of vehicle-bicycle crashes per year for a three-leg turning intersection is estimated as:

$$N_{bikei} = N_{bi} \times f_{bikei} \qquad \text{(Eq. 12)}$$

Where:

f_{bikei} = bicycle crash adjustment factor for intersection type *i*

Table 83. Bicycle crash adjustment factor for three-leg turning intersections on urban and suburban arterials

Intersection Type	Bicycle Crash Adjustment Factor (f_{bikei})
Three-Leg Turning	0.000

Following the development of the crash prediction models for three-leg turning intersections in rural and urban areas, compatibility testing of the new models was conducted to confirm that the new models provide reasonable results over a broad range of input conditions and that the new models integrate seamlessly with existing intersection crash prediction models in the first edition of the HSM. The graphical representations of the crash prediction models in Figures 57-63 provide some sense of the reasonableness of the new models for three-leg turning intersections. Nothing from these figures suggests that the models provide unreasonable results. In addition, the new models for three-leg turning intersections were compared to the associated minor road stop-controlled intersection SPFs in the HSM. Figure 64 illustrates a comparison of the predicted average crash frequency for total crashes based on the rural 3STT model (Table 77) to the predicted average crash frequency based on the 3ST model in Chapter 10 of the HSM. In the figure, the dashed lines represent the predicted average crash frequency for the 3STT model, and the solid lines represent the predicted average crash frequency for the 3ST model in the HSM. Similarly, Figures 65-68 illustrate a comparison of the predicted average crash frequency for MV total crashes, MV FI crashes, SV total crashes, and SV FI crashes, respectively, based on the 3STT model for urban and suburban arterials (Table 80) to the predicted average crash frequency based on the 3ST models in Chapter 12 of the HSM. In all instances, as major road AADT increases, the 3STT multi-vehicle SPFs predict fewer crashes than the 3ST multi-vehicle SPFs in the HSM. It seems reasonable to expect fewer multi-vehicle crashes at 3STT intersections than 3ST intersections. Vehicle speeds on uncontrolled approaches of the intersection must slow down to navigate the horizontal curve. This gives vehicles on stop-controlled approaches potentially more time to assess gaps once vehicles on the uncontrolled approach(es) are seen. Also, slower speeds allow for longer reaction times to potentially avoid a collision at the intersection.

The SV total SPF for 3STT intersections tends to predict more crashes than the similar model for 3ST intersections from the HSM, especially as major approach volume increases. This may be due to the fact that the major road curves at the intersection, which may potentially lead to more crashes. The SV FI SPF for 3STT intersections follow a similar trend to the 3ST SV FI model from the HSM.

In summary, the models for three-leg turning intersections appear to provide reasonable results over a broad range of input conditions and can be integrated seamlessly with existing intersection crash prediction models in the first edition of the HSM.

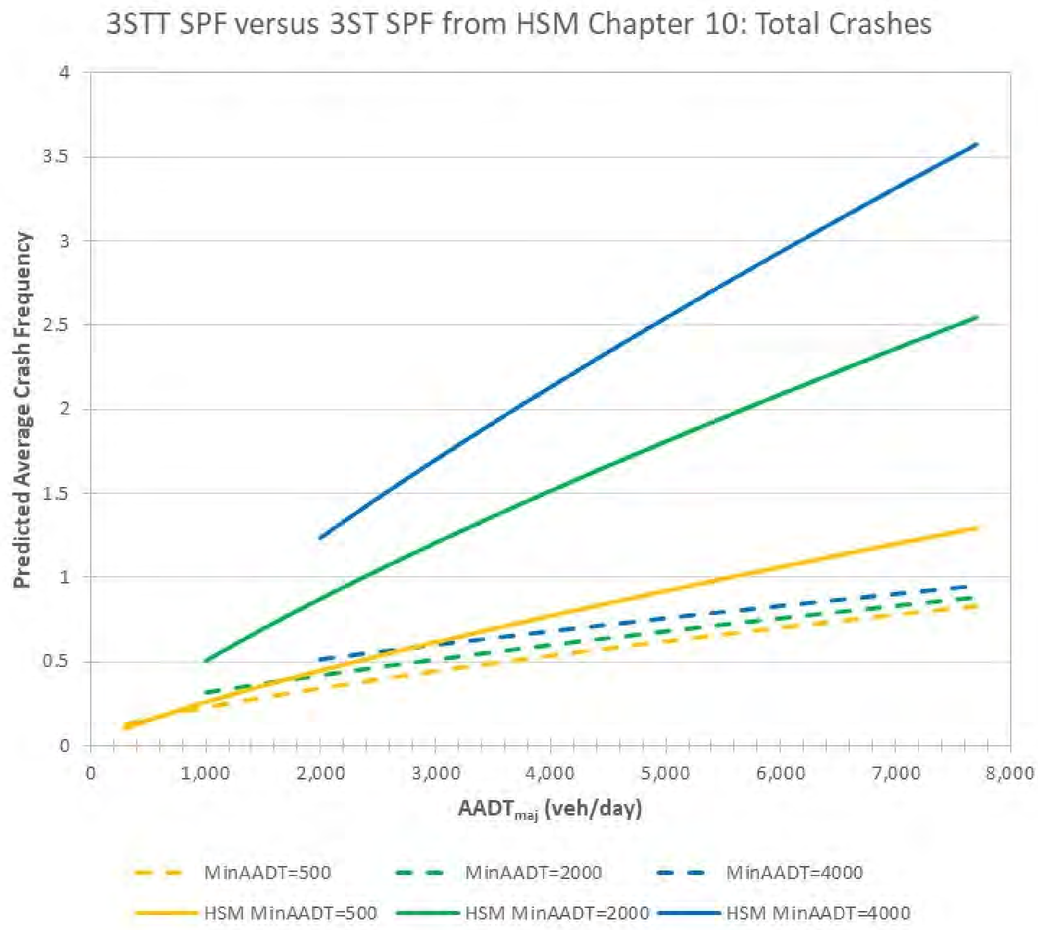


Figure 64. Comparison of new crash prediction model to existing model in HSM: 3STT vs 3ST on rural two-lane roads (total crashes)

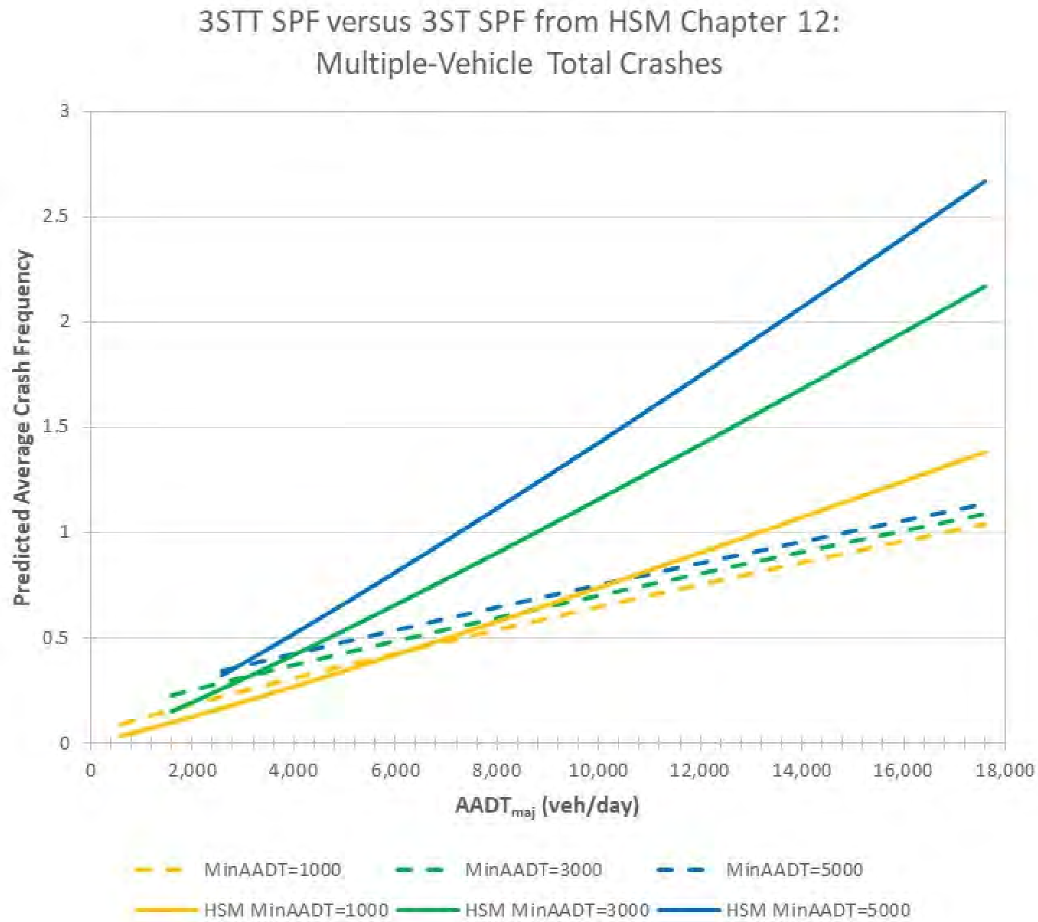


Figure 65. Comparison of new crash prediction model to existing model in HSM: 3STT vs 3ST on urban and suburban arterials (MV-total crashes)

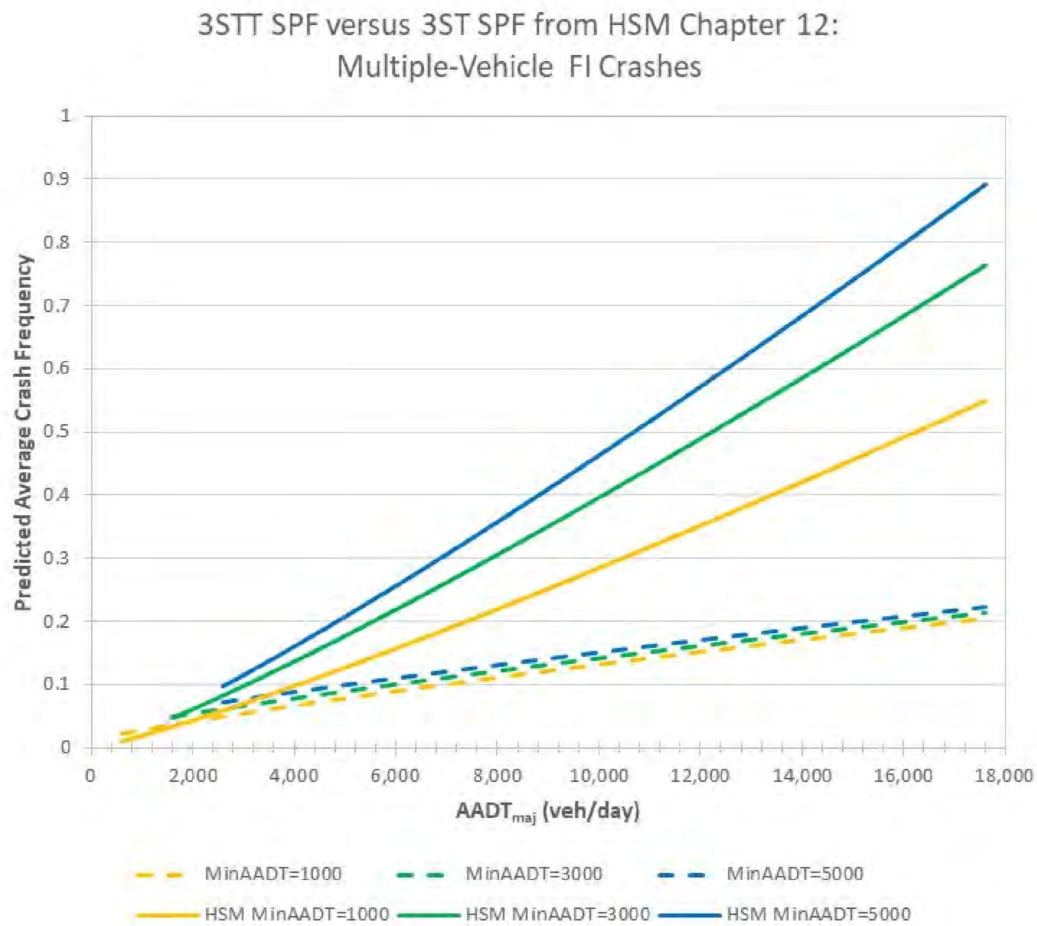


Figure 66. Comparison of new crash prediction model to existing model in HSM: 3STT vs 3ST on urban and suburban arterials (MV FI crashes)

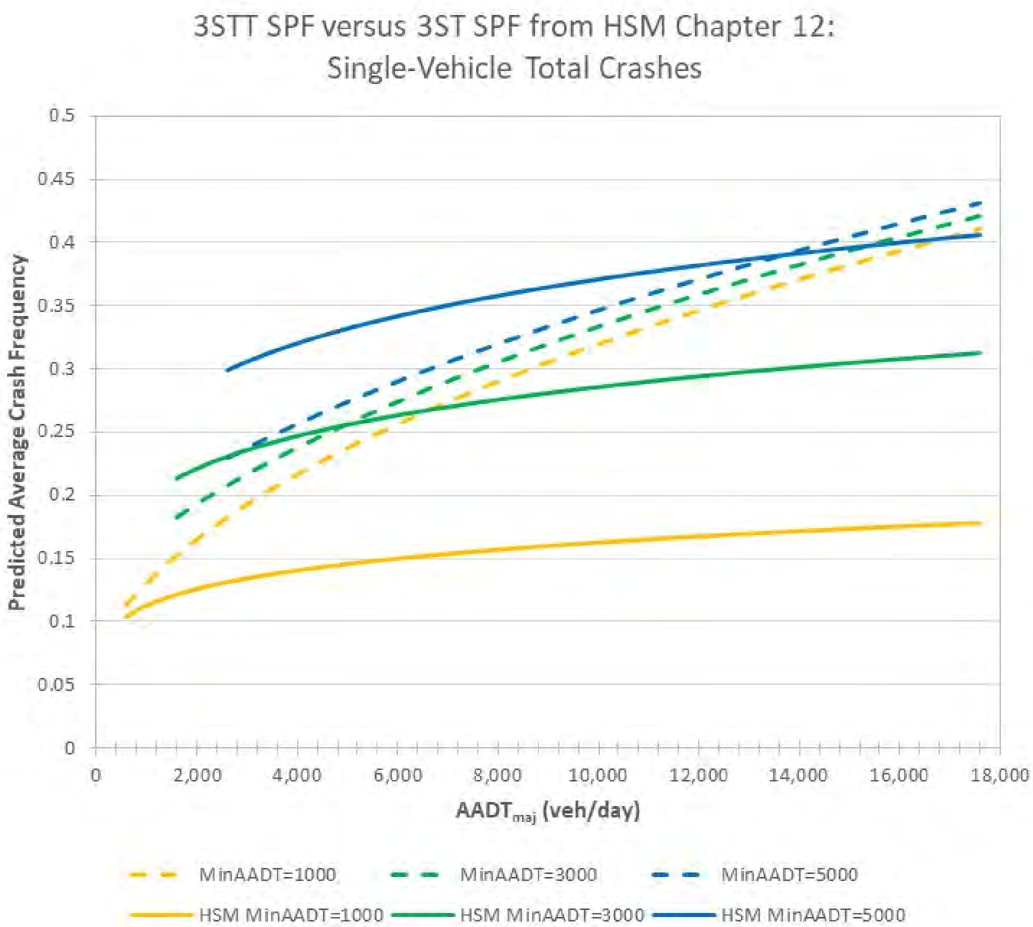


Figure 67. Comparison of new crash prediction model to existing model in HSM: 3STT vs 3ST on urban and suburban arterials (SV-total crashes)

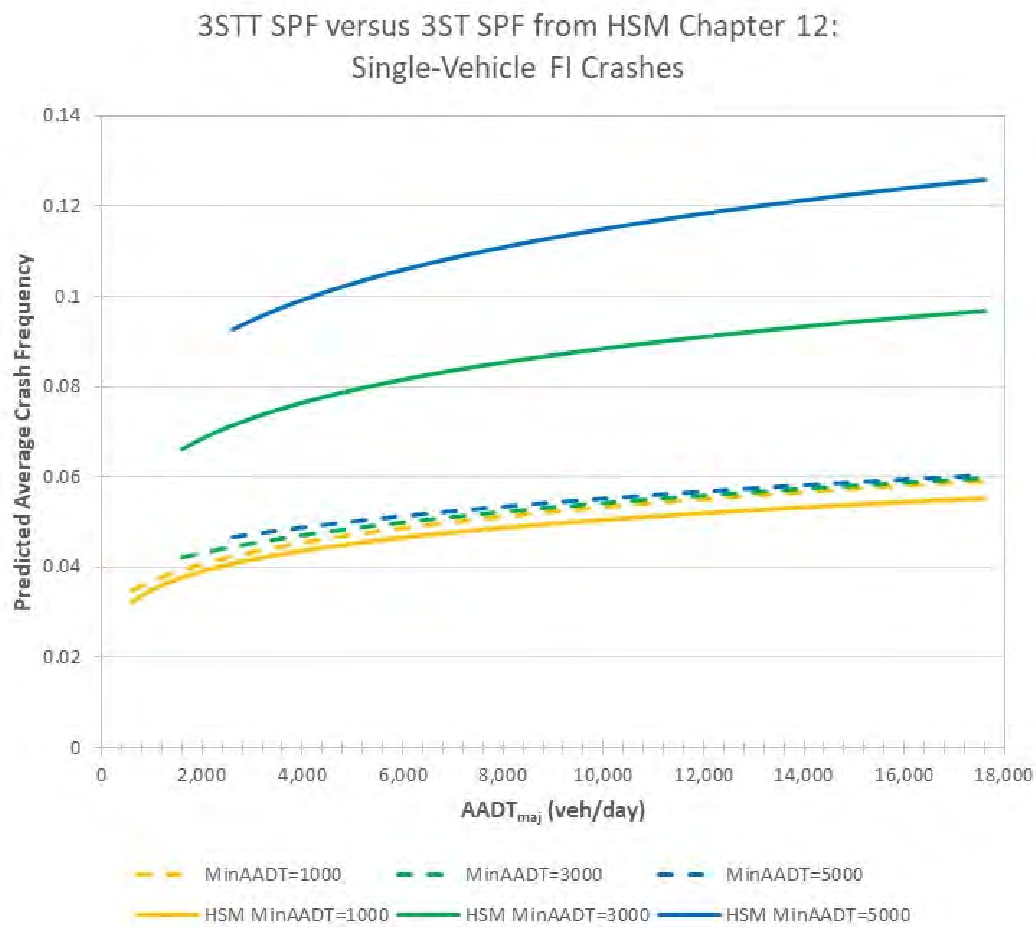


Figure 68. Comparison of new crash prediction model to existing model in HSM: 3STT vs 3ST on urban and suburban arterials (SV FI crashes)

7.4 Crash Modification Factors

During the development of the crash prediction models for three-leg turning intersections, three potential sources of CMFs for use with the SPFs were considered:

- CMFs developed as part of this research based on a cross-sectional study design and regression modeling
- CMFs already incorporated into the first edition of the HSM and applicable to three-leg turning intersections
- High-quality CMFs applicable to three-leg turning intersections developed using defensible study designs (e.g., observational before-after evaluation studies using SPFs—the EB method), as referenced in FHWA's CMF Clearinghouse with four- or five-star quality ratings or based on a review of relevant intersection safety literatures

Based on a review of the CMFs already incorporated in the first edition of the HSM and other potential high-quality CMFs developed using defensible study designs, the only CMF that was identified for potential use with the crash prediction models for rural three-leg turning intersections was the CMF for intersection lighting based on the work by Elvik and Vaa (2004), which is identified for use with the intersection crash prediction models in Chapters 10, 11, and 12 of the first edition of the HSM. Thus, the only CMF recommended for use with the final SPF for rural three-leg turning intersections is the CMF for intersection lighting based on the work by Elvik and Vaa (2004). With this CMF, the base condition is the absence of intersection lighting. The CMF for lighted intersections is similar to the CMF in Equation 10-24 in the HSM and has the form:

$$CMF_i = 1 - 0.38 \times p_{ni} \quad \text{(Eq. 39)}$$

Where:

CMF_i = crash modification factor for the effect of lighting on total crashes
p_{ni} = proportion of total crashes for unlighted intersections that occur at night

This CMF applies to total intersection crashes. Table 10-15 in the HSM presents values for the nighttime crash proportion, p_{ni}, by intersection type. Based on crash data used in this research, p_{ni} for rural three-leg turning intersections is 0.503.

Recent research by Washington State DOT has raised concerns about whether use of the lighting CMF in the HSM is appropriate. Based on their research, van Schalkwyk et al. (2016) concluded that the contribution of continuous illumination to nighttime crash reduction is negligible. However, this CMF is recommended for application to rural three-leg turning intersections because this CMF has been used in the first edition of the HSM. If any decision to remove or change the lighting CMFs is made, it should be done consistently for all facility types as part of the development of the second edition of the HSM.

Based on the regression modeling as part of this research, curve length and radius were also identified for CMF development for three-leg turning intersections on urban and suburban arterials. Therefore, a CMF was developed as part of this research for curve length and radius. The curve to which this CMF applies is the turn for the through movement. Curve length and radius are measured along the centerline of the roadway. The base condition of this CMF is a curve length equal to 100 ft, and a curve radius equal to 84 ft. This CMF was developed based on curves with radii ranging from 25 to 270 ft and lengths ranging from 40 to 240 ft. The CMF is presented in Equation 55 with accompanying coefficients shown in Table 84.

$$CMF_i = e^{a(R-84)+b(L_c-100)} \quad \text{(Eq. 55)}$$

Where:

CMF_i = crash modification factor for the effect of curve length and radius on crashes
R = curve radius (ft)
L_c = curve length (ft)
a, b = regression coefficients

Table 84 presents the values of the coefficients *a* and *b* used in applying Equation 55.

Table 84. CMF coefficients for curve CMF at three-leg turning intersections on urban and suburban arterials

SPF to which the CMF applies	Coefficients used in Equation 55	
	A	*b*
MV-Total	-0.014	0.017
MV FI	-0.014	0.019
MV PDO	-0.017	0.020
SV-Total	0[a]	0.009
SV FI	0[a]	0.013
SV PDO	0[a]	0.008

[a] Curve radius was not found to be statistically significant in predicting SV crashes

Tables 85-90 show computed curve CMF values for various crash types and severities by various levels of curve length and radius. There are certain combinations of curve length and radius that are not realistic for this intersection type and are not shown in these tables.

Table 85. Curve CMF values for MV total crashes

Curve Radius (ft)	Curve Length (ft)							
	50	75	100	125	150	175	200	225
25	0.976	--	--	--	--	--	--	--
50	0.688	1.052	1.610	2.462	--	--	--	--
75	0.485	0.742	1.134	1.735	2.654	4.059	--	--
100	0.342	0.523	0.799	1.223	1.870	2.861	4.375	6.693
125	--	0.368	0.563	0.862	1.318	2.016	3.083	4.716
150	--	0.259	0.397	0.607	0.929	1.420	2.173	3.323
175	--	--	0.280	0.428	0.654	1.001	1.531	2.342
200	--	--	0.197	0.301	0.461	0.705	1.079	1.650
225	--	--	--	0.212	0.325	0.497	0.760	1.163
250	--	--	--	0.150	0.229	0.350	0.536	0.820

Table 86. Curve CMF values for MV FI crashes

Curve Radius (ft)	Curve Length (ft)							
	50	75	100	125	150	175	200	225
25	0.883	--	--	--	--	--	--	--
50	0.623	1.001	1.610	2.588	--	--	--	--
75	0.439	0.705	1.134	1.824	2.933	4.716	--	--
100	0.309	0.497	0.799	1.285	2.067	3.323	5.344	8.593
125	--	0.350	0.563	0.906	1.456	2.342	3.766	6.056
150	--	0.247	0.397	0.638	1.026	1.650	2.654	4.267
175	--	--	0.280	0.450	0.723	1.163	1.870	3.007
200	--	--	0.197	0.317	0.510	0.820	1.318	2.119
225	--	--	--	0.223	0.359	0.578	0.929	1.493
250	--	--	--	0.157	0.253	0.407	0.654	1.052

Table 87. Curve CMF values for MV PDO crashes

Curve Radius (ft)	Curve Length (ft)							
	50	75	100	125	150	175	200	225
25	1.003	--	--	--	--	--	--	--
50	0.656	1.081	1.782	2.939	--	--	--	--
75	0.429	0.707	1.165	1.921	3.168	5.223	--	--
100	0.280	0.462	0.762	1.256	2.071	3.414	5.629	9.281
125	--	0.302	0.498	0.821	1.354	2.232	3.680	6.068
150	--	0.198	0.326	0.537	0.885	1.459	2.406	3.967
175	--	--	0.213	0.351	0.579	0.954	1.573	2.593
200	--	--	0.139	0.229	0.378	0.624	1.028	1.696
225	--	--	--	0.150	0.247	0.408	0.672	1.108
250	--	--	--	0.098	0.162	0.267	0.440	0.725

Table 88. Curve CMF values for SV total crashes

Curve Radius (ft)	Curve Length (ft)							
	50	75	100	125	150	175	200	225
25	0.638	--	--	--	--	--	--	--
50	0.638	0.799	1.000	1.252	--	--	--	--
75	0.638	0.799	1.000	1.252	1.568	1.964	--	--
100	0.638	0.799	1.000	1.252	1.568	1.964	2.460	3.080
125	--	0.799	1.000	1.252	1.568	1.964	2.460	3.080
150	--	0.799	1.000	1.252	1.568	1.964	2.460	3.080
175	--	--	1.000	1.252	1.568	1.964	2.460	3.080
200	--	--	1.000	1.252	1.568	1.964	2.460	3.080
225	--	--	--	1.252	1.568	1.964	2.460	3.080
250	--	--	--	1.252	1.568	1.964	2.460	3.080

Table 89. Curve CMF values for SV FI crashes

Curve Radius (ft)	Curve Length (ft)							
	50	75	100	125	150	175	200	225
25	0.522	--	--	--	--	--	--	--
50	0.522	0.723	1.000	1.384	--	--	--	--
75	0.522	0.723	1.000	1.384	1.916	2.651	--	--
100	0.522	0.723	1.000	1.384	1.916	2.651	3.669	5.078
125	--	0.723	1.000	1.384	1.916	2.651	3.669	5.078
150	--	0.723	1.000	1.384	1.916	2.651	3.669	5.078
175	--	--	1.000	1.384	1.916	2.651	3.669	5.078
200	--	--	1.000	1.384	1.916	2.651	3.669	5.078
225	--	--	--	1.384	1.916	2.651	3.669	5.078
250	--	--	--	1.384	1.916	2.651	3.669	5.078

Table 90. Curve CMF values for SV PDO crashes

Curve Radius (ft)	Curve Length (ft)							
	50	75	100	125	150	175	200	225
25	0.670	--	--	--	--	--	--	--
50	0.670	0.819	1.000	1.221	--	--	--	--
75	0.670	0.819	1.000	1.221	1.492	1.822	--	--
100	0.670	0.819	1.000	1.221	1.492	1.822	2.226	2.718
125	--	0.819	1.000	1.221	1.492	1.822	2.226	2.718
150	--	0.819	1.000	1.221	1.492	1.822	2.226	2.718
175	--	--	1.000	1.221	1.492	1.822	2.226	2.718
200	--	--	1.000	1.221	1.492	1.822	2.226	2.718
225	--	--	--	1.221	1.492	1.822	2.226	2.718
250	--	--	--	1.221	1.492	1.822	2.226	2.718

7.5 Severity Distribution Functions

Based on previous results of attempting to develop SDFs for intersections, it was decided not to explore the development of SDFs for three-leg turning intersections.

7.6 Summary of Recommended Models for Incorporation in the HSM

In summary, several crash prediction models were developed for three-leg intersections where the through movements make turning maneuvers at the intersections for consideration in the second edition of the HSM, including models for:

- Three-leg turning intersections on rural two-lane roadways
- Three-leg turning intersections on urban and suburban arterials

The final models recommended for inclusion in the second edition of the HSM are for total crashes on three-leg turning intersections on rural two-lane roadways (as shown in Table 77) and MV total, MV FI, MV PDO, SV total, and SV PDO crashes at three-leg turning intersections on urban and suburban arterials (as shown in Table 80). The model for SV FI crashes for three-leg turning intersections on urban and suburban arterials in Table 80 is not recommended for inclusion in the second edition of the HSM because the parameter for total entering volume is not statistically significant in the model.

In addition, SDFs were not developed for three-leg turning intersections. Therefore, it is recommended for the second edition of the HSM that crash severity for three-leg turning intersections on rural two-lane highways and urban and suburban arterials be addressed in a manner consistent with existing methods in Chapter 10 and Chapter 12 of the HSM, respectively, without use of SDFs.

Appendix A presents recommended text for incorporating the final recommended models for three-leg turning intersections into Chapters 10 and 12 of the HSM.

Chapter 8. Development of Models for Use in HSM Crash Prediction Methods: Crossroad Ramp Terminals at Single-Point Diamond Interchanges

This section describes the development of crash prediction models for crossroad ramp terminals at single-point diamond interchanges (SPs). Single-point diamond interchanges are implemented in urban areas. Their crossroad ramp terminals are characterized by one intersection through which all at-grade traffic movements are made (Leisch, 2005). Section 8.1 describes the site selection and data collection processes for developing crash prediction models for crossroad ramp terminals at single-point diamond interchanges. Section 8.2 provides descriptive statistics of the databases used for model development. Section 8.3 presents the statistical analysis and resulting SPFs for crossroad ramp terminals at single-point diamond interchanges. Section 8.4 discusses the CMFs recommended for use with the SPFs. Section 8.5 addresses the outcomes of the analysis to develop SDFs for crossroad ramp terminals of single-point diamond interchanges. Section 8.6 provides recommendations for incorporating the new crash prediction models for crossroad ramp terminals at single-point diamond interchanges in the second edition of the HSM.

8.1 Site Selection and Data Collection

A list of potential single-point diamond interchanges was developed by searching databases and satellite imagery in five states:

- Arizona (AZ)
- Missouri (MO)
- Nevada (NV)
- Tennessee (TN)
- Utah (UT)

Data collection activities for these sites included gathering geometric design attributes of the interchanges as well as traffic and crash data. Geometric attributes were collected from aerial imagery in Google Earth®, as well as Google Street View®. Table 91 lists the geometric attributes collected (and respective definitions and permitted values) for each single-point diamond interchange.

Table 91. Site characteristic variables collected for crossroad ramp terminals at single-point diamond interchanges

Variable	Definition	Range or Permitted Values
General Intersection Attributes		
Intersection configuration (i.e., number of legs and type of traffic control)	Indicates the number of legs and type of traffic control	4SG
Area type	Indicates whether the intersection is in a rural or urban area	Urban
Presence of intersection lighting	Indicates if overhead lighting is present at the intersection proper	Yes, no
Crossroad over or under freeway	Indicates whether the crossroad passes over or under the freeway	Over or under
Construction year	Estimated year when the interchange was constructed	Range: 1992 to 2014

Table 91. Site characteristic variables collected for crossroad ramp terminals at single-point diamond interchanges (Continued)

Approach Specific Attributes		
Route name or number	Specifies the route name or number of the approach	
Location at intersection	Side of the intersection the approach is located	Primary, secondary
Presence of left-turn lanes	The number of approaches with one or more left-turn lanes	4
Number of left-turn lanes	Number of left-turn lanes provided for turning movements to/from each freeway ramp	0, 1, 2, 3
Left-turn protected only	Number of approaches with protected only left-turn options	4
Presence of right-turn lane	Number of approaches with one or more right-turn lanes	0,1,2,3,4
Number of right-turn lanes	Number of right-turn lanes provided for turning movements to/from each freeway ramp	0, 1, 2, 3
Number of through lanes	Number of through lanes present on each crossroad approach to the crossroad ramp terminal	1, 2, 3, 4
Presence of frontage roads	Indicates the presence of frontage roads at the interchange, where a through movement is added between the exit and entrance ramps	Yes, no
Presence of crosswalk	Indicates the presence of crosswalks at the crossroad ramp terminal	Yes, no
Presence of bike lane	Indicates the presence of a bike lane on the crossroad at the crossroad ramp terminal	Yes, no
Median width	Width of median (in feet) on each crossroad approach to the crossroad ramp terminal	Range: 0 to 47 ft
Median type	Type of median present on each crossroad approach to the crossroad ramp terminal	Raised, flush, depressed, none
Skew angle	The intersection skew angle of the freeway mainline and crossroad	Range: 0 to 90 degrees
Number of driveways	Number of driveways located within 250 ft of the crossroad stop bars/lines	Range: 0 to 10
Presence of public street approach	Indicates if an unsignalized public street approach is present within 250 ft of a crossroad stop bar/line	Yes, no
Presence of railroad crossing	Indicates the presence of a railroad crossing on the crossroad	Yes, no
Freeway posted speed limit	The posted speed limit on the freeway mainline	Range: 45 to 75 mph
Crossroad posted speed limit	The posted speed limit on the crossroad	Range: 30 to 55 mph
Terminal length	The distance measured along the crossroad between the outermost ramp terminal boundaries	Range: 468 to 2274 ft
Traffic control type for right turns	Type of traffic control for right-turn movements	Signal, stop, yield, none
U-turns allowed	Indicates if a U-turn is allowed between exit ramps and entrance ramps	Yes, no
Distance to right-turn approach	Distance from the center of the crossroad ramp terminal to the center of the right turn approach	Range: 100 to 1654 ft

The “construction year” was estimated using the “Clock” feature in Google Earth® as the earliest year with the interchange present in aerial imagery. Some single-point diamond interchanges in the database were built during the study period and therefore had fewer years of data available for analysis. Additional information about the interchange configuration was used to exclude sites with uncommon or inconsistent geometric conditions, such as the lack of a crossroad approach or ramp approach.

Traffic data collection activities primarily involved accessing publicly available traffic volumes and statistics.

Crash data were obtained from state DOTs. The crash data generally included details about the crash location (geographic coordinates), as well as attributes describing the crash, people involved in the crash, and the road and environmental conditions at the location and time of the crash.

Identifying crashes associated with the ramp terminal required a clear definition of a ramp terminal-related crash based on geographic location and crash attributes. To maintain a level of consistency with the ramp terminal models in NCHRP Project 17-45, these crashes were selected using the following criteria:

- Crashes occurring on the crossroad within the ramp terminal boundary, defined as a point 100 ft from the gore or curb return of the outermost ramp connection, and having one of the following attributes:
  - at intersection
  - intersection-related
  - at driveway
  - driveway-related
  - involving a pedestrian or bicyclist
- Crashes occurring on a ramp with at least one of the following attributes:
  - at intersection;
  - intersection-related;
  - involving a pedestrian or bicyclist, or
  - located on an exit ramp and manner of collision is rear-end.

This definition departs from the NCHRP Project 17-45 ramp terminal definition, using a different distance reference to define the crossroad ramp terminal boundary. The NCHRP Project 17-45 definition used 250 ft from the crossroad ramp terminal, measured from the center of the intersection. The definition implemented for the crossroad ramp terminals of single-point diamond interchanges is based on the American National Standards Institute (ANSI) D16.1-2007 (Manual on Classification of Motor Vehicle Traffic Accidents) definition of an interchange crash. According to the ANSI definition, an interchange crash is a crash in which the first harmful event occurs within a boundary defined by a point 100 ft from the gore or curb return of the outermost ramp connection. Figure 69 illustrates the boundaries for defining ramp terminal crashes at a single-point diamond interchange.

This ramp terminal boundary adjustment was necessary for this application due to the size of a typical crossroad ramp terminal at a single-point diamond interchange and its main characteristic of operating as one intersection. Figure 70 shows an example of a single-point diamond interchange with a crossroad terminal size/length approximately equal to the average of terminal sizes/lengths at sites in Arizona and Utah. At this location, the maximum distance between the center of the interchange and the outermost ramp connection is approximately 330 ft, more than the 250 ft used in the NCHRP Project 17-45 definition. As a result, using the 250 ft boundary would have resulted in missing crashes associated with right turn movements at the entrance and exit ramps. It would have also resulted in missing part of the longer left-turn lanes on the cross street that are common to crossroad ramp terminals at single-point diamond interchanges. The ramp terminal boundary that is based on the ANSI definition and implemented in this research extends 100 ft beyond the outermost ramp connections, capturing crashes associated with the right-turn movements and the left-turn lanes.

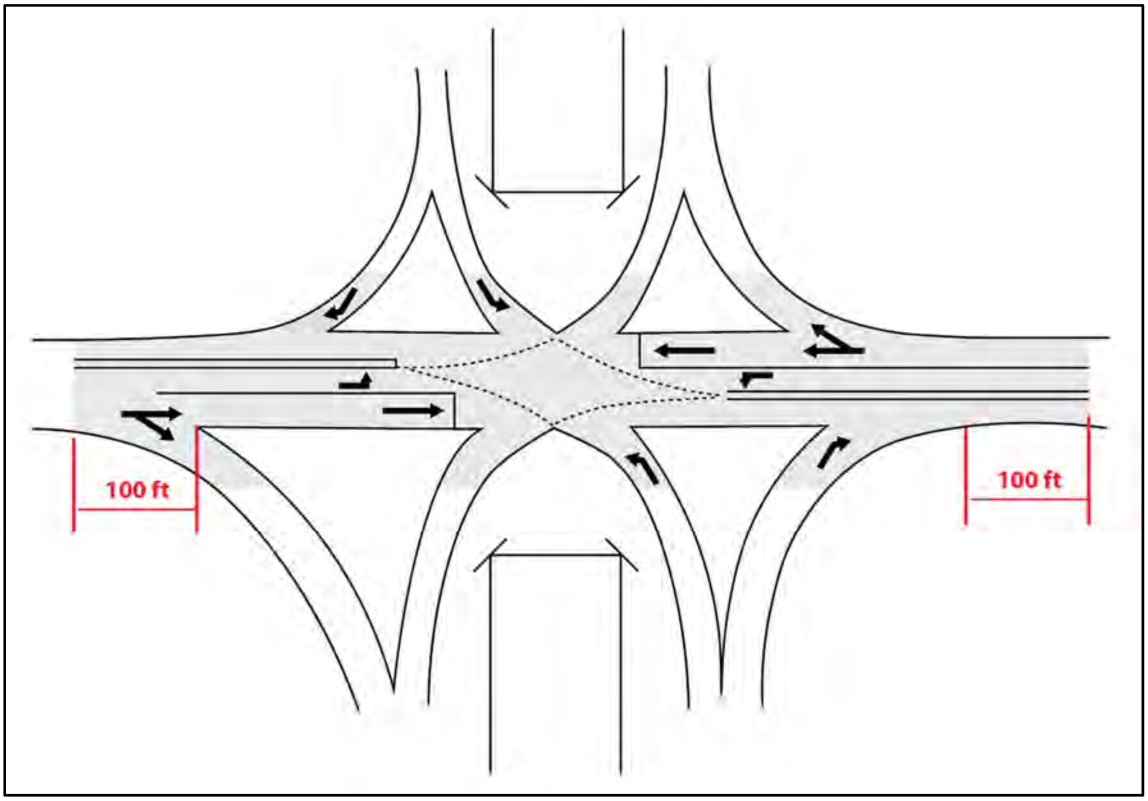


Figure 69. Single-point diamond interchange ramp terminal boundaries for defining ramp terminal crashes (adapted from Bonneson et al., 2012)

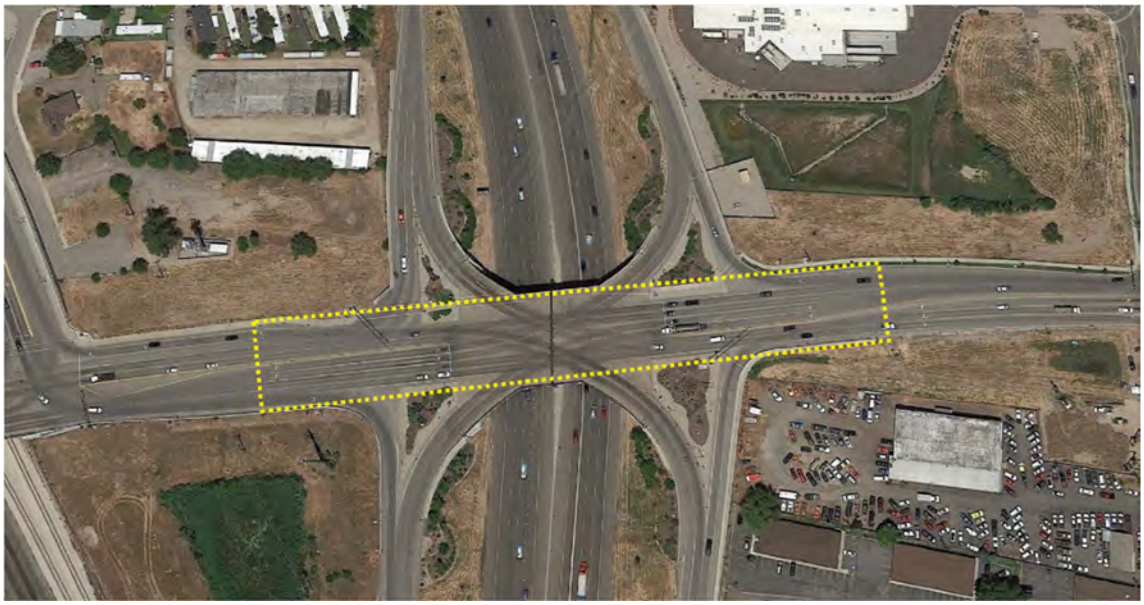


Figure 70. Example of a single-point diamond interchange with the implemented ramp terminal boundary identified along the crossroad (Source: ArcMap)

All of the collected data (i.e., site characteristics, crashes, and traffic volumes) were assembled into one database for the purposes of model development. After initial database development and quality assessments, interchanges in Arizona and Utah were selected for model development due to a higher level of confidence in accurately and reliably locating and identifying terminal-related crashes in those states. This decision resulted in 70 potential crossroad ramp terminals for model development. This list of interchanges was further reduced due to unusual geometric attributes and missing traffic data. Specifically, 12 sites were excluded due to missing ramp volumes on at least one ramp approach, four were excluded for unusual ramp terminal configurations (e.g., exit ramp integration with nearby intersections or streets), and two were excluded for unusual crossroad configurations (e.g., missing crossroad approach, resulting in a three-leg variation of a single-point diamond interchange).

With 52 potential sites remaining for model development, cumulative residual (CURE) plots for preliminary models indicated three potential outliers were present in the database. These locations generally had an excessive number of PDO crashes relative to their reported traffic volumes on the crossroads and ramps, resulting in unusually large residuals. The final database excluded these three sites, resulting in 49 crossroad ramp terminals at single-point diamond interchanges for model development.

8.2 Descriptive Statistics of Database

A total of 49 crossroad ramp terminals at single-point diamond interchanges were used for crash prediction model development. The selected sites were from two states: Arizona and Utah. To remain consistent with the standards for development of the intersection predictive models in the first edition of the HSM, the goal of this research was to develop crash prediction models with a minimum of 200 site-years of data, and preferably 450 site-years of data or more.

8.2.1 Traffic Volumes and Site Characteristics

Traffic volumes and crash data from years 2011 through 2015 were used for analysis. Table 92 provides summary statistics for traffic volume at the study sites used for model development. Study period (date range), number of sites and site-years, and traffic volume descriptive statistics are shown by state.

Table 92. Crossroad and ramp AADT statistics at single-point diamond interchange crossroad ramp terminals

State	Date Range	Number of Sites	Number of Site-Years	Crossroad AADT (veh/day)				Ramp AADT (sum of all four ramps) (veh/day)			
				Min	Max	Mean	Median	Min	Max	Mean	Median
AZ	2011-2015	28	140	14,934	70,790	36,169	36,302	16,556	64,648	40,113	39,308
UT	2011-2015	21	99	13,445	47,295	29,255	29,315	14,069	80,030	42,326	38,075
All states	**2011-2015**	**49**	**239**	**13,445**	**70,790**	**33,305**	**33,800**	**14,069**	**80,030**	**41,030**	**39,169**

Interchange geometric characteristics were collected using Google Earth® and Google Street View® (Table 91). The key variables of interest for modeling were:

- Terminal length (measured along crossroad)
  - Min = 605 ft, Max = 1236 ft, Mean = 829 ft
- Number of through lanes on crossroad approaches
  - Min = 1, Max = 4, Mean = 2.53
- Number of left-turn lanes
  - Exit (from freeway) and entrance (to freeway) movements: Min = 1, Max = 3, Mean = 1.94
- Number of right-turn lanes
  - Entrance (to freeway) movements: Min = 1, Max = 2, Mean = 1.05
  - Exit (from freeway) movements: Min = 1, Max = 2, Mean = 1.43
  - All movements: Min = 1, Max = 2, Mean = 1.24
- Traffic control type for right turns
  - To entrance ramp:
    - Both signalized (frontage roads): 7 sites
    - Both no control: 42 sites
  - From exit ramp
    - Both signalized: 19 sites
    - Both yield control: 19 sites
    - Both no control (free right): 2 sites
    - 1 signalized, 1 stop control: 3 sites
    - 1 signalized, 1 yield control: 1 site
    - 1 stop control, 1 yield control: 1 site
    - 1 signalized, 1 no control: 1 site
    - 1 stop control, 1 no control: 1 site
    - 1 yield control, 1 no control: 1 site

The findings with respect to some of these site characteristics are discussed in Section 8.3 on SPF development.

8.2.2 Crash Counts

All 49 interchanges included in the study experienced crashes. The average number of single- and MV crashes per terminal was 124.6 crashes (approximately 25.0 crashes per terminal per year), and the average number of vehicle-pedestrian plus vehicle-bicycle crashes per intersection was 2.1 over the entire study period (approximately 0.4 pedestrian and bicycle crashes per terminal per year). Table 93 shows all, SV, and MV crash counts by crash severity and time of day for each state over the entire study period. Crash counts are tallied by collision type and manner of collision across all states in Table 94.

Table 93. All crashes combined, single- and MV, and pedestrian and bicycle crash counts by crash severity—single-point diamond interchange crossroad ramp terminals

State	Date Range	Number of Sites	Number of Site-Years	Time of Day	All Crashes Combined			SV Crashes			Multiple-Vehicle Crashes			Pedestrian Crashes	Bicycle Crashes
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO	FI	FI
AZ	2011-2015	28	140	All	4071	1079	2992	287	83	204	3723	941	2782	18	43
UT	2011-2015	21	99	All	2133	504	1629	53	15	38	2040	454	1586	16	24
All states	**2011-2015**	**49**	**239**	**All**	**6,204**	**1,583**	**4,621**	**340**	**98**	**242**	**5,763**	**1,395**	**4,368**	**34**	**67**

Table 94. Crash counts by collision type and manner of collision and crash severity at single-point diamond interchange crossroad ramp terminals

Collision Type	Total	FI	PDO
Single-Vehicle Crashes			
Collision with animal	0	0	0
Collision with fixed object	288	74	214
Collision with other object	8	3	5
Collision with parked vehicle	0	0	0
Other SV collision	44	21	23
Total SV crashes	**340**	**98**	**242**
Multiple-Vehicle Crashes			
Head-on collision	83	54	29
Angle collision	573	205	368
Rear-end collision	4485	1056	3429
Sideswipe collision	579	63	516
Other MV collision	43	17	26
Total MV crashes	**5763**	**1395**	**4368**
Nonmotorized Crashes			
Pedestrian	34	34	0
Bicycle	67	67	0
Total nonmotorized crashes	**101**	**101**	**0**
Total Crashes	**6204**	**1594**	**4610**

8.3 Safety Performance Functions—Model Development

SPFs for the crossroad ramp terminal of a single-point diamond interchange were initially developed using Equation 56:

$$N_{spf\ int} = exp\left[a + b \times \ln(AADT_{xrd}) + c \times \ln\left(AADT_{ramp}\right) + d \times exit\_free\_right\right] \quad \text{(Eq. 56)}$$

Where:

$N_{spf\ int}$ = predicted average crash frequency of a crossroad ramp terminal at a single-point diamond interchange with base conditions (crashes/year)

$AADT_{xrd}$ = AADT on the crossroad (veh/day)

$AADT_{ramp}$ = um of ramp AADTs (veh/day)

exit_free_right = number of exit ramps with free-flow right turns (0, 1, or 2)

a, *b*, *c*, and *d* = estimated regression coefficients

All SPFs were developed using a NB regression model based on all sites combined. Based on a review of the number of states, sites, site-years, and crashes for the database assembled, data for all sites were used for model development to maximize the sample size rather than using a portion of the data for model development and a portion for model validation. Separate models using data from Arizona and Utah were initially explored and showed relatively consistent model coefficients. This increased confidence in the approach to pool all data for model development. STATA 12.1 was used for modeling. The final SPFs based on Equation 57 for crossroad ramp terminals at single-point diamond interchanges are shown in Table 95. Table 95 shows the estimated model coefficients and overdispersion parameter (estimate), their standard error, and associated p-values (or significance level) for each severity level. Figures 71-73 graphically present the SPFs shown in Table 95 for various crossroad and ramp AADTs.

SPFs for vehicle-pedestrian and vehicle-bicycle crashes at crossroad ramp terminals of single-point diamond interchanges could not be developed as pedestrian and bicycle volumes were not available. The SPFs in Table 95 predict the average crash frequency at the crossroad ramp terminal for all crash types (i.e., multi-vehicle, SV, pedestrian, and bicyclist) for total, FI, and PDO severity levels.

The estimated SPFs use both the crossroad AADT and sum of AADTs on all ramps connected to the interchange. The coefficients for these terms are positive and statistically significant (at greater than 99% confidence level) in each SPF, although their magnitudes fluctuate between the FI and PDO models. The estimated coefficient for crossroad AADT was lower for PDO crashes than for FI crashes. The estimated coefficient for ramp AADT was higher for PDO crashes than for FI crashes and greater than unity. This is associated with the larger number of rear-end PDO crashes occurring on the ramps at the study sites with larger ramp volumes.

Multiple models were tested considering the effects of different geometric attributes, including the interchange length, number of turn lanes (right and left), number of through lanes, and number of approaches with a particular right turn control type. Only the right turn control type was found to have a statistically significant effect. However, the type of right turn control also coincides with a particular state (i.e., Arizona uses more yield control, Utah uses more signal control and free-flow right turns), limiting the ability to estimate the effect of the right turn control variable without confounding effects. The free-right turn on exit ramps variable was included in the model because it was statistically significant, and its coefficient was relatively consistent between models. This variable is capturing not only the differences in right turn capacity and its effect on rear-end exit ramp crashes, but also the removal of conflict points within the defined ramp terminal area. The free-flow right turns at the study locations are accommodated by an auxiliary lane along the crossroad (thereby removing the need for right-turning vehicles to merge within the terminal area). Rather than presenting the free-flow right-turn effects as CMFs, separate SPFs were developed in the form of Equation 57 based on number of exit ramps with free-flow right turns to the crossroad – 0, 1, or 2. The final adjusted values for the estimated parameters are presented in Table 96. There are no additional base conditions for the SPFs.

$$N_{spf} = exp\left[a + b \times \ln(AADT_{xrd}) + c \times \ln\left(AADT_{ramp}\right)\right] \quad \text{(Eq. 57)}$$

Table 95. SPF coefficients for crossroad ramp terminals at single-point diamond interchanges (based on Equation 56)

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
Total Crashes	Intercept	-15.31	1.70	--	--
	$\ln(AADT_{xrd})$	0.69	0.17	0.000	Significant at 99% level
	$\ln(AADT_{ramp})$	1.08	0.18	0.000	Significant at 99% level
	exit_free_right	-0.60	0.11	0.000	Significant at 99% level
	Overdispersion	0.10	0.02	--	--
FI Crashes	Intercept	-16.71	2.06	--	--
	$\ln(AADT_{xrd})$	0.88	0.20	0.000	Significant at 99% level
	$\ln(AADT_{ramp})$	0.88	0.21	0.000	Significant at 99% level
	exit_free_right	-0.58	0.13	0.000	Significant at 99% level
	Overdispersion	0.11	0.03	--	--
PDO Crashes	Intercept	-15.60	1.72	--	--
	$\ln(AADT_{xrd})$	0.61	0.17	0.000	Significant at 99% level
	$\ln(AADT_{ramp})$	1.15	0.18	0.000	Significant at 99% level
	exit_free_right	-0.60	0.11	0.000	Significant at 99% level
	Overdispersion	0.10	0.02	--	--

Base Conditions: 0, 1, and 2 are valid values for the number of exit ramps with free-flow right turns to the crossroad. There are no additional base conditions.

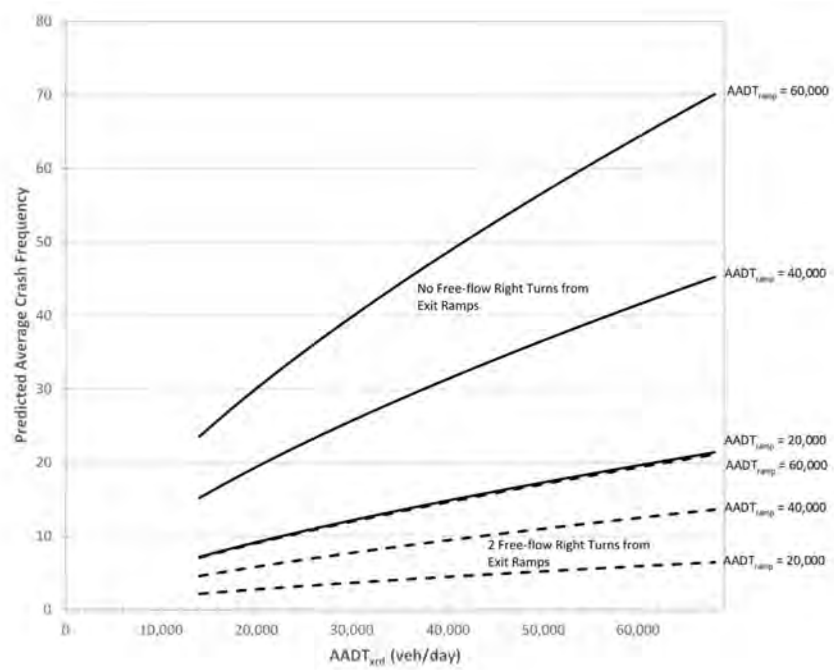


Figure 71. Graphical representation of the SPF for total crashes at crossroad ramp terminals at single-point diamond interchanges

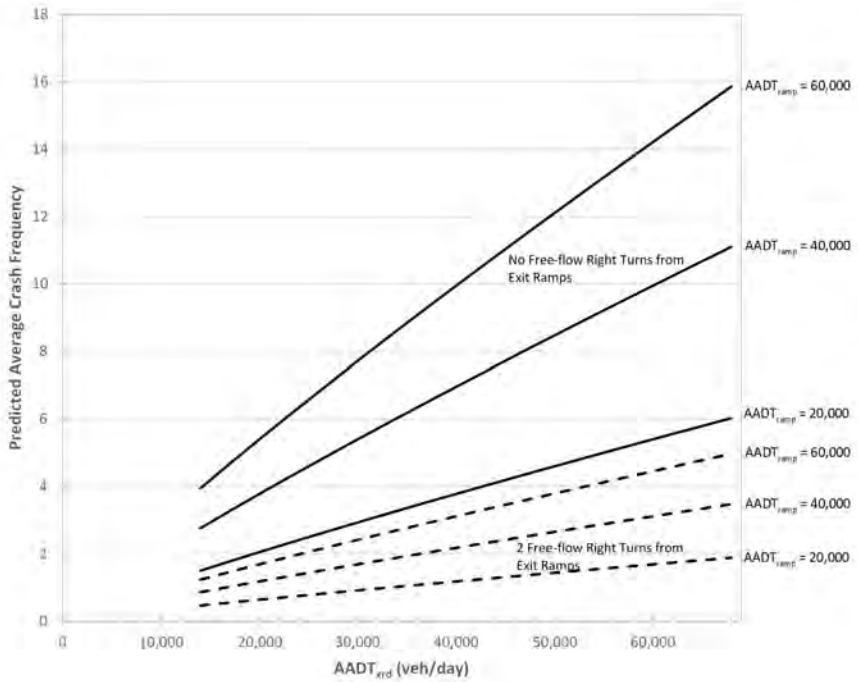


Figure 72. Graphical representation of the SPF for FI crashes at crossroad ramp terminals at single-point diamond interchanges

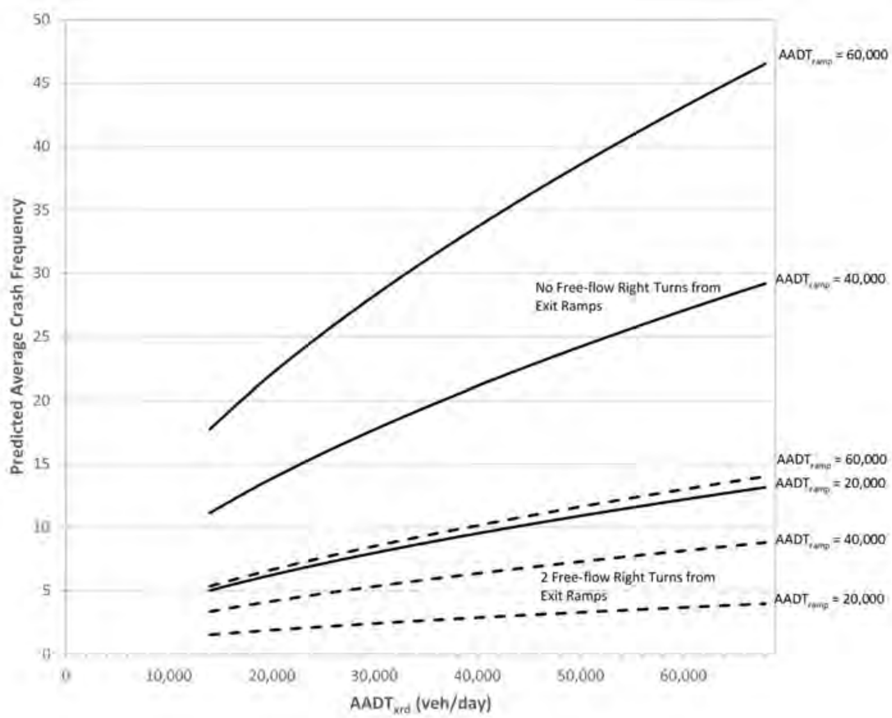


Figure 73. Graphical representation of the SPF for PDO crashes at crossroad ramp terminals at single-point diamond interchanges

Table 96. SPF coefficients for crossroad ramp terminals at single-point diamond interchanges (based on Equation 57)

Crash Severity	Number of Free-Flow Right Turns from Exit Ramp to Crossroad	SPF Coefficient			Dispersion Parameter
		a	*b*	*c*	
Total crashes	0	-15.31	0.69	1.08	0.10
	1	-15.91	0.69	1.08	0.10
	2	-16.51	0.69	1.08	0.10
Fatal-and- injury crashes	0	-16.71	0.88	0.88	0.11
	1	-17.29	0.88	0.88	0.11
	2	-17.87	0.88	0.88	0.11
Property- damage- only crashes	0	-15.60	0.61	1.15	0.10
	1	-16.20	0.61	1.15	0.10
	2	-16.80	0.61	1.15	0.10

There are no additional base conditions.

Tables 97 and 98 provide proportions for crash severity levels and collision types and manner of collision, respectively, for crashes at crossroad ramp terminals of single-point diamond interchanges. These proportions were calculated based on the observed data from both states combined.

Table 97. Distributions for crash severity level at crossroad ramp terminals at single-point diamond interchanges

Crash Severity Level	Percentage of Total Crashes	Percentage of FI Crashes
Fatal	0.16	0.6
Incapacitating injury	1.19	4.7
Non-incapacitating injury	7.09	27.8
Possible injury	17.07	66.9
Total fatal plus injury	25.52	
Property-damage-only	74.48	
Total	**100.0**	**100.0**

Table 98. Distributions for collision type and manner of collision at crossroad ramp terminals at single-point diamond interchanges

Collision Type	Percentage of Total Crashes	
	FI	PDO
Single-Vehicle Crashes		
Collision with animal	0.0	0.0
Collision with fixed object	4.6	4.6
Collision with other object	0.2	0.1
Collision with parked vehicle	0.0	0.0
Other SV collision	1.3	0.5
Multiple-Vehicle Crashes		
Head-on collision	3.4	0.6
Angle collision	12.9	8.0
Rear-end collision	66.2	74.4
Sideswipe collision	4.0	11.2
Other MV collision	1.1	0.6
Nonmotorized Crashes		
Pedestrian	2.1	0.0
Bicycle	4.2	0.0
Total Crashes	**100.0**	**100.0**

Following the development of the crash prediction models for crossroad ramp terminals at single-point diamond interchanges, compatibility testing of the new models to confirm that the new models provide reasonable results over a broad range of input conditions and that the new models integrate seamlessly with existing intersection crash prediction models in the first edition of the HSM was conducted. The graphical representations of the crash prediction models in Figures 71-73 provide some sense of the reasonableness of the new models for crossroad ramp terminals at single-point diamond interchanges. Nothing from these figures suggests that the models provide unreasonable results. Comparison of the crash prediction models for crossroad ramp terminals at single-point diamond interchanges to the crash prediction models for tight diamond interchanges is presented in Section 9.3.

Regarding seamlessly integrating the new crash prediction models for crossroad ramp terminals at single-point diamond interchanges with existing crash prediction models in Chapter 19 of the HSM, the primary issue that needs to be clearly addressed is the approach for defining crashes associated with the crossroad ramp terminals at single-point diamond interchanges. As stated in Section 8.1, crashes associated with the crossroad ramp terminals at single-point diamond interchanges are defined as follows:

- Crashes occurring on the crossroad within the ramp terminal boundary, defined as a point 100 ft from the gore or curb return of the outermost ramp connection, and having one of the following attributes:
  - at intersection
  - intersection-related
  - at driveway
  - driveway-related
  - involving a pedestrian or bicyclist
- Crashes occurring on a ramp with at least one of the following attributes:
  - at intersection
  - intersection-related
  - involving a pedestrian or bicyclist
  - located on an exit ramp and manner of collision is rear-end

This definition departs from the NCHRP Project 17-45 and HSM approach for defining ramp terminal crashes, using a different distance reference to define the crossroad ramp terminal boundary, so this needs to be clearly stated in the second edition of the HSM. No other issues were identified concerning integrating the new crash prediction models for crossroad ramp terminals at single-point diamond interchanges with existing crash prediction models in Chapter 19 of the HSM.

8.4 Crash Modification Factors

During the development of the crash prediction models for crossroad ramp terminals at single-point diamond interchanges, three potential sources of CMFs for use with the SPFs were considered:

- CMFs developed as part of this research based on a cross-sectional study design and regression modeling
- CMFs already incorporated into the first edition of the HSM and applicable to crossroad ramp terminals at single-point diamond interchanges
- High-quality CMFs applicable to crossroad ramp terminals at single-point diamond interchanges developed using defensible study designs (e.g., observational before-after evaluation studies using SPFs—the EB method), as referenced in FHWA's CMF Clearinghouse with four or five-star quality ratings or based on a review of relevant intersection safety literature

Based on a review of the CMFs already incorporated in the first edition of the HSM and other potential high-quality CMFs developed using defensible study designs, no CMFs were identified that were adaptable to the predictive models for crossroad ramp terminals at single-point diamond interchanges. New potential CMFs were explored during regression modeling, but only the right turn configuration from the exit ramps to the crossroad (i.e., free-flow versus yield/stop/signal control) showed consistent and statistically significant safety effects. As noted in Section 8.3, instead of presenting the free-flow right-turn effects as CMFs, separate SPFs are recommended for three exit ramp to crossroad right turn configurations (defined by the number of exit ramps with free-flow right turns to the crossroad—0, 1, or 2). No additional base conditions or CMFs are recommended for use with the SPFs for crossroad ramp terminals as single-point diamond interchanges. The lack of other effects is not necessarily surprising. Crossroad ramp terminals at single-point diamond interchanges are relatively similar in some of their major features (e.g., one intersection through which all at-grade traffic movements are made, signal timing, presence of exclusive left-turn lanes). Sample sizes did not allow any differences in safety performance to be detected at finer levels of detail (e.g., number of exclusive left-turn lanes, number of cross street through lanes).

8.5 Severity Distribution Functions

Development of SDFs was explored for crossroad ramp terminals at single-point diamond interchanges using methods outlined in Section 2.2.3 of this report. SDFs were not used in the development of crash prediction methods in the first edition of the HSM but were subsequently used in the Supplement to the HSM for freeways and ramps (AASHTO, 2014). The database used to explore SDFs for crossroad ramp terminals at single-point diamond interchanges consisted of the same crashes and crossroad ramp terminals as the database used to estimate the SPFs, but restructured so that the basic observation unit (i.e., database row) was a crash instead of a ramp terminal. No traffic or geometric variables showed consistent and statistically significant effects in the SDFs for crossroad ramp terminals at single-point diamond interchanges.

8.6 Summary of Recommended Models for Incorporation in the HSM

In summary, several crash prediction models were developed for crossroad ramp terminals at single-point diamond interchanges for consideration in the second edition of the HSM. The final models for FI and PDO severity levels presented in Table 96 are recommended for inclusion in the second edition of the HSM, consistent with existing methods in HSM Chapter 19. Separate SPFs are presented in the form of Equation 57 based on number of exit ramps with free-flow right turns to the crossroad (– 0, 1, or 2).

Attempts to develop SDFs for crossroad ramp terminals at single-point diamond interchanges proved unsuccessful for the reasons explained in Section 4.6. The SPFs by severity for crossroad ramp terminals at single-point diamond interchanges provided in Table 96, combined with the severity distributions provided in Table 97, are recommended for addressing crash severity at these intersection types, without use of SDFs. The SPFs predict FI and PDO crashes separately. Additional disaggregation of FI crashes into fatal, incapacitating injury, non-incapacitating injury, and possible injury crashes can be accomplished using the severity distributions provided in Table 97.

Appendix A presents recommended text for incorporating the final recommended models for crossroad ramp terminals at single-point diamond interchanges into Chapter 19 of the HSM.

Chapter 9. Development of Models for Use in HSM Crash Prediction Methods: Crossroad Ramp Terminals at Tight Diamond Interchanges

This section describes the development of crash prediction models for crossroad ramp terminals at tight diamond interchanges (TDs). Tight diamond interchanges are implemented in urban areas. Their crossroad ramp terminals are characterized by two at-grade intersections spaced between 200 and 400 ft apart, through which all at-grade traffic movements are made (Leisch, 2005, Hughes et al., 2010, Sellinger and Sharp, 2000). An additional characteristic of the tight diamond interchange is exclusive left-turn lanes, for movements from the crossroad to the freeway, in advance of the upstream ramp terminal (Leisch, 2005). The two intersections of the tight diamond interchange are signalized to operate as one (Leisch, 2005). Section 9.1 describes the site selection and data collection processes for developing crash prediction models for crossroad ramp terminals at tight diamond interchanges. Section 9.2 provides descriptive statistics of the databases used for model development. Section 9.3 presents the statistical analysis and resulting SPFs for crossroad ramp terminals at tight diamond interchanges. Section 9.4 discusses CMF development for use with the SPFs. Section 9.5 addresses the outcomes of the analysis to develop SDFs for crossroad ramp terminals of tight diamond interchanges. Section 9.6 provides recommendations for incorporating the new crash prediction models for crossroad ramp terminals at tight diamond interchanges in the second edition of the HSM.

9.1 Site Selection and Data Collection

A list of potential tight diamond interchanges was developed by searching databases and satellite imagery in six states:

- Arizona (AZ)
- California (CA)
- Florida (FL)
- Minnesota (MN)
- Ohio (OH)
- Utah (UT)

Data collection activities for these sites included gathering geometric design attributes of the interchanges as well as traffic and crash data. Geometric attributes were collected from aerial imagery in Google Earth®, as well as Google Street View®. Table 99 lists the geometric attributes collected (and respective definitions and permitted values) for each tight diamond interchange.

Table 99. Site characteristic variables collected for crossroad ramp terminals at tight diamond interchanges

Variable	Definition	Range of Permitted Values
General Intersection Attributes		
Intersection configuration (i.e., number of legs and type of traffic control)	Indicates the number of legs and type of traffic control	4SG
Area type	Indicates whether the intersection is in a rural or urban area	Urban
Presence of intersection lighting	Indicates if overhead lighting is present at the intersection proper	Yes, no
Crossroad over or under freeway	Indicates whether the crossroad passes over or under the freeway	Over or under
Construction year	Estimated year when the interchange was constructed	Range: 2005 to 2018
Approach Specific Attributes		
Route name or number	Specifies the route name or number of the approach	
Location at intersection	Side of the intersection the approach is located	Primary, secondary
Presence of left-turn lanes	The number of approaches with one or more left-turn lanes	2,3,4
Left-turn protected only	Number of approaches with protected only left-turn operations	0,1,2
Number of left-turn lanes	Number of left-turn lanes provided for turning movements to/from each freeway ramp	0, 1, 2
Presence of right-turn lane	Number of approaches with one or more right-turn lanes	0,1,2
Number of right-turn lanes	Number of right-turn lanes provided for turning movements to/from each freeway ramp	0, 1, 2
Number of through lanes	Number of through lanes present on each crossroad approach to each crossroad ramp terminal	1, 2
Presence of frontage roads	Indicates the presence of frontage roads at the interchange, where a through movement is added between the exit and entrance ramps	Yes, no
Presence of crosswalk	Indicates the presence of crosswalks at the crossroad ramp terminal	Yes, no
Presence of bike lane	Indicates the presence of a bike lane on the crossroad at each crossroad ramp terminal	Yes, no
Median width	Width of median (in feet) on each crossroad approach to each crossroad ramp terminal	Range: 0 to 32
Median type	Type of median present on each crossroad approach to the crossroad ramp terminal	Raised, flush, none
Number of driveways	Number of driveways located within 250 ft of the crossroad stop bars/lines	Range: 0 to 2
Number of intersections	Number of intersections with public streets located within 250 ft of the stop bars/lines	Range: 0 to 4
Presence of railroad crossing	Indicates the presence of a railroad crossing on the crossroad	Yes, no
Traffic control type for right turns	Type of traffic control for right-turn movements	Signal, stop, yield, none
Number of channelized right turns	Number of right-turn movements from the crossroad to ramps and from the ramps to the crossroad with raised or painted island	0, 1, 2, 3, 4
U-turns allowed	Indicates if a U-turn is allowed between exit ramps and entrance ramps	Yes, no

The "construction year" was estimated using the "Clock" feature in Google Earth® as the earliest year with the interchange present in aerial imagery. Some tight diamond interchanges in the database were built during the study period and, therefore, had fewer years of data available for analysis. Additional information about the interchange configuration was used to exclude sites with uncommon or inconsistent geometric conditions, such as the lack of a left-turn lane on a crossroad approach. Speed limits on freeways and crossroads were not collected because they were not statistically significant in single-point interchange models.

Traffic data collection activities primarily involved accessing publicly available traffic volumes and statistics.

Crash data were obtained from state DOTs. The crash data generally included details about the crash location (geographic coordinates), as well as attributes describing the crash, people involved in the crash, and the road and environmental conditions at the location and time of the crash.

Identifying crashes associated with the ramp terminal required a clear definition of a ramp terminal-related crash based on geographic location and crash attributes. To maintain a level of consistency with the ramp terminal models in NCHRP Project 17-45 and in the single-point diamond chapter of this report, these crashes were selected using the following criteria:

- Crashes occurring on the crossroad within the ramp terminal boundary, defined as a point 100 ft from the gore or curb return of the outermost ramp connection, and having one of the following attributes:
    - at intersection
    - intersection-related
    - at driveway
    - driveway-related
    - involving a pedestrian or bicyclist
- Crashes occurring on a ramp with at least one of the following attributes:
    - at intersection
    - intersection-related
    - involving a pedestrian or bicyclist
    - located on an exit ramp and manner of collision is rear-end

This definition departs from the NCHRP 17-45 ramp terminal definition, using a different distance reference to define the crossroad ramp terminal boundary. The NCHRP 17-45 definition used 250 ft from the crossroad ramp terminal, measured from the center of the intersection. The definition implemented for the crossroad ramp terminals of tight diamond interchanges (as well as single-point diamond interchanges) is based on the ANSI D16.1-2007 (Manual on Classification of Motor Vehicle Traffic Accidents) definition of an interchange crash. According to the ANSI definition, an interchange crash is a crash in which the first harmful event occurs within a boundary defined by a point 100 ft from the gore or curb return of the outermost ramp connection.

Figure 74 shows an example of a tight diamond interchange with the boundaries for identifying interchange-related crashes. Crossroad crashes were identified using the yellow boundary, while ramp crashes were identified using the white boundaries.

Figure 74. Example of a tight diamond interchange with the ramp boundaries definition

All of the collected data (i.e., site characteristics, crashes, and traffic volumes) were assembled into one database for the purpose of model development. After initial database development and quality assessments, interchanges in Arizona and Utah were selected for model development due to a higher level of confidence in accurately and reliably locating and identifying terminal-related crashes in those states. This decision resulted in 57 potential crossroad ramp terminals for model development. This list of interchanges was further reduced due to lack of available or compatible data. Specifically, one interchange was removed because it was not striped as a tight diamond interchange until 2015, one was removed because crossroad traffic volumes were not available, and three were removed because ramp traffic volumes were either questionable or nonexistent.

With 52 potential sites remaining for model development, CURE plots for preliminary models indicated one potential outlier was present in the database. This location had an exceptionally low number of crashes compared to the traffic volume in the interchange. The final database excluded this site, resulting in 51 crossroad ramp terminals at tight diamond interchanges for model development.

9.2 Descriptive Statistics of Database

A total of 51 crossroad ramp terminals at tight diamond interchanges were used for crash prediction model development. The selected sites were from two states: Arizona and Utah.

9.2.1 Traffic Volumes and Site Characteristics

Traffic volumes and crash data from years 2011 through 2015 were used for analysis. Table 100 provides summary statistics for traffic volumes at the study sites used for model development. Study period (date range), number of sites and site-years, and traffic volume descriptive statistics are shown by state.

Table 100. Crossroad and ramp AADT statistics at tight diamond interchange crossroad ramp terminals

State	Date Range	Number of Sites	Number of Site-Years	Crossroad AADT				Ramp AADT (sum of all four ramps)			
				Min	Max	Mean	Median	Min	Max	Mean	Median
AZ	2011-2015	45	225	8,921	51,438	27,467	26,357	9,955	74,656	38,827	38,386
UT	2011-2015	6	28	14,600	34,200	24,322	25,200	20,416	72,179	35,842	31,523
All states	**2011-2015**	**51**	**253**	**8,921**	**51,438**	**27,097**	**26,357**	**9,955**	**74,656**	**38,476**	37,049

Interchange geometric characteristics were collected using Google Earth® and Google Street View® (Table 99). The key variables of interest for modeling were:

- Distance between terminals
  - Min = 174.5 ft, Max = 387 ft, Mean = 307.9 ft
- Number of through lanes on crossroad approaches
  - Min = 0, Max = 4, Mean = 2.20
- Number of left-turn lanes
  - Exit (from freeway) and entrance (to freeway) movements: Min = 0, Max = 2, Mean = 1.30
- Number of right-turn lanes
  - Entrance (to freeway) movements: Min = 0, Max = 2, Mean = 0.82
  - Exit (from freeway) movements: Min = 0, Max = 2, Mean = 1.05
  - All movements: Min = 0, Max = 2, Mean = 0.94
- Traffic control type for right turns
  - To entrance ramp:
    - Both signalized: 47 sites
    - Both yield control: 1 site
    - 1 signalized, 1 no control (free right): 2 sites
    - 1 yield, 1 no control: 1 site
  - From exit ramp
    - Both signalized: 44 sites
    - Both yield control: 3 site
    - 1 signalized, 1 no control: 3 sites
    - 1 yield control, 1 no control: 1 site

The findings with respect to some of these site characteristics are discussed in Section 9.3 on SPF development.

9.2.2 Crash Counts

All 51 interchanges included in the study experienced crashes. The average number of SV and MV crashes per terminal was 92.8 crashes (approximately 18.7 crashes per terminal per year), and the average number of vehicle-pedestrian plus vehicle-bicycle crashes per intersection was 1.4 over the entire study period (approximately 0.3 pedestrian and bicycle crashes per terminal per year). Table 101 shows all, SV, and MV crash counts by crash severity and time of day for each state over the entire study period. Crash counts are tallied by collision type and manner of collision across all states in Table 102.

Table 101. All crashes combined, single- and MV, and pedestrian and bicycle crash counts by crash severity—tight diamond interchange crossroad ramp terminals

State	Date Range	Number of Sites	Number of Site-Years	Time of Day	All Crashes			SV Crashes			Multiple-Vehicle Crashes			Pedestrian Crashes			Bicycle Crashes		
					Total	FI	PDO	Total	FI	PDO	Total	FI	PDO	Total	FI	PDO	Total	FI	PDO
AZ	2011-2015	45	225	All	4185	1215	2970	144	30	114	3974	1125	2849	23	22	1	44	38	6
UT	2011-2015	6	28	All	621	199	422	14	3	11	601	190	411	2	2	0	4	4	0
All states	**2011-2015**	**51**	**253**	**All**	**4806**	**1414**	**3392**	**158**	**33**	**125**	**4575**	**1315**	**3260**	**25**	**24**	**1**	**48**	**42**	**6**

Table 102. Crash counts by collision type and manner of collision and crash severity at tight diamond interchange crossroad ramp terminals

Collision Type	Total	FI	PDO
Single-Vehicle Crashes			
Collision with animal	1	0	1
Collision with fixed object	128	19	109
Collision with other object	2	0	2
Collision with parked vehicle	0	0	0
Other SV collision	27	14	13
Total SV crashes	**158**	**33**	**125**
Multiple-Vehicle Crashes			
Head-on collision	31	14	17
Angle collision	1428	617	811
Rear-end collision	2592	628	1964
Sideswipe collision	461	39	422
Other MV collision	63	17	46
Total MV crashes	**4575**	**1315**	**3260**
Nonmotorized Crashes			
Pedestrian	25	24	1
Bicycle	48	42	6
Total nonmotorized crashes	**73**	**66**	**7**
Total Crashes	**4806**	**1414**	**3392**

9.3 Safety Performance Functions—Model Development

SPFs for the crossroad ramp terminal of a tight diamond interchange were developed using Equation 57:

$$N_{spf\ int} = exp\left[a + b \times \ln(AADT_{xrd}) + c \times \ln\left(AADT_{ramp}\right)\right] \qquad \text{(Eq. 57)}$$

Where:

$N_{spf\ int}$ = predicted average crash frequency of a crossroad ramp terminal at a tight diamond interchange with base condition (crashes/year);
$AADT_{xrd}$ = AADT on the crossroad (veh/day);
$AADT_{ramp}$ = sum of ramp AADTs (veh/day); and
a, b, c = estimated regression coefficients.

The SPFs were developed using NB regression. All data from AZ and UT were used in developing the SPFs to maximize the sample size. However, separate models for AZ and UT were first compared and showed consistency between the models, which increased confidence that combining the data was appropriate. STATA 14.2 was used for modeling. The final SPF models for tight diamond interchange crossroad ramp terminals are shown in Table 103, and separate SPFs are provided for different crash severity levels—total, FI, and PDO crashes. Table 103 displays the overdispersion parameter (estimate), standard error, and significance level (p-value) for the model variables for each severity level.

SPFs for vehicle-pedestrian and vehicle-bicycle collisions at crossroad ramp terminals of tight diamond interchanges could not be developed as pedestrian and bicycle volumes were not available. The SPFs predict the average crash frequency at the crossroad ramp terminal for all crash types (i.e., multi-vehicle, SV, pedestrian, and bicyclist) of different injury severities.

Table 103. SPF coefficients for tight diamond interchange cross ramp terminals

Crash Severity	Parameter	Estimate	Standard Error	Pr > F	Significance Level
Total Crashes	Intercept	-11.46	2.03	--	--
	$\ln(AADT_{xrd})$	0.68	0.26	0.008	Significant at 99% level
	$\ln(AADT_{ramp})$	0.71	0.22	0.002	Significant at 99% level
	Overdispersion	0.25	0.05	--	--
FI Crashes	Intercept	-11.90	2.09	--	--
	$\ln(AADT_{xrd})$	0.50	0.26	0.05	Significant at 95% level
	$\ln(AADT_{ramp})$	0.81	0.22	<0.001	Significant at 99% level
	Overdispersion	0.23	0.06	--	--
PDO Crashes	Intercept	-11.99	2.19	--	--
	$\ln(AADT_{xrd})$	0.77	0.28	0.006	Significant at 99% level
	$\ln(AADT_{ramp})$	0.63	0.24	0.009	Significant at 99% level
	Overdispersion	0.29	0.06	--	--

No base conditions

The estimated SPFs use both the crossroad AADT and sum of AADTs on all ramps connected to the interchange. The natural log of the years of data was included as an offset in all models. The coefficients for these terms are positive and statistically significant (at greater than or equal to 95% confidence level) in each SPF, although their magnitudes fluctuate between the FI and PDO models. The crossroad and ramp volume coefficients indicate that as the volumes increase, the predicted crash frequency increases.

Before finalizing the models in Table 103, multiple models were developed testing other variables, such as traffic control type for right turns, number of left-turn lanes, number of right-turn lanes, number of channelized right turns, distance between terminals, number of driveways, and number of intersections. However, none of the parameters associated with the tested variables were statistically significant in the models. The only statistically significant variables, which were included in the final models, were the crossroad and ramp AADT.

In addition, CURE plots were developed for the SPFs to determine and analyze the functional form of the models. Separate CURE plots were created for each SPF (i.e., total, FI, and PDO crashes) and for each independent variable (crossroad AADT and ramp AADT). The CURE plots indicated the model functional forms in Table 103 are fitting based on the fluctuations of the residuals around the zero cumulative residuals line and based on the cumulative residuals within the upper and lower bounds.

Figures 78-80 present graphical representations of the SPFs for the different crash severity levels and crossroad and ramp AADTs.

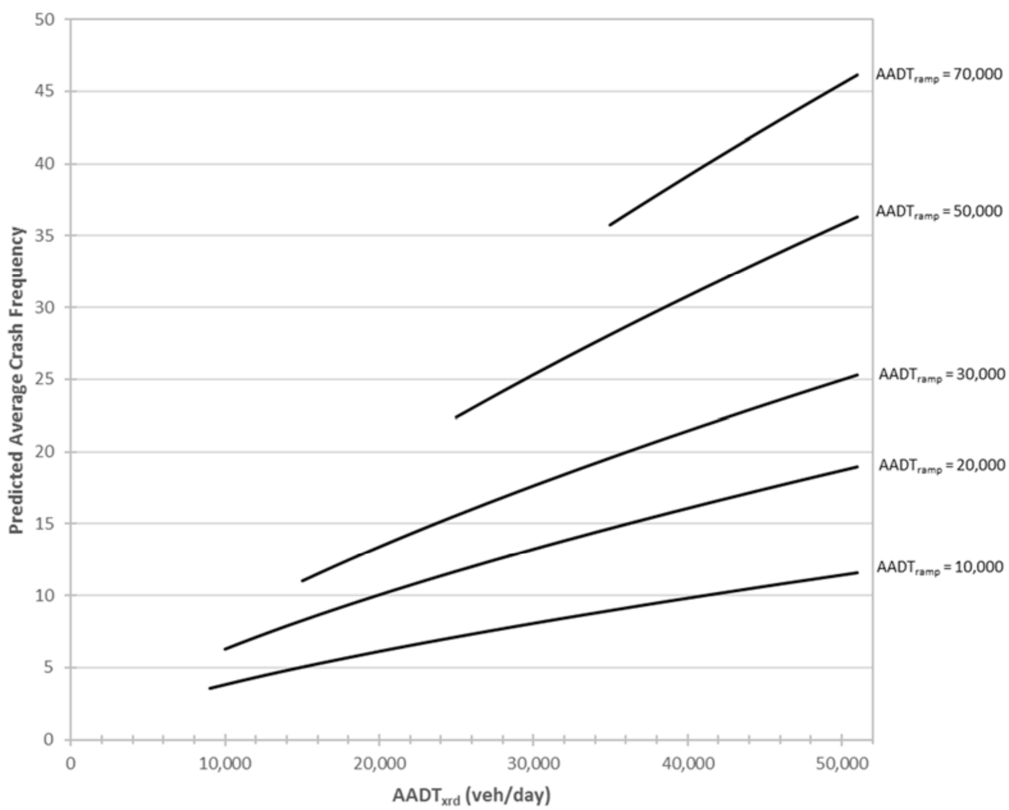


Figure 75. Graphical representation of the SPF for total crashes at crossroad ramp terminals at tight diamond interchanges

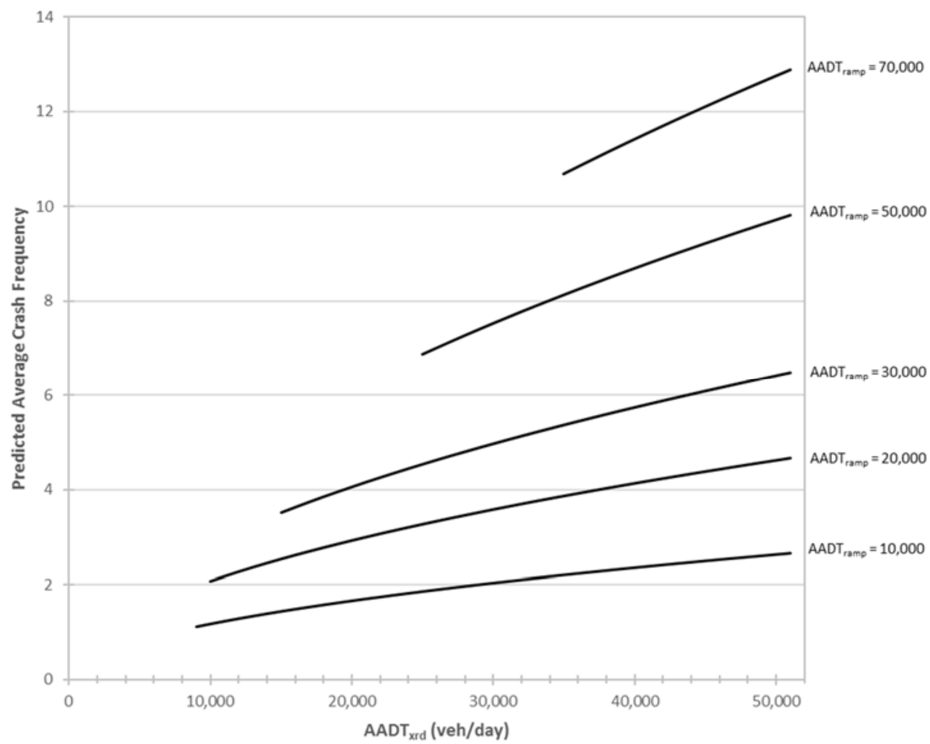


Figure 76. Graphical representation of the SPF for FI crashes at crossroad ramp terminals at tight diamond interchanges

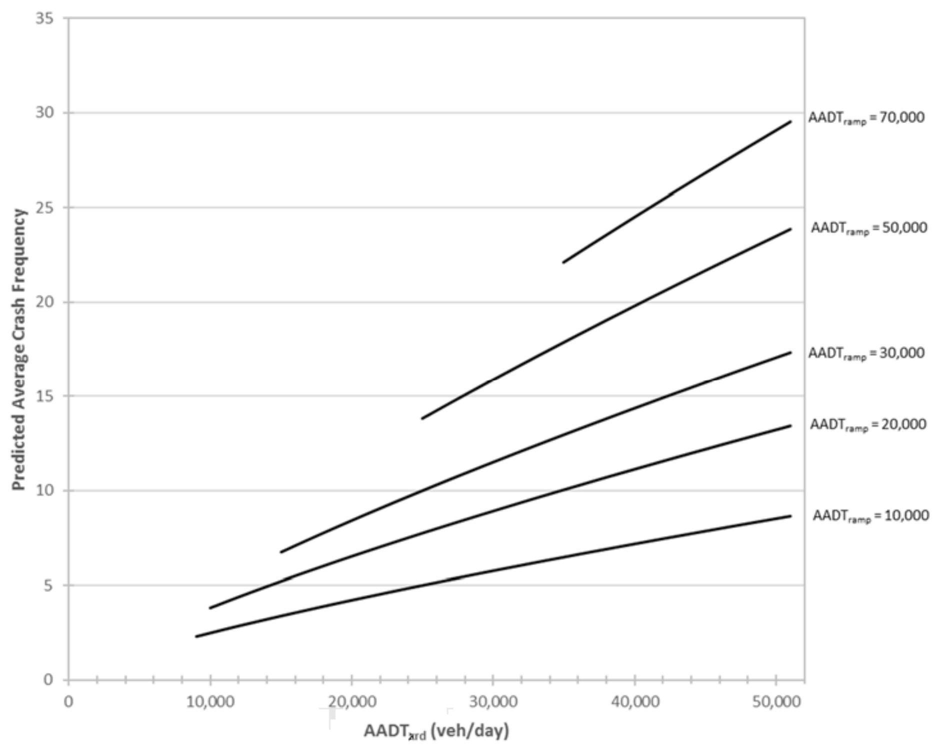


Figure 77. Graphical representation of the SPF for PDO crashes at crossroad ramp terminals at tight diamond interchanges

Following the development of the crash prediction models for crossroad ramp terminals at tight diamond interchanges, the research team conducted compatibility testing of the new models to confirm that the new models provide reasonable results over a broad range of input conditions and that the new models integrate seamlessly with existing intersection crash prediction models in the first edition of the HSM. The graphical representations of the crash prediction models in Figures 75-77 provide some sense of the reasonableness of the new models for crossroad ramp terminals at tight diamond interchanges. Nothing from these figures suggests that the models provide unreasonable results. Figures 78-80 compare the crash prediction models for crossroad ramp terminals at tight diamond interchanges to the crash prediction models from Section 8.3 for crossroad ramp terminals at single-point diamond interchanges when there are no free-flow right turns from the exit ramps to the crossroads. In general, the SPFs for single-point diamond ramp terminals predict more crashes than the SPFs for tight diamond ramp terminals in higher volume conditions, and the differences are primarily driven by the PDO models. In summary, the comparisons show that the two sets of models appear compatible and provide reasonable results over the range of applicable traffic volume conditions. The figures do not display the general ranges of prediction error and therefore readers should not put too much emphasis on the relative positions of predicted average crash frequencies when predictions are close.

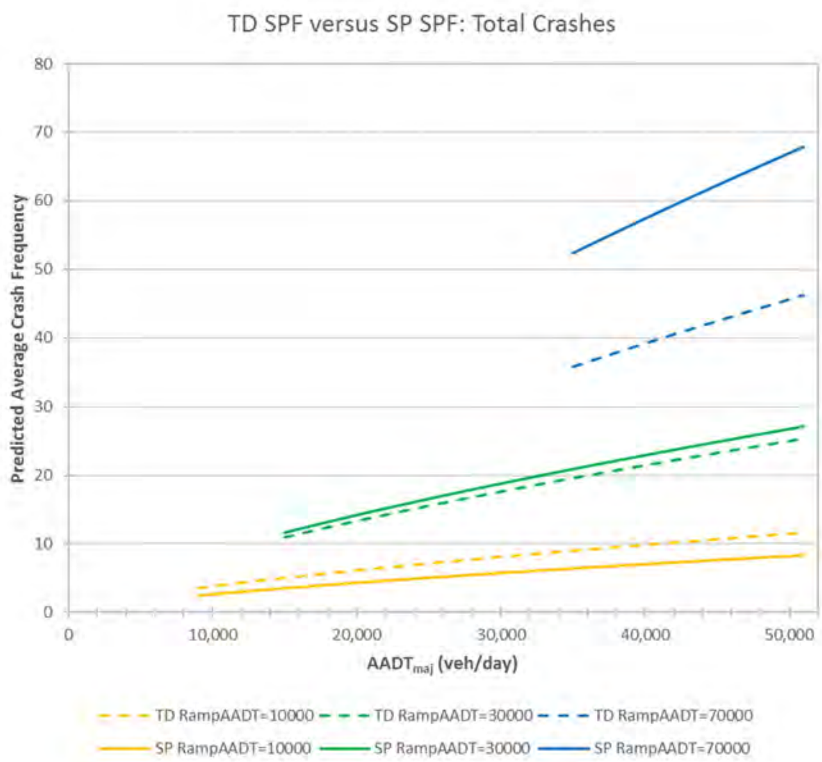


Figure 78. Comparison of crash prediction models for total crashes at crossroad ramp terminals at tight diamond interchanges and single-point diamond interchanges

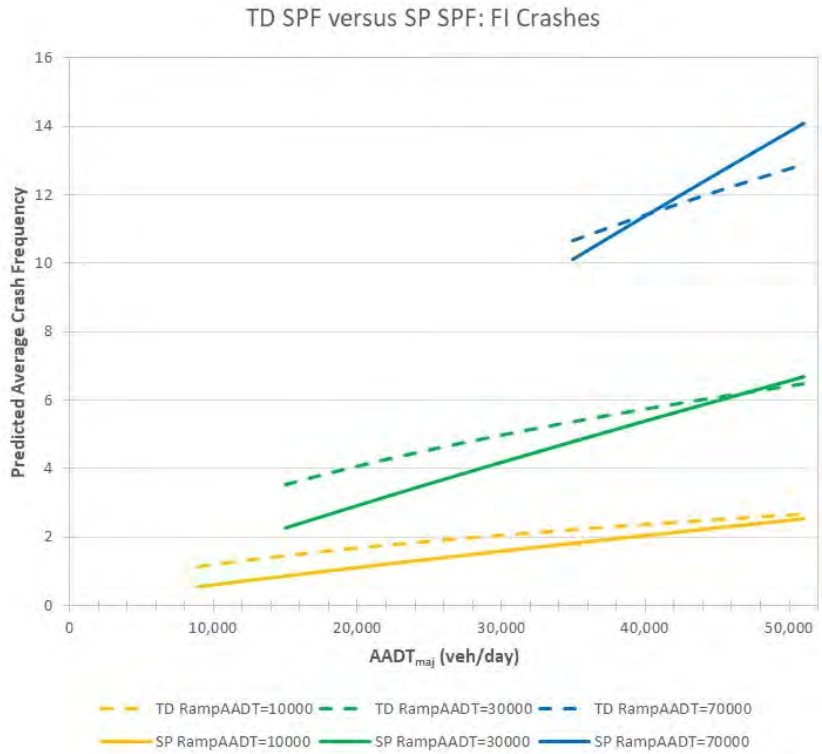


Figure 79. Comparison of crash prediction models for FI crashes at crossroad ramp terminals at tight diamond interchanges and single-point diamond interchanges

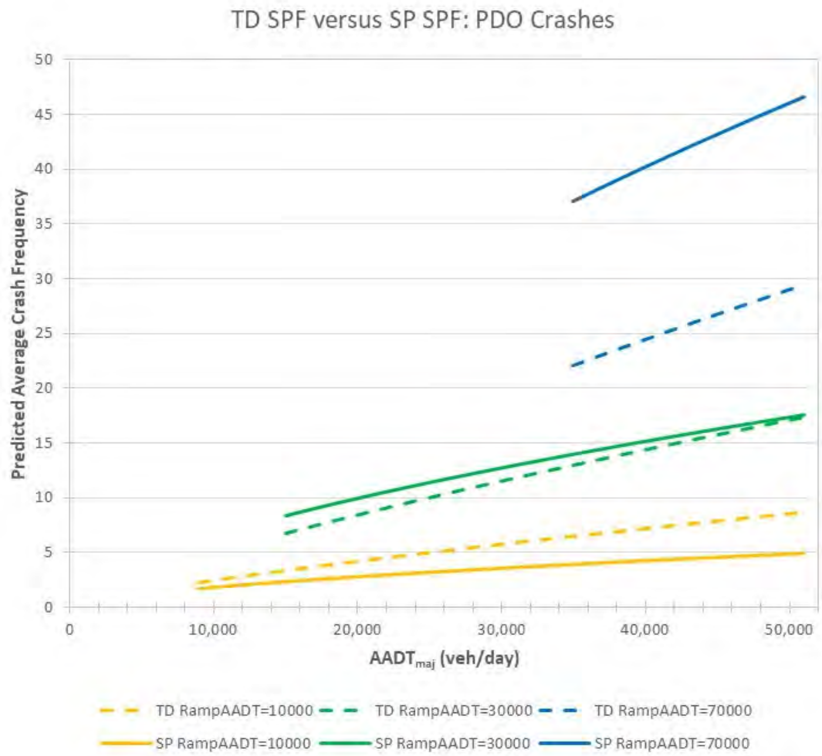


Figure 80. Comparison of crash prediction models for PDO crashes at crossroad ramp terminals at tight diamond interchanges and single-point diamond interchanges

Table 104 displays the distribution of crashes at tight diamond interchange crossroad ramp terminals by severity level. Table 105 displays the distribution of crashes at tight diamond interchange crossroad ramp terminals by collision type and manner of collision. The data from all states combined were used to calculate the proportions.

Table 104. Distributions for crash severity level at tight diamond interchange crossroad ramp terminals

Crash Severity Level	Percentage of Total Crashes	Percentage of FI Crashes
Fatal	0.08	0.28
Incapacitating injury	1.81	6.15
Non-incapacitating injury	9.68	32.89
Possible injury	17.85	60.68
Total fatal plus injury	29.42	
Property-damage-only	70.58	
Total	**100.0**	**100.0**

Table 105. Distributions for collision type and manner of collision at tight diamond interchange crossroad ramp terminals

Collision Type	Percentage of Total Crashes	
	FI	PDO
SV Crashes		
Collision with animal	0.0	0.0
Collision with fixed object	1.3	3.2
Collision with other object	0.0	0.1
Collision with parked vehicle	0.0	0.0
Other SV collision	1.0	0.4
Multiple-Vehicle Crashes		
Head-on collision	1.0	0.5
Angle collision	43.6	23.9
Rear-end collision	44.4	57.9
Sideswipe collision	2.8	12.4
Other MV collision	1.2	1.4
Nonmotorized Crashes		
Pedestrian	1.7	0.0
Bicycle	3.0	0.2
Total Crashes	**100.0**	**100.0**

9.4 Crash Modification Factors

There were no CMFs in the literature that were adaptable to the predictive models for tight diamond interchange crossroad ramp terminals. New potential CMFs were explored during this analysis using regression modeling; however, none showed statistically significant safety effects. As with the single-point diamond models, the lack of other effects is not necessarily surprising. Crossroad ramp terminals at tight diamond interchanges are relatively similar in some of their major features and operation (e.g., left-turn lanes for crossroad to freeway movements developed in advance of upstream terminal, signalized to operate as single intersection). Sample sizes, collinearity, and use of aggregate traffic volumes (i.e., AADT) did not allow any differences in safety performance to be detected at final levels of detail (e.g., number of lanes by movement).

9.5 Severity Distribution Functions

Development of SDFs was explored for tight diamond interchange crossroad ramp terminals using methods outlined in Section 2.2.3 of this report. The database used to explore SDFs consisted of the same crashes and crossroad ramp terminals as the database used to estimate the SPFs but restructured so that the basic observation unit (i.e., database row) was a crash instead of a ramp terminal. No traffic or geometric variables showed consistent, interpretable, and statistically significant effects in the SDFs for tight diamond interchange crossroad ramp terminals.

9.6 Summary of Recommended Models for Incorporation in the HSM

In summary, crash prediction models were developed for tight diamond interchange crossroad ramp terminals for consideration in the second edition of the HSM. The final models presented in Table 103 for FI and PDO crashes are recommended for inclusion in the second edition of the HSM.

Attempts to develop SDFs for tight diamond interchange crossroad ramp terminals proved unsuccessful. The SPFs by severity for tight diamond interchange crossroad ramp terminals provided in Table 103, combined with the severity distributions provided in Table 104, are recommended for addressing crash severity at these intersection types. The SPFs predict FI and PDO crashes separately. Additional disaggregation of FI crashes into fatal, incapacitating injury, non-incapacitating injury, and possible injury crashes can be accomplished using the severity distributions provided in Table 104.

Chapter 10.
Conclusions and Recommendations

The following conclusions and recommendations have been developed in this research:

1. The SPFs presented in this report have been developed consistent with existing methods in HSM Part C and comprehensive in their ability to address a wide range of intersection configurations and traffic control types in rural, urban, and suburban areas. The SPFs recommended for inclusion in the second edition of the HSM include:

   Intersections with All-Way Stop Control

   - Four-leg all-way stop-controlled intersections on rural two-lane highways
     - Total crashes
   - Three-leg all-way stop-controlled intersections on urban and suburban arterials
     - FI crashes
     - PDO crashes
   - Four-leg all-way stop-controlled intersections on urban and suburban arterials
     - FI crashes
     - PDO crashes

   Rural Three-Leg Intersections with Signal Control

   - Three-leg signalized intersections on rural two-lane highways
     - Total crashes
   - Three-leg signalized intersections on rural multilane highways
     - Total crashes
     - FI crashes

   Intersections on High-Speed Urban and Suburban Arterials

   - Three-leg stop-controlled intersections on high-speed urban and suburban arterials
     - MV total crashes
     - MV FI crashes
     - MV PDO crashes
     - SV total crashes
     - SV FI crashes
     - SV PDO crashes
   - Three-leg signalized intersections on high-speed urban and suburban arterials
     - MV total crashes
     - MV FI crashes
     - MV PDO crashes
     - SV total crashes
     - SV FI crashes
     - SV PDO crashes

- Four-leg stop-controlled intersections on high-speed urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV FI crashes
  - SV PDO crashes
- Four-leg signalized intersections on high-speed urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV FI crashes
  - SV PDO crashes

Five-Leg Intersections with Signal Control

- Five-leg signalized intersections on urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV FI crashes
  - SV PDO crashes

Three-Leg Intersections Where the Through Movement Makes a Turning Maneuver at the Intersection

- Three-leg turning intersections on rural two-lane highways
  - Total crashes
- Three-leg turning intersections on urban and suburban arterials
  - MV total crashes
  - MV FI crashes
  - MV PDO crashes
  - SV total crashes
  - SV PDO crashes

Crossroad Ramp Terminals at Single-Point Diamond Interchanges

- Crossroad ramp terminals at single-point diamond interchanges
  - FI crashes
  - PDO crashes

Crossroad Ramp Terminals at Tight Diamond Interchanges

- Crossroad ramp terminals at tight diamond interchanges
  - FI crashes
  - PDO crashes

Recommended draft text for inclusion in the second edition of the HSM is presented in Appendix A that incorporates the new crash prediction models for the intersection configurations and traffic control types developed as part of this research.

2. Development of SDFs for most of the new intersection configurations and traffic control types was explored for potential use in combination with the SPFs to estimate crash severity as a function of geometric design elements and traffic control features. Due to challenges and inconsistencies in developing and interpreting the SDFs, it is recommended for the second edition of the HSM that crash severity for the new intersection configurations and traffic control types be addressed in a manner consistent with existing methods in Chapters 10, 11, and 12 of the first edition of the HSM, without use of SDFs. Future research should continue to explore the most promising approaches for addressing crash severity in the HSM predictive methods.

3. Crash prediction models could be developed for additional intersection configurations and traffic control types that are not addressed in the first edition of the HSM and were not developed as part of this research. For example, several additional intersection configurations and traffic control types for which crash prediction models could be developed include:

- Three-leg intersections with a commercial driveway forming a fourth leg
- Intersections with indirect left turns from the minor road (e.g., U-turns or J-turns)
- Intersections with yield or no control
- Rural five-leg intersections
- Urban and suburban five-leg intersections with minor road stop control
- Six-or-more-leg intersections
- Diverging-diamond ramp terminals

4. Future research should be conducted to further evaluate approaches to defining boundaries of an intersection for purposes of assigning crashes to the intersection for model development. Especially for intersection configurations that have a large footprint such as the crossroad ramp terminal at a single-point diamond interchange and some of the new alternative intersection configurations (e.g., diverging-diamond ramp terminals), a consistent approach for assigning crashes to intersections for crash prediction and comparison of alternative intersection configurations is necessary.

Chapter 11.
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Chapter 12. Abbreviations, Acronyms, Initialisms, and Symbols

AASHTO	American Association of State Highway and Transportation Officials
ANSI	American National Standards Institute
CMF	crash modification factor
CURE	cumulative residual
DOT	Department of Transportation
EB	Empirical Bayes
FB	Full Bayesian
FHWA	Federal Highway Administration
HSIS	Highway Safety Information System
HSM	*Highway Safety Manual*
IHSDM	Interactive Highway Safety Design Model
IIA	independence of irrelevant alternatives
ISATe	Enhanced Interchange Safety Analysis Tool
ITE	Institute of Transportation Engineers
LRS	linear reference system
MNL	multinomial logit
MV	multiple-vehicle
NB	negative binomial
NeXTA	Network Explorer for Traffic Analysis
NCHRP	National Cooperative Highway Research Program
RTM	regression-to-the-mean
SDF	severity distribution function
SP	single-point diamond interchange
SPF	safety performance function
SV	single-vehicle
TD	tight diamond interchange
TWLTL	two-way left-turn lane
AZ	Arizona
CA	California
FL	Florida
IL	Illinois
KY	Kentucky
MA	Massachusetts
MI	Michigan
MN	Minnesota
MO	Missouri
NH	New Hampshire
NV	Nevada
OH	Ohio
PA	Pennsylvania

TN	Tennessee
UT	Utah
WA	Washington
ST	stop control
SG	signal control
3ST	three-leg intersections with minor road stop control
3STT	three-leg intersection with minor road stop control where through movement makes turning maneuver
3SG	three-leg intersections with signal control
3aST	three-leg intersections with all-way stop control
4ST	four-leg intersections with minor road stop control
4SG	four-leg intersections with signal control
4aST	four-leg intersections with all-way stop control
5SG	five-leg intersections with signal control
D3ex	three-leg terminals with diagonal exit ramp
D3en	three-leg terminals with diagonal entrance ramp
D4	four-leg terminals with diagonal ramps
A4	four-leg terminals at four-quadrant parclo A
B4	four-leg terminals at four-quadrant parclo B
A2	three-leg terminals at two-quadrant parclo A
B2	three-leg terminals at two-quadrant parclo B
K	fatal
A	incapacitating injury
B	non-incapacitating injury
C	possible injury
O or PDO	property-damage-only
FI	fatal-and-injury
j	refers to the severity levels predicted by the SDFs in Chapter 19 of the HSM and takes the value of either K, A, B, or C; for general discussion (e.g., Eq. 27 and beyond) j's values to be determined by the analysis
J	all possible injury outcomes for crash r
z	refers to the severity grouping predicted by the SPFs in Chapter 19 of the HSM; takes the value of either "FI" or "PDO"
m	refers to observed crashes at each fatal or injury severity level as part of SDF calibration; takes the value of either K, A, B, or C
AADT	annual average daily traffic
$AADT_{maj}$	AADT on the major road (veh/day)
$AADT_{min}$	AADT on the minor road (veh/day)
$AADT_{total}$	AADT on the major and minor roads combined (veh/day) or sum of $AADT_{maj}$, $AADT_{min}$, and $AADT_{fif}$ (veh/day)
$AADT_{fif}$	AADT on the fifth leg (veh/day)

$AADT_{min+fif}$	sum of $AADT_{min}$ and $AADT_{fif}$ (veh/day)
$AADT_{xrd}$	AADT on the crossroad (veh/day)
$AADT_{in}$	AADT on the crossroad leg between ramps (veh/day)
$AADT_{out}$	AADT on the crossroad leg outside of the interchange (veh/day)
$AADT_{ex}$	AADT on the exit ramp (veh/day)
$AADT_{en}$	AADT on the entrance ramp (veh/day)
$AADT_{ramp}$	sum of ramp AADTs
TEV	total entering volume
PedVol	sum of daily pedestrian volumes crossing all intersection legs (pedestrians/day), only considering crossing maneuvers immediately adjacent to the intersection (e.g., along a marked crosswalk or the extended path of any approaching sidewalk)
i	refers to intersection type *i*
y	refers to geometric/traffic control feature *y* with a CMF
$N_{expected}$	expected average crash frequency obtained by combining the predicted average crash frequency ($N_{predicted}$) with the observed crash frequency ($N_{observed}$) using the EB method
$N_{predicted}$	predicted average crash frequency obtained using the appropriate SPF
$N_{observed}$	observed crash frequency
$N_{predicted\ int}$	predicted average crash frequency for an individual intersection for the selected year (crashes/year)
$N_{spf\ int}$	predicted average crash frequency for an intersection with base conditions (crashes/year) or predicted total average crash frequency of intersection-related crashes for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions) (crashes/year) or predicted average crash frequency of a crossroad ramp terminal at a single-point diamond interchange with base conditions (crashes/year)
N_{bi}	predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle crashes) (crashes/year)
N_{bimv}	predicted average crash frequency of MV crashes of an intersection for base conditions (crashes/year)
N_{bisv}	predicted average crash frequency of SV crashes of an intersection for base conditions (crashes/year)
N_{pedi}	predicted average crash frequency of vehicle-pedestrian crashes of an intersection (crashes/year)
N_{bikei}	predicted average crash frequency of vehicle-bicycle crashes of an intersection (crashes/year)

$N_{bimv(FI)}$	predicted average crash frequency of MV, FI crashes of an intersection for base conditions (crashes/year)
$N_{bimv(total)}$	predicted average crash frequency of MV crashes (all severities) of an intersection for base conditions (crashes/year)
$N_{bimv(PDO)}$	predicted average crash frequency of MV, PDO crashes of an intersection for base conditions (crashes/year)
$N'_{bimv(FI)}$	preliminary value for predicted average crash frequency of MV, FI crashes of an intersection for base conditions (crashes/year)
$N'_{bimv(PDO)}$	preliminary value for predicted average crash frequency of MV, PDO crashes of an intersection for base conditions (crashes/year)
$N_{pedbase}$	predicted average crash frequency of vehicle-pedestrian crashes for base conditions at signalized intersections (crashes/year)
$N_{spf,w,SGn,at,z}$	predicted average crash frequency of a signal-controlled crossroad ramp terminal of site type *w* (*w* = D3ex, D3en, D4, A4, B4, A2, or B2) with base conditions, *n* crossroad lanes, all collision types (*at*), and severity *z* (*z* = FI, PDO) (crashes/year)
$N_{spf,w,ST,at,z}$	predicted average crash frequency of a one-way, stop-controlled crossroad ramp terminal of site type *w* (*w* = D3ex, D3en, D4, A4, B4, A2, B2) with base conditions, all collision types (*at*), and severity *z* (*z* = FI, PDO)
$N_{p,w(i),x(i),at,m,t}$	predicted crash frequency for site *i* with site type *w(i),* year *t*, control type *x(i),* for all collision types (*at*), and severity *m* (*m* = K, A, B, C)
C_i	calibration factor to adjust the SPF for intersection type *i* to local conditions
w	weighted adjustment to be placed on the HSM predictive model estimate
k	overdispersion parameter associated with the SPF
f_{bikei}	bicycle crash adjustment factor for intersection type *i*
f_{pedi}	pedestrian crash adjustment factor for intersection type *i*
CMF_{yi}	crash modification factors specific to intersection type *i* and specific geometric design and traffic control features *y*
CMF_{1p}	crash modification factor for number of bus stops within 1,000 ft of the center of the intersection
CMF_{2p}	crash modification factor for presence of one or more schools within 1,000 ft of the center of the intersection
CMF_{3p}	crash modification factor for number of alcohol sales establishments within 1,000 ft of the center of the intersection
n_{lanesx}	maximum number of traffic lanes crossed by a pedestrian, including through and turning lanes, in any crossing maneuver at the intersection considering the presence of refuge islands (only raised or depressed refuges are considered)
exit_free_right	number of exit ramps with free-flow right turns (0, 1, or 2)
p_{ni}	proportion of total crashes for unlighted intersections that occur at night
a, *b*, *c*, *d*, *e*, and *f*	estimated regression coefficients or estimated SDF coefficients
$P_{o,aS,ac,at,KAB}$	observed probability of a severe crash (i.e., K, A, or B) for all collision types (*at*), all sites (*aS*), and all control types (*ac*)

$N_{o,w(i),x(i),at,m,t}$	observed crash frequency for site *i* with site type *w(i),* year *t*, control type *x(i),* for all collision types (*at*), and severity *m* (*m*= K, A, B, C)
t	refers to year when summing observed crash counts for SDF calibration
r	refers to an individual crash in the general discussion of severity modeling
q	refers to a severity level that falls within the nest of the nested logit
$P_{aS,x,at,K}$	probability of a fatal crash (given that a fatal or injury crash occurred) for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
$P_{aS,x,at,A}$	probability of an incapacitating injury crash (given that a fatal or injury crash occurred) for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
$P_{aS,x,at,B}$	probability of a non-incapacitating injury crash (given that a fatal or injury crash occurred) for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
$P_{aS,x,at,C}$	probability of a possible injury crash (given that a fatal or injury crash occurred) for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
$P_{K\|K+A,aS,x,at}$	probability of a fatal crash given that the crash has a severity of either fatal or incapacitating injury for all ramp terminal sites (*aS*) based on all collision types (*at*) and control type *x* (*x* = ST: one-way stop control; Sgn: signal control, n-lane crossroad)
$P_{p,aS,ac,at,KAB}$	predicted probability of a severe crash (i.e., K, A, or B) for all collision types (*at*) all sites (*aS*) and all control types (*ac*)
$P_{3SG,at,K}$	probability of a fatal crash (given that a fatal or injury crash occurred) for three-leg signalized intersections (3SG) based on all collision types (*at*)
$P_{3SG,at,A}$	probability of an incapacitating injury crash (given that a fatal or injury crash occurred) for three-leg signalized intersections (3SG) based on all collision types (*at*)
$P_{3SG,at,B}$	probability of a non-incapacitating injury crash (given that a fatal or injury crash occurred) for three-leg signalized intersections (3SG) based on all collision types (*at*)
$P_{3SG,at,C}$	probability of a possible injury crash (given that a fatal or injury crash occurred) for three-leg signalized intersections (3SG) based on all collision types (*at*)
$P_{K\|KAB,3SG,at}$	probability of a fatal crash given that the crash has a severity of either fatal, incapacitating injury, or non-incapacitating injury for three-leg signalized intersections (3SG) based on all collision types (*at*)
$P_{A\|KAB,3SG,at}$	probability of an incapacitating injury crash given that the crash has a severity of either fatal, incapacitating injury, or non-incapacitating injury for three-leg signalized intersections (3SG) based on all collision types (*at*)

$P_{4x,at,K}$	probability of a fatal crash (given that a fatal or injury crash occurred) for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control)
$P_{4x,at,A}$	probability of an incapacitating injury crash (given that a fatal or injury crash occurred) for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control)
$P_{4x,at,B}$	probability of a non-incapacitating injury crash (given that a fatal or injury crash occurred) for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control)
$P_{4x,at,C}$	probability of a possible injury crash (given that a fatal or injury crash occurred) for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control)
$P_{K\|KA,4x,at}$	probability of a fatal crash given that the crash has a severity of either fatal or incapacitating injury for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control)
$P_{A\|KA,4x,at}$	probability of an incapacitating injury crash given that the crash has a severity of either fatal or incapacitating injury for 4-leg intersections (*4x*) based on all collision types (*at*) and control type *x* (*x* = ST: minor road stop control; SG: signal control)
$P_r(j)$	probability of crash *r* having injury outcome *j*
$P_r(q\|j)$	probability of crash *r* having injury outcome *q*, conditioned on the outcome being in category *j*
V_j	systematic component of crash severity likelihood for severity *j*
V_{KA}	systematic component of crash severity likelihood for severity *KA*
V_{KAB}	systematic component of crash severity likelihood for severity *KAB*
S_{jr}	set of linear functions that define how injury severity outcome *j* for crash *r* is determined
X_{jr}	a row of observed characteristics (e.g., driver, vehicle, roadway, environment) associated with crash *r* that have an impact on injury severity outcome *j*
β_j	a vector of parameters to be estimated that quantify how the characteristics in X_{jr} impact injury severity outcome *j*
ε_{jr}	a disturbance term that accounts for unobserved and unknown characteristics of crash *r* that impact injury severity outcome *j*
LS_{jr}	"log-sum" or "inclusive value" for the nest (i.e., the expected value of the linear functions for the outcomes within the nest)
ϕ_j	"log-sum coefficient" to be estimated
$C_{sdf,aS,x}$	calibration factor to adjust SDF for local conditions for all ramp terminal sites (*aS*) and control type *x* (*x* = ST: stop control, Sgn: signal control, n-lane crossroad)
$I_{p,lt}$	protected left-turn operation indicator variable for crossroad (= 1 if protected operation exists, 0 otherwise)
I_{ps}	non-ramp public street leg indicator variable (= 1 if leg is present, 0 otherwise)
I_{light}	intersection lighting indicator variable (1 if lighting is present, 0 otherwise)

n_{dw}	number of unsignalized driveways on the crossroad leg outside of the interchange and within 250 ft of the ramp terminal
n_{ps}	number of unsignalized public street approaches to the crossroad leg outside of the interchange and within 250 ft of the ramp terminal
n_{majLTL}	total number of left-turn lanes on both major road approaches (0, 1, or 2)
$n_{majthru}$	total number of through lanes on the major road
n_{majRTL}	total number of right-turn lanes on both major road approaches (0, 1, or 2)
n_{sites}	number of sites
n_c	number of years in calibration period