Appendix A – Literature Review

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Chapter 1: Executive Summary

The purpose of this literature review was to build a knowledge-base for Project 17-70 and address key topic areas to help the research team further refine initial elements of the research plan such as the framework for crash prediction models and the data collection plan. The following presents an overview of the literature review focused on the key findings that informed future activities within Project 17-70.

1.1 SCOPE AND ORGANIZATION OF THE LITERATURE REVIEW

The literature review focused on: 1) Reviewing the literature related to roundabout safety and crash prediction; 2) Identifying candidate roundabout sites and configurations; 3) Identifying key design considerations and challenges practitioners face in implementing roundabouts; and 4) Establishing a vernacular for roundabout-related crash terminology.

The research team consulted U.S.-based and international research findings focused on roundabout crash prediction and roundabout safety performance. The literature review also included outreach to 17 public agencies (state and local agencies) to learn and gather information from agencies with experience designing, implementing, maintaining, and operating single and multilane roundabouts.

The literature review is organized into six chapters.

Chapter 1: Executive Summary – Provides an overview of the findings from the literature review.

Chapter 2: Critical Review of Literature – Presents the roundabout geometric and operational features found in previous research to influence crash frequency and/or severity. It also includes an introduction to crash prediction modeling techniques and a summary of potential transferable crash prediction models and/or crash modification factors to Project 17-70.

Chapter 3: Outreach to Public Agencies with Roundabouts – Presents a summary of the outreach conducted to public agencies with roundabouts and findings related to topics such as roundabout inventories available for potential use in Project 17-70; roundabouts modified since initial construction that may be useful sites for Project 17-70; and information related to driver learning curve.

Chapter 4: Candidate Roundabout Configurations – Synthesizes findings from the literature and outreach to public agencies and presents candidate roundabout configurations for Project 17-70 to consider. The candidate roundabout configurations may be revised as Task 2 Develop Data Collection Plan and Task 3 Implement Data Collection Plan progress.

Chapter 5: Alternative and Initial Recommendation for Preferred Statistical Modeling Approaches – Presents more detailed information on the alternative statistical modeling approaches that could be used to develop the roundabout crash prediction models. This chapter

also includes an initial recommended modeling approach. As Task 3 Implement Data Collection Plan and Task 4 Identify Appropriate Modeling Procedure and Approach progress the selected modeling approach may change or evolve to better adapt to the data and initial findings.

Chapter 6: Roundabout Related Crash Definitions – Synthesizes findings from the literature and outreach to public agencies and presents a proposed crash definition for crashes at roundabouts.

1.2 FINDINGS FROM THE LITERATURE REVIEW

The following sub-sections present a brief overview of the findings from literature. For additional information regarding these findings, please see the chapters noted.

1.2.1 Geometric and Operational Features Influencing Crash Frequency and/or Severity

Previous roundabout research focused on crash prediction and safety performance of roundabouts found the following geometric and operational features to affect crash frequency and/or severity.

- Volume Characteristics
  - Vehicle AADT (disaggregated by entering, circulating and exiting movements); and
  - Pedestrian, bicycle, and motorcycle volume data.
- Basic Configuration Characteristics
  - Number of approaching or entering lanes; and
  - Number of circulating lanes.
- Geometric Characteristics
  - Entry width;
  - Central island diameter;
  - Angle between approach legs;
  - Inscribed circle diameter;
  - Circulating roadway width; and
  - Approach lane width.
- Speed Related Characteristics:
  - Entering, exiting and circulating vehicle speed (e.g., free mean speed, 85th percentile speed); and
  - Variation in vehicle speed (e.g., between vehicles, for one vehicle through roundabout).
- Other Characteristics
  - Sight distance (from perspective of approaching vehicle).

Chapter 2 provides additional information about the U.S.-based and international studies reviewed to create the summarized list above.

1.2.2 Existing Roundabout Databases or Inventories Available from Public Agencies

Based on outreach to 17 public agencies, there are approximately 11 agencies that maintain a roundabout database or inventory, are willing to, and able to share their respective roundabout inventory and other data for use in Project 17-70. The databases or inventories tend to contain basic attributes such as location, type (i.e., single-lane or multilane), and in some instances roundabout size (i.e., inscribed circle diameter). Additional important information such as traffic volumes and crash data will need to be obtained by the research team likely through specific data requests to the agency or in some limited cases the research team may be able to use databases from recently completed research projects. Chapter 3 contains additional information.

1.2.3 Modified Roundabouts and Driver Learning Curve

Based on outreach to public agencies and general knowledge from the research team members active participation in roundabout design and implementation in the U.S., there are few roundabouts that have been modified since their initial construction. The current list of roundabouts the research team is aware of that have been modified since their initial construction includes:

- SR 60/Coronado Drive – Clearwater, FL
- I-17 NB Ramps/Happy Valley Road – Phoenix, AZ
- York Road / Dulaney Valley Road / Joppa Road - Towson, MD
- US 1/34th Street/Perry Street – Mount Rainier, MD
- MD 175 / Odenton Rd / Sappington Station Rd– Fort Meade, MD
- US 15/US 50 – Gilberts Corner, VA
- Dowling/Seward Highway Ramp Terminal Intersections (two roundabouts) – Alaska
- US 34/US 63 – Iowa
- I-90/SR 202 Ramp Terminal Intersections (two roundabouts) – Washington State
- SR 117 in Lebanon County – Pennsylvania
- Hazel Dell Parkway/126th and Hazell Dell Parkway/Main Street – Carmel, Indiana
- Hillsborough/Pullen Rd-Oberlin Rd – North Carolina

The current sample listed above is likely too small to be able to definitively quantify the safety effects of those modifications; however, those specific roundabouts may be able to provide qualitative lessons-learned that practitioners could apply to roundabout design and implementation.

Driver learning curve at roundabouts in the U.S. has not been thoroughly researched to-date. Initial findings indicate that driver error or hesitancy at multilane roundabouts dissipates relatively quickly over the first six months of operation. The research team explored the driver learning curve topic in outreach conversations with public agency representatives and also reviewed the general literature to seek additional information. This is a topic area the research team will continue to be focused on as an area of interest for Project 17-70 and a topic that has yet to be well-researched.

Chapter 3 presents additional information regarding the modified roundabouts listed above as well as the additional discussion of the findings related to driver learning curve.

1.2.4 Summary of Candidate Roundabout Configurations

Based on review of the literature and outreach to public agencies, the research team identified as a starting point, two basic types of roundabout crash prediction models. One type focused on the intersection-level crash prediction at roundabouts. These would be used for planning-level analyses to inform network screening and initial considerations regarding type of intersection traffic control. The second type would be focused on approach-level crash prediction models to be used to inform design decisions.

The intersection-level roundabout configurations the research team would ideally include in Project 17-70 are:

- Urban/Suburban single-lane roundabouts (for posted speed limits <45 mph and ≥45 mph);
- Rural single-lane roundabouts (for posted speed limits <45 mph and ≥45 mph);
- Urban/Suburban multilane roundabouts (for posted speed limits <45 mph and ≥45 mph); and
- Rural multilane roundabouts (for posted speed limits <45 mph and ≥45 mph).

The approach-level roundabout configurations the research team would ideally include in Project 17-70 are:

- Single-lane roundabout approaches (with and without right-turn bypass lanes).
- Multilane roundabouts with differing approaching lane configurations:
    - Two-lane entry vs. a single circulating lane;
    - Two-lane entry vs. two circulating lanes; and
    - Single-lane entry vs. two circulating lanes.

If sufficient data is available, an additional a category for more than two entry lanes and/or more than two circulating lanes would be included in Project 17-70. Within the set of roundabout sites, the research team will work to identify variations in specific attributes such as inscribed circle diameter, angle between approach legs, and other similar characteristics found to or believed to possibly contribute to crash frequency and/or severity at roundabouts.

Chapter 4 presents additional discussion and information related to the candidate roundabout configurations briefly presented above.

1.2.5 Statistical Modeling Approaches

The SPFs and CMFs modeling approaches that have been applied in similar research efforts are listed below.

Safety Performance Functions (SPFs)

- **Negative binomial regression model** - The most frequently used method to develop the SPF. The negative binomial regression model is an appropriate model for rare events

(such as crashes) which show overdispersion. The negative binomial model also provides the estimated overdispersion parameter that is required to apply the EB method in the HSM.

Crash Modification Factors (CMFs)

- **Before-After with Empirical Bayes** - Before-after EB methods are preferred to cross-sectional if before-after data are available and the sample size is big enough. Compared with other methods, the before-after EB method is a statically robust method, which can effectively account for the regression-to-the-mean, traffic volume changes over time and non-treatment related time trends. Because roundabouts are fairly new in the U.S. and changes made are rarely done in isolation the opportunity to conduct before-after studies is probably limited.
- **Cross-Sectional** -When there are not enough before-after data for the before-after EB method, the cross-sectional method will be applied. The cross-sectional method has been proven as a feasible statistical method to develop CMF and CMF functions when limited before-after data are available. Results should be confirmed with limited before-after data if possible.

Chapter 2 provides an introduction to statistical modeling approaches that could be used to develop roundabout crash prediction models and models potential transferable to Project 17-70. Chapter 5 presents more detailed and thorough discussion of the alternative approaches as well as justification for the initial preferred approach identified above.

1.2.6 Roundabout Related Crash Definitions

The two key areas addressed in the roundabout related crash definitions are: 1) Identifying when a crash near a roundabout is considered a "roundabout-related" crash; and 2) Defining consistent definition for crash types at roundabouts.

With respect to identifying crashes that are roundabout-related crashes (i.e., more generally intersection-related crashes), the research team proposes using criteria similar to the *Highway Safety Manual* and crash reporting practices in most jurisdictions. Therefore, for Project 17-70 roundabout crashes are as those that occur at the intersection proper, coded by the responding police officer as intersection-related, and crashes occurring from yield line back approximately 250 feet.

With respect to crash type definitions, previous international and U.S.-based roundabout crash prediction research has used crash type definitions specific to roundabout or circular intersection forms. These crash types include: entering-circulating, exiting-circulating and other similar categories based on movement at or on approach to the roundabout. U.S. crash databases use the same crash type definitions across different types of intersections; this means crashes at roundabout intersections within the U.S. are coded as crash types like rear-end, sideswipe, and other traditional crash categories.

At this stage of Project 17-70, the research team proposes to advance for further discussion as part of Task 2 Develop Data Collection Plan development both potential crash type definitions (i.e., traditional definitions and roundabout specific definitions). Each of these are connected to

two alternative crash model development approaches being considered within Task 2 and so selecting one set of crash type definitions at this stage of the project would be premature.

Chapter 2: Critical Review of Literature

2.1 INTRODUCTION AND PURPOSE

The literature review within this chapter established a foundation for the research team to focus Project 17-70's resources on roundabout design attributes, data, and modeling approaches most likely to yield crash prediction models practitioners can use to make roundabout implementation and design decisions. To establish a strong foundation, the research team focused on the following topic areas:

- Crash modeling techniques and approaches used in similar crash prediction research that may be applicable to Project 17-70.;
- Roundabout geometric and operational features previous research projects and studies have found to influence crash frequency and severity; and
- Safety Performance Functions (SPFs) and Crash Modification Factors or Functions (CMFs) potentially applicable to Project 17-70 that were developed in previous research.

Chapter 5 of the literature review discusses alternate modeling techniques in further detail and initial recommendations for Project 17-70.

The review of the literature included U.S.-based research, as well as international resources that developed crash prediction models using roundabout data from other countries. The team also reviewed working papers and interim deliverables that researchers have generously shared with us from on-going National Cooperative Highway Research Program (NCHRP) projects and Federal Highway Administration's TOPR. This includes reviewing recent working papers from NCHRP Project 17-62 *Improved Prediction Models for Crash Types and Crash Severities*, and Project 17-63 *Guidance for the Development and Application of Crash Modification Factors*, which are setting the direction for future crash prediction modeling for the Highway Safety Manual. The most recent interim deliverable regarding fatal and severe injury crashes at roundabouts from FHWA's TOPR #34 *Accelerating Roundabout Implementation in the United States* was also reviewed.

The following sections summarize the key findings from the literature; the information is organized into three core sections consistent with the three general topic areas noted above.

2.2 OVERVIEW OF CRASH PREDICTION MODELING TECHNIQUES AND APPROACHES

This section summarizes the modeling approaches applied in developing Safety Performance Functions (SPFs) and Crash Modification Factors (CMFs) for similar research projects. Chapter 5 discusses alternative modeling approaches in more detail and identifies initial recommended approaches for Project 17-70.

2.2.1 Safety Performance Functions

A Safety Performance Function (SPF) is a crash prediction equation that predicts crash frequency (dependent variable) based on site characteristics (independent variables). Generally

speaking, SPFs are developed through multiple regression techniques based on crash data collected over a number of years at sites with similar characteristics and covering a wide range of AADTs (HSM, 2010).

Among different models, the most popular model for crash events is the negative binomial regression model. All of the SPFs developed in the HSM 2010 were determined by assuming that crash frequencies follow a negative binomial distribution.

For crash data, the variance typically exceeds the mean (HSM, 2010), which is called overdispersion. The negative binomial regression model accounts for the overdispersion of crash counts well. This is the reason that the negative binomial regression has become the dominating statistical modeling technique to develop SPFs. In the negative binomial regression model, the variance of number of crashes per year can be written as follows:

$$VAR(y) = E(y) + k * [E(y)]^2$$

Where,

VAR(y) = the variance of number of crashes per year

E(y) = the expected number of crashes per year

k = the overdispersion parameter. If k is zero, this negative binomial regression model becomes a Poisson regression model.

This form or similar forms of the negative binomial regression model has been applied in different studies (e.g., Cameron and Trivedi (1998) and Hauer et al., (2002)). The overdispersion parameter, k, is an important parameter in the negative binomial regression model. It can be used in the empirical Bayes (EB) method to adjust expected crash frequencies as determined by an SPF by considering observed crash counts as well. The EB method is used in various instances, including the development of Crash Modification Factors (CMFs) through before-after studies. When proceeding with the EB method, the expected crashes are calculated as a weighted average of the expected crashes predicted by the SPF and the observed crashes. The weight involved in the calculation can be determined based on the overdispersion parameter:

$$w = \frac{1}{1 + k * n * SPF}$$

Where,

SPF denotes the dependent variable of the Safety Performance Function (i.e., crashes per year) and n is the number of years of observed crash data to be used in the EB method.

The common practice is to assume the overdispersion parameter as a constant. However, several studies suggested that the overdispersion parameter should be modeled as a function of site characteristics when roadway segments are modeled (Hauer (2001), Cafiso et al. (2010), Miaou and Lord (2003), Mitra and Washington (2007)). Although these studies did not focus on

roundabouts, their findings that the overdispersion parameter may vary as some function of site characteristics should be considered for roundabout modeling.

Srinivasan and Bauer (2013) discussed several statistical issues associated with the development of SPFs. These include:

- **Overfitting** - If too many parameters are incorporated into the SPF development, overfitting will occur. The overfitting will result in poor predictive models and will introduce correlation into the model.
- **Correlation among Variables** - Correlation is a statistical measure that indicates the extent to which two or more variables fluctuate together. If strong correlation exists between variables, these variables are not independent to each other. Therefore, it will be difficult to obtain reliable estimation of the effects of a variable. This is critical if these effects are used to infer crash modification factors.
- **Outliers** -The presence of outliers can significantly impact the model development and result in incorrect predictions.

A more detailed review of SPF estimation methods and statistical issues is provided in Chapter 5.

2.2.2 Crash Modification Factors

Crash Modification Factors (CMFs) represent the relative change in crash frequency due to a change in one specific condition, when all other conditions and site characteristics remain constant (HSM, 2010). Therefore, a CMF serves as a multiplicative factor to estimate the number of crashes if a change were made at the study site. Several statistical methods have been used to develop CMFs; these are discussed below.

The before-after method and cross-sectional studies are two most frequently used methods in developing CMFs. Generally speaking, before-after methods are preferred to cross-sectional if before-after data are available and provide a large enough sample size. Within before-after methods, before-after with empirical Bayes is widely used to develop the CMFs (e.g., HSM 2010, NCHRP Report 672, and NCHRP Report 705).

A detailed review of CMF estimation methods and statistical issues is provided in Chapter 5.

2.3 ROUNDABOUT FEATURES INFLUENCING CRASH FREQUENCY AND SEVERITY

A range of U.S.-based and international research has identified roundabout features—geometry, volume, and speed related characteristics—that influence crash frequency and severity. In some instances past research has been able to quantify with some level of confidence the effects on crashes, and in other studies the relationship was found to exist but difficult to quantify.

2.3.1 U.S.-Based Crash Evaluation and Prediction at Roundabouts

Many of the U.S.-based research studies found in the literature focused solely on evaluating the effectiveness of converting existing intersections to roundabouts and/or were not able to (or were not intended to/scoped to) develop crash prediction models that quantified the effect of

geometric or operational features of roundabouts on crash frequency and severity. For example Persaud et al. (2001) and Eisenman et al. (2004) were two of the earliest research projects that evaluated the safety effectiveness of converting stop control and signal controlled intersection to roundabouts. These studies provided the initial findings regarding the safety effectiveness of roundabouts for subsequent studies such as Rodegerdts et al. (2007) to build upon. Since Rodegerdts et al. (2007), Srinivasan et al. (2011) evaluated the safety effectiveness of converting signalized intersections to roundabouts using data from six states. Bagdade et al. (2011) evaluated the safety effectiveness of intersection conversion to roundabouts specific to Michigan. Qin et al. (2013) evaluated the safety effectiveness of intersection conversion to roundabouts specific to 24 roundabouts in Wisconsin.

Persaud et al. (2001) found the safety effect of roundabout conversions in the U.S. to be 40% reduction in total crashes and 80% reduction in injury crashes; this was based on a data set of 23 intersections across seven states. Rodegerdts et al. (2007) considered 55 intersections and found a 35.4% reduction in total crashes and 75.8% reduction in injury crashes.

The above research efforts have been valuable in identifying contexts in which converting existing stop controlled and signal controlled intersections to roundabouts result in reduced crash frequency and severity. These trends include:

- **Control type before roundabout installation** – Consistent safety benefits are realized when converting two-way stop controlled and many signal control intersections to roundabouts (single-lane and double lane). Benefits are larger for injury crashes than total crashes. Conversion from AWSC intersections to roundabout found no apparent safety effect.
- **Number of Lanes** –The safety benefit is larger for single-lane roundabouts than multilane roundabouts. Research for U.S. multilane roundabouts with more than two circulating lanes is limited to Bagdade et al. (2011), which did not find consistent safety benefits relative to previous intersection control.
- **Setting** – Safety benefits for rural intersections converted to single-lane roundabouts tend to be larger than for urban and suburban single-lane roundabouts.
- **AADT** - Safety benefits appear to decrease with increasing AADT, irrespective of control type before conversion, number of lanes and setting (Rodegerdts et al., 2007). This was substantiated by Srinivasan et al. (2011) for previously signal controlled intersections. Using cross-sectional data, Srinivasan et al. (2011) found that as AADT increased total crashes may become higher with the roundabout control than signal control.

Based on these evaluations of intersection conversion to roundabouts, features influencing the safety effectiveness of roundabouts include number of circulatory lanes, setting (or attributes reflected in the setting such as high or low speed approaches), and AADT.

Of the U.S-based research found in the literature, Rodegerdts et al. (2007), Bagdade et al. (2011), and Dixon and Zheng (2013) were the studies that developed safety performance functions (i.e., crash prediction models) to use for crash prediction at roundabout intersections.

Rodegerdts et al. (2007) developed intersection-level crash prediction models and approach-level crash prediction models. The intersection-level models are used for network screening and planning-level assessment of potential safety performance of a single-lane or multilane roundabout at an intersection. These were developed from a data set of 90 roundabouts from across the U.S.; the majority of the roundabouts were single-lane roundabouts in urban or suburban environments. The approach-level models are intended to be used to inform design decisions. These were developed from a subset of approaches in the intersection-level model data set; a total of 139 approaches at 39 roundabouts were used.

Rodegerdts et al. (2007) used a generalized linear modeling approach assuming a negative binomial error distribution for the intersection-level and approach-level crash prediction models. The preferred intersection-level crash models include AADT, number of approaches and number of circulating lanes as the significant variables. Separate intersection-level crash prediction models were developed for single-lane, multilane lane (with two lanes or less), and multilane (with three or four lanes) for total crashes and injury crash prediction. The approach-level models considered a wide range of variables including entry radius, entry width, central island diameter, approach half-width (i.e., approach width on Figure 9), circulating width and others. The approach-level models were developed for specific crash types: entering-circulating, exiting-circulating, and approaching crashes. The preferred crash prediction models for these crash types included a mix of AADT, entry width, angle to next approach, inscribed circle diameter, circulating width, and approach width. These are discussed in more detail in the following section (Section 2.3.2) where the U.S.-based approach-level crash prediction models are compared to crash prediction models developed in other countries. Section 2.4 presents the preferred intersection-level and approach-level crash prediction models developed by Rodegerdts et al. (2007).

Bagdade et al. (2011) developed intersection-level crash prediction models for roundabouts in Michigan. The data set for the crash prediction model development consisted of 36 roundabouts with two or fewer circulating lanes. Bagdade et al. (2011) used a generalized linear modeling approach assuming a negative binomial error distribution. The model development considered total entering AADT, number of circulating lanes, number of approaches, environment (urban vs. rural), and whether or not the intersection was a ramp terminal intersection. The preferred crash prediction model included total entering AADT, number of circulating lanes and if the intersection was a ramp terminal intersection. Crash prediction models were developed for total crashes and a combined fatal and injury crash prediction.

Dixon and Zheng (2013) used a dataset from Oregon focused on single-lane roundabouts with four approaches. Given the similar volume and geometric characteristics represented in their data of 21 roundabouts, they developed a base safety performance function. The base-SPF is an intersection-level crash prediction model; the only input into the SPF is total entering AADT.

2.3.2 International Crash Prediction at Roundabouts

The international literature had more research studies that included crash prediction model development with a focus of identifying roundabout features that influence crash frequency. The literature was limited in connecting roundabout features to crash severity. Studies appear

to have focused on predicting specific crash types to better understand how roundabout design features influence crash occurrence. From those crash types, one could draw connections to potential severities (e.g., sideswipe crashes tend to be less severe than other crash types).

The international literature review included studies from United Kingdom, Australia, Sweden, United States and New Zealand. The team considered hallmark studies from Maycock and Hall (1984), Arndt (1994 and 1998) and Brude and Larsson (2000) that were previously reviewed as part of Rodegerdts et al. (2007) work. Findings from Rodegerdts et al. (2007) were added to this international review to be able to compare side-by-side crash prediction models from other countries with those from the U.S. that considered approach-level geometric characteristics. Also reviewed were two studies from New Zealand one by Harper and Dunn (2005) and most recently by Turner et al. (2009). Table A2-1 summarizes characteristics regarding sample size and models developed of the studies included in the review.

Table A2-1. Summary of Crash Model Characteristics

Corresponding Author	Sample Size	Models Developed
United Kingdom: Maycock and Hall (1984)	84	Total crashes/roundabout Total crashes/crash type Total crashes/approach and by crash type
Australia: Arndt (1994 and 1998)	100	Total crashes/approach and by crash type
Sweden: Brude and Larsson (2000)	650	Crashes per million entering vehicles
United States: Rodegerdts et al. (2007)	90: Intersection-Level 39: Approach-Level (139 approaches)	Total crashes/roundabout Injury crashes/roundabout Total crashes/approach and by crash type
New Zealand: Harper and Dunn (2005)	95	Total crashes/approach and by crash type
New Zealand: Turner et al. (2009)	104	Total crashes/approach and by crash type
India: Anjana and Anjaneyulu (2014)	75 approaches at 20 roundabouts	Total crashes/approach Injury crashes/approach PDO crashes/approach

Note: Table A2-1 is an expansion and update to Rodegerdts et al. (2007) Table 1 Summary of safety models.

Table A2-2, on the following page, summarizes the geometric, operational and other characteristics found to affect crash frequency and/or severity in the international studies noted in Table A2-1. Table A2-2 is an expansion and update to Rodegerdts et al. (2007) Table 2 Summary of geometric, traffic, and other characteristics affecting safety. Below is a legend for the symbols used in Table A2-2:

Table A0-1. Summary of Volume, Geometrics, Speed, and Other Characteristics Affecting Crashes (Table A2-2 is expansion and update to Rodegerdts et al. (2007) Table 2)

Measures	United Kingdom (Maycock & Hall, 1984)					Australia (Arndt, 1994, 1998)					Sweden (Brude & Larsson, 2000)			U.S. (Rodegerdts et al., 2007)			New Zealand (Harper & Dunn, 2005)			New Zealand (Turner et al., 2009)									India: Anjana and Anjaneyulu (2014)		
	SV	APP	Ent/C	Other	Ped	SV	RE	Ent/C	Ext/C	SS	All	Bike	Ped	APP	Ent/C	Ext/C	All	Ent/C	Ped	All	AllH	RE	ENT/C	LoC	Other	Ped	Ent / Bike	Other Bike	All	Injury	PDO
AADT and/or Vehicle Volumes	+	+	+	+	+	+	+	+	+	+	+			+	+	+	+		+	+	+	+	+	+	+	+	+	+	+	+	+
Pedestrian Volumes					+								+													+					
Bicycle Volumes																											+	+			
Percentage of Motorcycles			+	+																											
Number of Approaching/Entering Lanes							+				+	+	+							+		I	I		+	I					
Number of Circulating Lanes								+			+	+	+					-													
Three legs instead of fourlegs											-																				
Presence of bicycle crossings												-																			
Radius (or Diameter) of Central Island			-					+			1	-			D														+		+
Radius of Vehicle Path	-	-				-												-											+	+	
Approach Curvature or Deflection	-							-	-																						

SV = single vehicle;
APP = approaching;
Ent/C = crashes between an entering vehicle and a circulating vehicle;
Other = other non-pedestrian crashes;
Ped = pedestrian crashes;
RE = rear-end crashes on approach;
Ext/C = crashes between an exiting vehicle and a circulating vehicle at multilane roundabouts;
SS = sideswipe crashes on two-lane segments;
AllH = total crashes at roundabouts with approaches posted speed greater than 70 km/hr;
LoC = loss of control crashes;
Ent/Bike = crashes between a vehicle entering and a bicyclist circulating the roundabout;
+ = an increase in this measure increases crash frequency;
- = an increase in this measure decreases crash frequency;
* = the measure had a significant relationship with crash frequency but the relationship was not specified;
I = an increase in this measure increases crash frequency; relationship was not quantified in the preferred model;
D= an increase in this measure decreases crash frequency; relationship was not quantified in the preferred model; and
[1]Optimum 10m (32.8 feet) to 25m (82.0 feet).

Table A0-2. (continued) Summary of Volume, Geometrics, Speed, and Other Characteristics Affecting Crashes (Table A2-2 is expansion and update to Rodegerdts et al. (2007) Table 2)

Measures	United Kingdom (Maycock & Hall, 1984)					Australia (Arndt, 1994, 1998)					Sweden (Brude & Larsson, 2000)			U.S. (Rodegerdts et al., 2007)			New Zealand (Harper & Dunn, 2005)			New Zealand (Turner et al., 2009)									India: Anjana and Anjaneyulu (2014)		
	SV	APP	Ent/C	Other	Ped	SV	RE	Ent/C	Ext/C	SS	All	Bike	Ped	APP	Ent/C	Ext/C	All	Ent/C	Ped	All	AllH	RE	ENT/C	LoC	Other	Ped	Ent / Bike	Other Bike	All	Injury	PDO
Angle to next leg		*	-												-														-		
Road width at entry	*/+	*/-	+/-												+																
Approach Half Width														+																	
Inscribed Circle Diameter																+															
Circulating Width																+	+												-	-	
Gradient		+/+%	+/-%																								I/-%				-%
Pedestrian Crossing Distance																			+												
Weaving Length between Splitter Islands		-			*																								+	+	
Average Flare Length		*																													
Length of Vehicle Path						+																									

SV = single vehicle;
APP = approaching;
Ent/C = crashes between an entering vehicle and a circulating vehicle;
Other = other non-pedestrian crashes;
Ped = pedestrian crashes;
RE = rear-end crashes on approach;
Ext/C = crashes between an exiting vehicle and a circulating vehicle at multilane roundabouts;
SS = sideswipe crashes on two-lane segments;
AllH = total crashes at roundabouts with approaches posted speed greater than 70 km/hr;
LoC = loss of control crashes;
Ent/Bike = crashes between a vehicle entering and a bicyclist circulating the roundabout;
+ = an increase in this measure increases crash frequency;
- = an increase in this measure decreases crash frequency;
* = the measure had a significant relationship with crash frequency but the relationship was not specified;
I = an increase in this measure increases crash frequency; relationship was not quantified in the preferred model;
D= an increase in this measure decreases crash frequency; relationship was not quantified in the preferred model; and
[1]Optimum 10m (32.8 feet) to 25m (82.0 feet).

Table A0-3. (continued) Summary of Volume, Geometrics, Speed, and Other Characteristics Affecting Crashes (Table A2-2 is expansion and update to Rodegerdts et al. (2007) Table 2)

	United Kingdom (Maycock & Hall, 1984)					Australia (Arndt, 1994, 1998)					Sweden (Brude & Larsson, 2000)			U.S. (Rodegerdts et al., 2007)			New Zealand (Harper & Dunn, 2005)			New Zealand (Turner et al., 2009)									India: Anjana and Anjaneyulu (2014)		
Measures	SV	APP	Ent/C	Other	Ped	SV	RE	Ent/C	Ext/C	SS	All	Bike	Ped	APP	Ent/C	Ext/C	All	Ent/C	Ped	All	AllH	RE	ENT/C	LoC	Other	Ped	Ent / Bike	Other Bike	All	Injury	PDO
Splitter Island Type																													+/-		
Posted speed limit	*										+										+										
Free Mean Speed of Circulating Vehicles																							+				+				
Entering Vehicle Speed																				I		I	I	I							
Variation in Vehicle Speed																							I			I					+
85th Percentile Speed						+	+	+	+																						
Reduction in 85th Percentile Speed						+																									
Sight Distance	+	+																				I	I	+							
Distance to first sight of roundabout					*																										
Potential side friction										+																					

SV = single vehicle;
APP = approaching;
Ent/C = crashes between an entering vehicle and a circulating vehicle;
Other = other non-pedestrian crashes;
Ped = pedestrian crashes;
RE = rear-end crashes on approach;
Ext/C = crashes between an exiting vehicle and a circulating vehicle at multilane roundabouts;
SS = sideswipe crashes on two-lane segments;
AllH = total crashes at roundabouts with approaches posted speed greater than 70 km/hr;
LoC = loss of control crashes;
Ent/Bike = crashes between a vehicle entering and a bicyclist circulating the roundabout;
+ = an increase in this measure increases crash frequency;
- = an increase in this measure decreases crash frequency;
* = the measure had a significant relationship with crash frequency but the relationship was not specified;
I = an increase in this measure increases crash frequency; relationship was not quantified in the preferred model;
D= an increase in this measure decreases crash frequency; relationship was not quantified in the preferred model; and
[1]Optimum 10m (32.8 feet) to 25m (82.0 feet).

Findings from the research projects summarized in Table A2-2 indicate the international trends discussed below.

Volume Characteristics

Vehicle volumes are a consistently strong characteristic influencing crash frequency at roundabouts. More recent research categorizes volume as entering AADT and circulating AADT (Harper and Dunn, 2005; Rodegerdts et al., 2007; and Turner et al. 2009), when possible to disaggregate it from total entering AADT at the intersection.

When available, pedestrian, bicycle and motorcycle volume information are useful data for predicting crash types unique those to modes as well as more severe crashes. This is consistent with findings from TOPR 34 Accelerating Roundabout Implementation in the United States, which found motorcyclists more likely to crash at roundabouts than other road users and also found motorcyclists to be involved in 34% of fatal crashes that have occurred at roundabouts.

Basic Configuration Characteristics

The approaching or entering number of lanes at the roundabout was one of the more prominent basic roundabout configurations found to be associated with crashes. Several studies found that crashes increase as the number of these lanes increase (Arndt, 1994 & 1998; Brude & Larsson, 2000; Turner et al., 2009). Arndt (1994 & 1998) and Brude & Larsson (2000) also found that crashes increase as the number of circulating lanes increases. When considering width in the aggregate, Maycock and Hall (1984) also found that increasing the entry width tends to be associated with an increase in entry-circulating crashes.

Geometric Characteristics

There is a wide range of detailed geometric characteristics that various studies, most notably Maycock and Hall (1984), have included and quantified in crash prediction models for roundabouts. No single geometric measure is seen consistently across the international literature. This may reflect the range of information available to researchers when undertaking their respective studies as well as the variation in roundabout design across different countries. Specific to the U.S., Rodegerdts et al. (2007) found that increasing the dimensions of the following roundabout characteristics is associated with changes in crashes as noted below:

- Entry width increases entering/circulating collisions;
- Central island diameter decreases entering/circulating collisions;
- Angle between approach legs decreases entering/circulating collisions;
- Inscribed circle diameter increases exiting/circulating collisions;
- Central island diameter increases exiting/circulating collisions;
- Circulating width increases exiting/circulating crashes; and
- Lane width increases approach crashes.

Speed Related Characteristics

Turner et al. (2009) found significant relationships between different measures of speed and crash frequency of different crash types. Free mean speed (i.e. average free flow speed) of circulating vehicles is a significant variable in the entering-circulating and entering-bicycling preferred crash prediction models. Higher entering vehicle speeds were also found to be associated with increases in total, rear-end, entering-circulating, and loss of control crashes. Entering vehicle speed was not a variable in the preferred crash prediction models for those crashes types, but research by Turner et al. (2009) did find a distinct relationship between it and crash occurrence.

Similarly, Arndt (1994 & 1998) found 85th percentile speed entering the roundabout and reduction in 85th percentile speed as a driver travels through the roundabout to influence certain crash types at a roundabout. Arndt (1994 & 1998) found that an increase in the 85th percentile speeds entering the roundabout was associated with an increase each of the crash types explored in that research, except for sideswipe crashes. Higher reductions in 85th percentile speed (i.e., the more a driver would need to reduce his/her speed to travel through the roundabout) were found to be associated with an increase single-vehicle crashes.

Anjana and Anjaneyulu (2014) found that an increase in the relative difference between approach and circulating speeds was associated with an increase in all PDO crashes at an approach.

Finally, Rodegerdts et al. (2007) also explored the contribution of vehicle speeds to crash frequency at roundabouts. The research explored the effects of absolute vehicle speeds (e.g., free mean entering vehicle speed) and relative speeds (e.g., speed consistency or variation). They developed models using AADT and observed speeds measured at four different points through the roundabout. The speed related models were deemed inadequate based on the weak effects found with the speed variables. However, further exploring vehicle speeds as a predictor of crash frequency at roundabouts was found to be promising particularly with an expanded data set.

More recent research regarding the influence of vehicle speeds and crashes is reviewed in Chapter 5 of the literature review.

Other Characteristics

Increasing sight distance (from the perspective of approach driver) was previously quantified by Maycock and Hall (1984) as being associated with an increase in single-vehicle crash and crashes on roundabout approaches. More recent research by Turner et al. (2009) confirmed this initial finding and found increasing sight distance to be associated with an increase loss-of-control crashes, rear-end and entering-circulating crashes. The relationship of sight distance to rear-end and entering-circulating crashes was not included in the preferred crash model for those crash types but was found to be significant in alternate models which did not fit the data as well as the preferred model. U.S.-based research of 26 single-lane roundabouts by Angelastro (2010) also found sight distance to be a statistically significant variable in predicting entry rear-

end, loss of control and total crashes as well as a statistically significant variable in predicting 85th percentile approach and entry vehicle speeds.

2.3.3 U.S. Use of International Roundabout Crash Prediction Models

As part of the research effort by Rodegerdts et al. (2007), the research team evaluated the feasibility of using non-U.S. models to represent U.S. crash frequency at roundabouts and found this approach to be undesirable. Researchers calibrated the models developed from sites and data in other countries to U.S. data and then used them to predict crashes at U.S. roundabouts. Statistical goodness of fit tests were used to evaluate the how well the internationally developed, U.S.-calibrated models predicted crash performance at U.S. roundabouts. Rodegerdts et al. (2007) found the intersection-level models from Sweden, United Kingdom and France did not fit U.S. data very well. The models also included other limitations such as limited number of approaches and inherent assumption of linear relationship between volume and crashes. The approach-level models from the United Kingdom were calibrated and evaluated. Similarly, poor fit was found with U.S. data. Rodegerdts et al. (2007) concluded that it was undesirable to use international roundabout crash prediction models at the intersection or approach-levels.

2.3.4 Bicycle and Pedestrian Crash Evaluation and Prediction at Roundabouts

Four of the international research studies discussed above developed crash prediction models specific to bicycle and/or pedestrian crashes.

The four studies that developed pedestrian crash prediction models consistently included vehicle and/or pedestrian volume data as statistically significant input variables indicating that, as each increases, the number of pedestrian crashes also increases. Brude and Larsson (2000) also found that increasing entry lanes and circulating lanes was associated with an increase in pedestrian crashes. This was substantiated by Turner et al. (2009) finding related to increasing the number of entering lanes. Turner et al. (2009) also found increasing variation in vehicle speed to be associated with an increase in pedestrian crashes. Harper and Dunn (2005) found increasing pedestrian crossing distance, which is directly related to the number of entering vehicle lanes, to be associated with an increase in pedestrian crashes.

Two of the studies discussed above developed crash prediction for bicycle crashes. Brude and Larsson (2000) found increasing entering lanes and increasing circulating lanes to be associated with an increase in bicycle crashes. They also found that increasing bicycle crossings and increasing the radius of the central island was associated with a decrease in bicycle crashes. Turner et al. (2009) found that collisions between entering vehicles and circulating bicyclists increased as vehicle and bicycle volumes increased, as free mean speed of vehicles increased. Turner et al. (2009) also found in an increase in entering vehicle-circulating bicyclists collisions as the downgrade approach to the roundabout increased.

In addition to the above bicycle and pedestrian research at roundabouts, several other relatively recent studies in the literature have considered bicycle and pedestrian safety at roundabouts.

An Australian study, Cummings (2012) found that during 2005-2009 nearly 50% of crashes at roundabouts involved bicyclists in Melbourne and 24% in Victoria. "Entering-circulating" crashes accounted for 48% of total crashes at roundabouts in Victoria and 82% of total vehicle-bicycle crashes. This indicates that the primary conflict point at roundabouts at which vehicle-bicycle crashes occur is the entering-vehicle to circulating bicyclist conflict.

Daniels et al. (2009) considered the influence of bicycle facilities at roundabouts on safety for roundabouts in Belgium. Findings from the study indicated that roundabouts with bicycle lanes adjacent to the circulating vehicle lane experienced an increase in bicycle injury crashes of 93%. This is likely due to the right-hook conflict between a vehicle exiting the roundabout and bicyclist continuing to circulate around the roundabout. The same study indicated that the increase in bicycle injury crashes may be negated if a separated bicycle lane is provided and designed such that motor vehicles must yield to bicyclists (Daniels et al., 2009). Daniels et al. (2010) evaluated 90 roundabouts in Belgium in a cross-sectional study that created risk models for various users based on traffic and geometric features. In general, Daniels et al. (2010) found that bicyclists and pedestrians are overrepresented in crashes at the roundabouts evaluated. For bicyclists, increasing motor vehicle volumes, increasing bicycle volumes, increasing moped volumes, and the presence of a bicycle lane were correlated with an increase in bicycle crash frequency. For pedestrians, in addition to volume metrics, presence of sidewalk and Zebra crossings were tested for the pedestrian model and found to be significant, but multicollinearity issues (i.e., their presence tends to correspond with increasing pedestrian volumes) led Daniels et al. (2010) to leave these variables out of the final models.

There is limited U.S.-based research specific to bicycle and pedestrian safety at roundabouts. Increasing attention in the U.S. has been given to access needs for disabled (e.g., visually impaired pedestrians) at roundabouts. Less focus has been given to the general bicycle and pedestrian population. Rodegerdts et al. (2007) gathered data specific to bicycle and pedestrian behavior at roundabouts. The data were used to develop design guidance ultimately incorporated into NCHRP Report 672 *Roundabouts: An Informational Guide, Second Edition*. These data were also used by Harkey and Carter (2006) who analyzed the interactions between motorists and pedestrians or bicyclists at roundabouts. They did not find any substantial safety problems for non-motorists at roundabouts based on conflicts or collisions, but did highlight some aspects of roundabout design that require additional care to ensure safe access for pedestrians and bicyclists, for example, in designing exit legs to ensure proper sight lines and motor vehicle speeds and in designing the junction of the circulatory lane and exit lane. Harkey et al. also suggested that multilane roundabouts may require additional traffic control measures to enhance pedestrian. Other recent U.S.-based research specific to bicycle and pedestrian safety at roundabouts include Arnold et al. (2013) and Hourdos et al. (2012).

Arnold et al. (2013) conducted a study of three multilane roundabouts for the California Department of Transportation (Caltrans). The study used video analysis and in-person observations and surveys to identify factors that contribute to crashes involving bicyclists and pedestrians at multilane roundabouts. Crash data were analyzed for two roundabouts and

video data analyzed at three roundabouts. The study identified the following qualitative findings:

- Bicyclists are more likely than pedestrians to perceive multilane roundabouts as being uncomfortable to travel through (53% of bicyclists said they were comfortable surveyed onsite, compared to 60% of pedestrians surveyed);
- Drivers were more likely to yield to pedestrians at locations with higher pedestrian volumes;
- Bicyclists must take a lane in the roundabout to avoid right-hook collisions; and
- Bicyclists must be able to judge whether there is enough of a gap to enter and take a lane, which can be difficult with higher circulating speeds.

Hourdos et al. (2012) evaluated pedestrian and bicycle risk at roundabout crossings in Minnesota. The study was conducted for Minnesota DOT in response to public concerns about being able to safely cross at roundabouts. Hourdos et al. (2012) conducted an observational study between pedestrian and/or bicyclists and vehicles at two urban roundabouts in Minneapolis and St. Paul. Findings from the study focused on road user behavior as opposed to the impact on frequency or severity of crashes. Researchers found drivers entering the roundabout were more likely to yield to pedestrians than drivers exiting the roundabout.

2.4 SAFETY PERFORMANCE FUNCTIONS AND CRASH MODIFICATION FACTORS POTENTIALLY APPLICABLE TO PROJECT 17-70

This section presents safety performance functions (SPFs) and crash modification factors (CMFs) potentially transferrable to Project 17-70. SPFs and CMFs are the basic building blocks to crash prediction models. There are two basic approaches to using SPFs and CMFs for crash prediction purposes.

1. Develop SPFs with AADT only variables that are associated with a base condition and provide CMFs that are multiplied to the results of the SPF prediction when a practitioner analysis scenario differs from the base condition. For example, a SPF for a signal controlled intersection could be set to have no exclusive left-turns present. If a practitioner is evaluating a scenario with a left-turn lane present, he or she would apply the CMF for a left-turn lane. The current crash prediction models in the Highway Safety Manual (HSM) follow this approach.
2. Develop fully specified SPFs that include input variables for each geometric and operational attribute found to be significant. Practitioners input the variables as reflected in their analysis scenario directly into the SPF. CMFs are not generated or provided to use with the SPF. Recent research developing crash prediction models for roundabouts follow this approach (e.g., Rodegerdts et al., 2007; Turner et al., 2009; Bagdade et al., 2011).

The intent of this section is to identify what previous research produced these crash modeling building blocks (SPFs and CMFs) that Project 17-70 may be able to use or work from. This section focuses on U.S.-based research given that past attempts to calibrate non-U.S. based research to U.S. data found that such an approach was undesirable (Rodegerdts et al., 2007).

2.4.1 Safety Performance Functions Potentially Transferable to Project 17-70

Rodegerdts et al. (2007) provide crash prediction models estimated from a national US database and in this regard hold the most promise for Project 17-70 to review more closely and potentially build from. Given that Bagdade et al. (2011) and Dixon and Zheng (2013) focused exclusively on roundabouts in Michigan and Oregon, respectively, the specific crash prediction models developed are less immediately transferable to Project 17-70. However, the data used for the crash prediction models developed by Bagdade et al. (2011) and Dixon and Zheng (2013) do serve as source of potential additional data to include in Project 17-70's crash prediction model development.

As discussed above, the crash prediction models developed by Rodegerdts et al. (2007) are fully specified SPFs with only number of legs, number of lanes and entering AADT as explanatory variables for intersection-level SPFs thus assuming average conditions for any other variables. For approach level SPFs further geometric variables were considered. Therefore, the safety effects of the geometric and operational elements are incorporated into the SPF rather than quantified separately as CMFs. The models can be converted into a form where there is a base-SPF accompanied by CMFs similar to the current approach used in the HSM.

The intersection-level and approach-level crash prediction models from Rodegerdts et al. (2007) are presented below.

Intersection-Level Crash Prediction Models

The intersection-level crash prediction models follow the general form below:

$$Crashes/year = exp(Intercept)*AADT^{b_1}exp(X_1+\ldots..+X_n)$$

Where

AADT = average annual daily traffic entering the intersection

$X_1 \ldots X_n$ = independent variables other than AADT in the model equation

b_1 = calibration parameter

Table A2-3 presents the preferred intersection-level crash prediction models for total crashes; Table A2-4 presents the preferred intersection-level crash prediction models for injury crashes.

Table A2-3. Rodegerdts et al. (2007) Intersection-Level Safety Prediction Model for Total Crashes

Number of Circulating Lanes	Safety Performance Functions [Validity Ranges]		
	3 Legs	4 Legs	5 legs
1	$0.0011(AADT)^{0.7490}$ [4,000 to 31,000 AADT]	$0.0023(AADT)^{0.7490}$ [4,000 to 37,000 AADT]	$0.0049(AADT)^{0.7490}$ [4,000 to 18,000 AADT]
2	$0.0018(AADT)^{0.7490}$ [3,000 to 20,000 AADT]	$0.0038(AADT)^{0.7490}$ [2,000 to 35,000 AADT]	$0.0073(AADT)^{0.7490}$ [2,000 to 52,000 AADT]
3 or 4	Not in Dataset	$0.0126(AADT)^{0.7490}$ [25,000 to 59,000 AADT]	Not in Dataset
Dispersion factor, k=0.8986			

Table A2-4. Rodegerdts et al. (2007) Intersection-Level Safety Prediction Model for Injury Crashes

Number of Circulating Lanes	Safety Performance Functions [Validity Ranges]		
	3 Legs	4 Legs	5 legs
1 or 2	$0.0008(AADT)^{0.5923}$ [3,000 to 31,000 AADT]	$0.0013(AADT)^{0.5923}$ [2,000 to 37,000 AADT]	$0.0029(AADT)^{0.5923}$ [2,000 to 52,000 AADT]
3 or 4	Not in Dataset	$0.0119(AADT)^{0.5923}$ [25,000 to 59,000 AADT]	Not in Dataset
Dispersion factor, k=0.9459			

Approach –Level Crash Prediction Models

The approach-level crash prediction models follow the general form below:

$$Crashes/year = exp(Intercept)*AADT_1^{b1}\ldots AADT_m^{bm}*exp(c_1X_1+\ldots+c_nX_n)$$

Where,

$AADT_1\ldots AADT_m$ = average annual daily traffic

$X_1\ldots X_n$ = independent variables other than AADT in the model equation

$b_1\ldots b_m$, $c_1\ldots c_n$ = calibration parameters

The approach level models were developed for specific crash types: entering-circulating, exiting-circulating, and approaching. The models predict total crashes only due to the relatively small number of crashes in the data set. Table A2-5 presents the preferred crash prediction models per crash type.

Table A2-5. Rodegerdts et al. (2007) Crash Prediction Models by Crash Type

Preferred Model Characteristics	Entering-Circulating Crashes	Exiting-Circulating Crashes	Approach Crashes
Dispersion	1.080	2.769	1.289
Intercept	-7.2158	-11.6805	-5.1527
Entering AADT	0.7018	0.2801	0.4613
Circulating AADT	0.1321	0.2530	n/a
Entry Width (ft)	0.0511	n/a	n/a
Angle to Next Leg (degrees)	-0.0276	n/a	n/a
Inscribed Circle Diameter (ft)	n/a	0.0222	n/a
Circulating Width (ft)	n/a	0.1107	n/a
Approach Half-Width (ft)	n/a	n/a	0.0301

Notes: Table A2-5 was developed from Rodegerdts et al. (2007) Tables 21, 22, and 23.
"n/a" Indicates not applicable – the attribute is not in preferred model for that specific crash type; attribute is included in preferred model for one of the other crash types modeled.

2.4.2 Crash Modification Factors Potentially Transferable to Project 17-70

As discussed in Section 2.3.1, much of the U.S.-based research that has developed roundabout related CMFs has focused on the effectiveness of converting stop and signal controlled intersections to roundabouts (e.g., Persaud et al., 2001; Eisenman et al., 2004). The Kansas Roundabout Guide (2014) summarized the most recent CMFs related to intersection conversions to roundabouts; these are summarized in Table A2-6.

Table A2-6. CMFs for Conversion of Stop-Control and Signalized Intersections to a Roundabout

Treatment	Setting	Crash Type		Source
		All	Injury	
TWSC to single-lane roundabout	Rural	0.29	0.13	HSM 2010
	Suburban	0.22	0.22	HSM 2010
	Urban	0.61	0.22	HSM 2010
TWSC to two-lane roundabout	Suburban	0.81	0.32	HSM 2010
	Urban	0.88	-	HSM 2010
TWSC to single or two-lane roundabout	Suburban	0.68	0.29	HSM 2010
	Urban	0.71	0.19	HSM 2010
	All	0.56	0.18	HSM 2010
AWSC to single or two-lane roundabout	All	1.03	-	HSM 2010
Signal to single-lane roundabout	All	0.74	0.45	Gross et al.
Signal to two-lane roundabout	Suburban	0.33	-	HSM 2010
	All	0.81	0.29	Gross et al.
Signal to single or two-lane roundabout	Suburban	0.58	0.26	Gross et al.
	Urban	0.99	0.4	HSM 2010
	Urban	1.15	0.45	Gross et al.
	3-approach	1.07	0.37	Gross et al.
	4-approach	0.76	0.34	Gross et al.
	All	0.52	0.22	HSM 2010
	All	0.79	0.34	Gross et al.

Notes: TWSC = Two-way-stop-control; AWSC = All-way-stop-control

It may be feasible to use some of the CMFs from the effectiveness evaluations to create CMFs that indicate the degree to which crashes change relative to single to multilane roundabouts or roundabouts with three to four approaches. However, with the more robust analysis by Rodegerdts et al. (2007) such evaluation may not be as valuable to Project 17-70.

Other sources of potential CMFs applicable to Project 17-70 are to consider using the fully specified SPFs from Rodegerdts et al. (2007) as well as potentially Bagdade et al. (2011). Table A2-7 is an example of how the fully specified SPFs from Rodegerdts et al. (2007) could be converted to CMFs that would be accompanied by a base-SPF (i.e., consistent with current HSM crash prediction modeling approach). In the table the variable coefficients have been converted to a CMF as $\exp^{(\text{coefficient})}$ reflecting the log-linear model form. For each unit change in the variable the CMF is applied.

Table A2-7. CMFs Implied in Rodegerdts et al. (2007) Approach-Level Models for Unit Change in Variable

Variables from Preferred Models	Entering-Circulating Crashes	Exiting-Circulating Crashes	Approach Crashes
Entry Width (ft)	1.0524	n/a	n/a
Angle to Next Leg (degrees)	0.9728	n/a	n/a
Inscribed Circle Diameter (ft)	n/a	1.0224	n/a
Circulating Width (ft)	n/a	1.1171	n/a
Approach Half-Width (ft)	n/a	n/a	1.0306

Notes: Table A2-7 was developed from Rodegerdts et al. (2007) Table 24 CMFs implied from candidate approach-level models for unit change in variable

"n/a" Indicates not applicable – the attribute is not in preferred model for that specific crash type; attribute is included in preferred model for one of the other crash types modeled.

2.5 SUMMARY OF KEY FINDINGS

The summary of key findings is organized into the three core topic areas:

1. Crash modeling techniques and approaches used in similar crash prediction research that may be applicable to Project 17-70;
2. Roundabout geometric and operational features that previous research projects and studies have found to influence crashes frequency and severity; and
3. Safety Performance Functions (SPFs) and Crash Modification Factors or Functions (CMFs) potentially applicable to Project 17-70 that were developed in previous research.

The findings presented below and supporting background material informed the Task 2 Data Collection Plan development and activities in Project 17-70.

2.5.1 Crash Modeling Techniques and Approaches

The SPFs and CMFs modeling approaches that have been applied in similar research efforts are listed below.

Safety Performance Functions (SPFs)

Negative binomial regression model - As the most frequently used method to develop the SPFs, negative binomial regression model is probably the first choice in Project 17-70. The negative binomial regression model is an appropriate model for rare events (such as crashes) which show overdispersion. The negative binomial model also provides the estimated overdispersion parameter that is required to apply the EB method in the HSM. Moreover, the extensive experience of negative binomial regression modeling from previous safety studies will facilitate the model development in this project.

Crash Modification Factors (CMFs)

- **Before-After with Empirical Bayes** - Before-after EB methods are preferred to cross-sectional if before-after data are available and the sample size is big enough. Compared with other methods, the before-after EB method is a statically robust method, which can effectively account for the regression-to-the-mean, traffic volume changes over time and non-treatment related time trends. Because roundabouts are fairly new in the U.S. and any changes made are rarely done in isolation the opportunity to conduct before-after studies is probably limited.
- **Cross-Sectional** -When there are not enough before-after data for the before-after EB method, the cross-sectional method will be applied. The cross-sectional method has been proven as a feasible statistical method to develop CMF and CMF functions when very limited before-after data are available. Results should be confirmed with limited before-after data if possible.

Chapter 5 discusses alternative statistical modeling approaches in more detail.

2.5.2 Roundabout Features Influencing Crash Frequency and Severity

The roundabout geometric and operational features consistently found to influence crash frequency and severity are listed below.

- Volume Characteristics
    - Vehicle AADT (disaggregated by entering, circulating and exiting movements provides additional value); and
    - Pedestrian, bicycle, and motorcycle volume data particularly useful for predicting crashes by mode).
- Basic Configuration Characteristics
    - Number of approaching or entering lanes; and
    - Number of circulating lanes.
- Geometric Characteristics:
    - Entry width;
    - Central island diameter;
    - Angle between approach legs;
    - Inscribed circle diameter;

    - Circulating width; and
    - Approach lane width.
- Speed Related Characteristics:
    - Entering, exiting and circulating vehicle speed (e.g., free mean speed, 85th percentile speed); and
    - Variation in vehicle speed (e.g., between vehicles, for one vehicle as traveling through the roundabout).
- Other Characteristics
    - Sight distance (from perspective of approaching vehicle).

2.5.3 Crash Modification Factors and Safety Performance Functions Potentially Applicable to Project 17-70

Rodegerdts et al. (2007) provides the most robust U.S.-based roundabout crash prediction models in the form of fully specified SPFs that Project 17-70 may be able to build from to improve their predictive power and applicability to practitioners. Other recent U.S.-based research related to roundabout safety performance by Bagdade et al. (2011), Dixon and Zheng (2013), Qin et al. (2013) and Srinivasan et al. (2011) are valuable sources of data and findings. International research discussed above, is valuable in identifying additional roundabout geometric and operational characteristics that have been found to be related to safety performance in other studies.

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Chapter 3: Outreach to Public Agencies with Roundabouts

3.1 INTRODUCTION AND PURPOSE

As part of the literature review activities, the research team reached out to 17 public agencies with roundabouts. The purpose of these outreach activities were to identify existing databases and data sources that could be used in Project 17-70, potential focus sites for roundabouts that had been modified in the past, information that could be used to understand or evaluate driver learning curve, and general information related to each agency's experiences with roundabouts that could be informative to Project 17-70. The following subsections present the agencies contacted and a summary of key findings.

3.2 AGENCIES CONTACTED

The public agencies that were contacted focused on U.S. state department of transportations (DOTs) and local agencies with roundabouts. In total, the research team reached out to 17 agencies; three cities and 14 state DOTs. Table A3-1 summarizes the agencies contacted.

Table A3-1. Summary of Agencies Contacted

Agency	Estimated # of Roundabouts	Maintain an Inventory?	Received Response
Georgia DOT	99	Yes	Yes
Wisconsin DOT	266	Yes	Yes
Florida DOT	258	Yes	Yes
Pennsylvania DOT	20	Yes	Yes
Alaska DOT & PF	14	Do not have one.	Yes
Colorado DOT	154	-	No
Kansas DOT	103	Yes	Yes
Washington State DOT	212	Yes	Yes
Maryland SHA	92	-	No
Caltrans	166	Does not have one.	Yes
Iowa DOT	36	Yes	Yes
Arizona DOT	58	Do not have one.	Yes
City of Carmel, IN	55	Yes	Yes
City of Bend, OR	28	-	No
New York State DOT	110	Yes	Yes
City of Columbia, MO	30	Yes	Yes
North Carolina DOT	223	Yes	Yes

Notes:

"-" Indicates to be determined upon discussion with agency.

"Yes" Indicates research team has interviewed one or more agency representatives and obtained (or will soon obtain) available roundabout related information from the agency.

"No" Indicates research team has attempted to contact one or more persons at agency via email and phone and has not received a response.

The research team spoke with 14 of the 17 public agencies and obtained the inventory information they had available regarding roundabouts within their jurisdiction.

3.3 EXISTING AGENCY DATABASES

Eleven of the agencies maintain an inventory of roundabouts within their jurisdiction. The inventories tend to include roundabout location (i.e., intersecting streets), opening year of roundabout, and type of roundabout (i.e., single-lane or multilane roundabout). Additional information such as traffic volume and crash data are available upon request but not necessarily maintained within the inventories. Most state DOTs are confident in their inventories of roundabouts on their state system and attempt to include roundabouts on local roads to the best of their knowledge and awareness. Chapter 4 presents a summary of the data attributes contained in the state roundabout inventories.

3.4 MODIFIED ROUNDABOUTS

As part of the agency outreach, the research team asked agency representatives to identify the roundabouts that had been built within their jurisdiction that they knew had been modified since their initial construction. The purpose of this was to help identify roundabouts the research team may be able to consider for before/after analysis within Project 17-70. The research team is aware the following roundabouts have undergone modifications since there initial construction.

SR 60/Coronado Drive/Madalay Avenue – Clearwater, FL: The roundabout opened in December 1999, replacing a one-way couplet system with signalized intersections. Within its first year of operation, the roundabout experienced a very high frequency of exit-circulating property damage crashes, particularly at the exits to Coronado Drive and to SR 60. Among many modifications made to the roundabout, geometric modifications were made to the entries and exits to reduce the area of conflict. In addition, striping changes were made to mark the Coronado Drive and SR 60 exits as two-lane exits rather than as a concentric circulatory stripe that requires a lane change to exit. For reasons unknown to the research team at this time, the striping on the circulatory roadway was reverted back to a concentric stripe at the Coronado Drive exit, sometime between 2004 and 2006 according to the historical photos available on Google Earth.

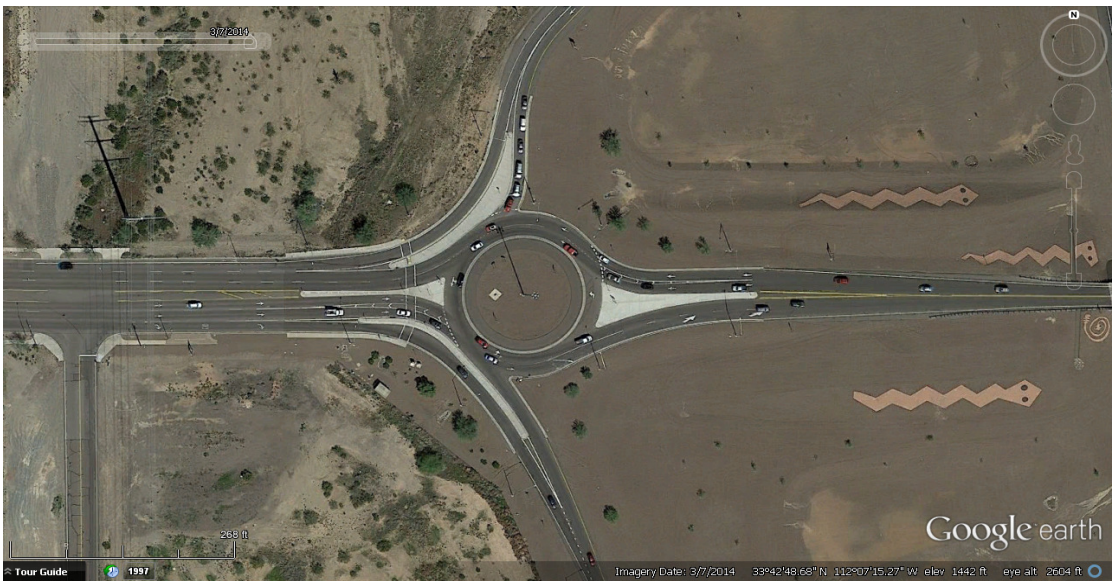
I-17 NB and SB Ramps/Happy Valley Road (two roundabouts) – Phoenix, AZ: These two roundabouts (one per ramp terminal intersection) were constructed as multilane roundabouts and open to traffic sometime before or during 2002. The multilane roundabouts were a combination of single-lane and multilane entries with right-turn by-pass lanes. The initial design included five approach legs two of which were relatively closely spaced. The closely spaced approaches were access either to or from I-17 and an adjacent frontage road. Figure A3-1 illustrates the roundabout configuration for the I-17 SB Ramps/Happy Valley Road in 2002.

Figure A3-1. Initial I-17 SB Ramps/Happy Valley Road Roundabout (2002)

Source: Google Earth 2014

Modifications were made to the roundabouts around 2006 to implement spiral striping in the circulatory road narrowing it for the single-lane portions of the roundabout. Pavement markings were also modified on the multilane approaches to mitigate path overlap. Around 2009, the frontage road access at both roundabouts was removed simplifying both to four approaches. The splitter islands were also reconstructed to eliminate the need for the spiral striping in the circulatory roadway. Figure A3-2 illustrates the reconfiguration in 2014.

Figure A3-2. Modified I-17 SB Ramps/Happy Valley Road Roundabout (2014)



Source: Google Earth 2014

York Road / Dulaney Valley Road / Joppa Road - Towson, MD: Modifications were made at three points in the life of this roundabout. The roundabout has five approaches and is oval in shape. Figure A3-3 illustrates the initial roundabout form.

Figure A3-3. Initial York Road/Dulaney Valley Road/Joppa Road Roundabout (1996)

Photo Source: Kittelson & Associates, Inc.

The circulatory roadway was initially constructed without circulatory roadway markings (consistent with the practice at that time in 1995/1996); the first modification was to add pavement markings within the circulatory roadway. After the initial modification, lane use control signs were added on several approaches. During the third iteration, channelization was added in the circulatory roadway by extending several of the splitter islands. The purpose of this was to reduce portions of the circulating roadway from two lanes to one lane. Figure A3-4 illustrates the modifications to the roundabout.

Figure A3-4. Modifications to York Road/Dulaney Valley Road/Joppa Road Roundabout

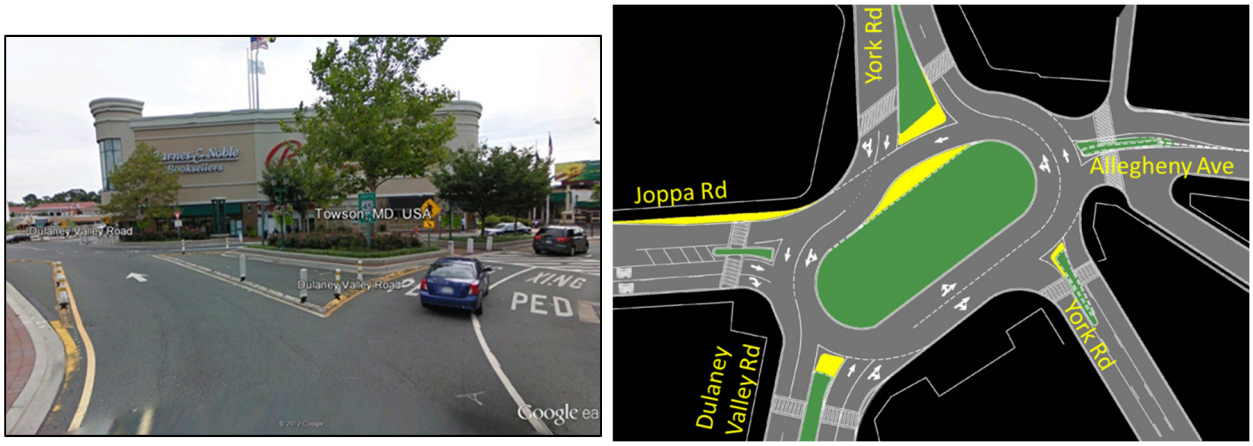


Image Source: Google Earth (left) and Kittelson & Associates, Inc. (right)

US 1/34th Street/Perry Street – Mount Rainier, MD: The circulatory roadway was redesigned to improve entry geometry. Several portions of the circulatory roadway (opposite the one lane entries) were reduced to one lane. In addition, street lighting was added. The lights were designed initially but not installed. They were added later after crash data indicated an unexpectedly high frequency of nighttime crashes.

MD 175 / Odenton Rd / Sappington Station Rd– Fort Meade, MD: Initially constructed in 2002 as a multilane roundabout with a combination of single- and two-lane entries, exits and circulating lanes. The intersection is skewed, includes five approaches, and the two-lane entries initially exhibited path overlap. Figure A3-5 illustrates the initial roundabout configuration.

Figure A3-5. Initial MD 175/Odenton Road/Sappington Station Road Roundabout (2002)

Image Source: Google Earth

Modifications were made to address path overlap and resign and restripe the roundabout to single-lane entries, exits and circulating lane. Single-lane provides sufficient capacity for the near-term; modifications made do not preclude the roundabout to being returned to a multilane roundabout if/when future traffic volumes increase. Figure A3-6 illustrates the roundabout after the above modifications were made.

Figure A3-6. Modifications to MD 175/Odenton Road/Sappington Station Road Roundabout

Image Source: BingMaps © 2013

US 15/US 50 – Gilberts Corner, VA: The roundabout was originally constructed as a multilane roundabout with a combination of single- and two-lane entries, exits and circulating lanes. Due to the frequency of crashes, Virginia DOT hired a consultant to evaluate alternative geometric improvements. The consultant and Virginia DOT decided it was feasible from a capacity perspective to convert the multilane roundabout to a pure single-lane roundabout. Modifications included redesigning splitter islands to improve speed control on approaches and signing and pavement marking changes to provide clearer guidance to drivers and improve operations. These modifications were installed in December 2013.

Based on conversations with the agencies identified above, the research team has also been made aware of the locations listed below.

Dowling/Seward Highway Ramp Terminal Intersections (two roundabouts) – Alaska: Speed tables were added to the approaches for both roundabouts to help slow vehicle approach and entry speeds. Geometric changes to the approaches were ruled out due to site constraints.

US 34/US 63 – Iowa: Three-legged roundabout at which heavy vehicles were not initially using the truck apron and as a result tracking too wide creating rutting issues. Delineators along the outside of roundabout were installed as well as signs to encourage heavy trucks to use the truck apron.

Two Roundabouts – Wisconsin: Two roundabouts were modified. The research team is waiting for clarification from Wisconsin DOT regarding the specific intersection locations. One due to a higher crash frequency; signing and pavement markings were modified at the intersection. The second was modified due to heavy vehicles bottoming out; the curb at the truck apron was modified to try to prevent heavy vehicles from bottoming out when using the truck apron.

I-90/SR 202 Ramp Terminal Intersections (two roundabouts) – Washington State: Initially constructed as multilane roundabouts. The intersections had more capacity than needed and higher frequency of PDO crashes. Therefore, Washington DOT revised the signing and pavement markings to so that the roundabouts would function as single-lane roundabouts until additional capacity is needed. Revisions to the signing and pavement markings appear to have helped reduce crashes and improve operations.

SR 117 in Lebanon County – Pennsylvania: Initially built without curbs and had issues with heavy vehicle tracking and rutting. Plan to install curbs as well as a separated multiuse path around the roundabout.

Hazel Dell Parkway/126th and Hazell Dell Parkway/Main Street – Carmel, Indiana: Modified in 2009 after opening 10 to 15 years prior. The City modified the approaches to reduce entry speeds and path overlap. They accomplished this through adding horizontal deflection on the approaches and then altering the entry angle (see aerials on this website for details: http://gis.carmel.in.gov/Zoning/index.html). They also added pavement markings indicating lane assignments and path assignments through the roundabout. They have noticed a decrease

in crashes since the modifications. The City has also added turn lanes to these two roundabouts since they initially opened.

Hillsborough/Pullen Rd-Oberlin Rd – North Carolina: Removed second through-lane designation on three of the approaches (converting them to right-turn only lanes on two approaches and reducing one approach to a single-lane approach), which converted the roundabout to a single circulating lane with right-turn slip lanes. They also added buttons to help keep people out of hatched areas and in the proper lane. This has reduced crashes in the following years.

3.5 DRIVER LEARNING CURVE

Each of the agencies the research team talked with indicated they generally have observed or anecdotally knew that when roundabouts are initially built, especially in areas without a roundabout, there is typically a period of time where driver's tend to make more mistakes and/or are more hesitant in decision-making at roundabouts. None of the agencies the research team talked to had done a formal study regarding driver learning curve.

Recognizing that one of the objectives of Project 17-70 is to learn more about driver learning curve at roundabouts, the research team searched the general literature for studies completed regarding this subject and specific to U.S. drivers. The research team found two relevant studies. One study, Hanscom (2010), focused on driver understanding of signing and pavement markings at multilane roundabouts related to lane use. The second study, Joerger (2007), focused on driver learning curve at the first multilane roundabout constructed in Springfield, Oregon.

3.5.1 Hanscom (2010) – Pavement Markings and Guide Signs at Multilane Roundabouts

Hanscom (2010) documents the approach and findings from a study that investigated drivers' understanding of fishhook pavement markings and curved-stem guide sign arrows at multilane roundabouts. Twenty-eight combinations of guide signs and pavement markings were tested in a laboratory setting using a study methodology used by Federal Highway Administration (FHWA) to determine the design of diagrammatic guide signs. The study had 117 participants. Specific to multilane roundabout pavement markings and guide signs, the study evaluated:

- Pavement Markings
    - Fishhook;
    - Turn and through-lane arrows, both entry lanes;
    - Left-turn only arrow, left lane; right-turn, and through lane arrows, right lane;
    - Through lane use arrow, left lane; turn and through-lane arrows, right lane; and
    - No pavement markings.
- Guide Signs
    - Conventional arrows; and
    - Curved stem arrows.

The study found the combination of the curved-stem advance sign and the fishhook pavement marking was associated with the highest percentage of correct lane choice from study participants. The study also found that conventional advanced signing for typical roundabout situations works well; therefore, concluded the curved stem arrow advance sign should be reserved for situations in which drivers' may need more direction for correct left-turn path guidance.

3.5.2 Joerger (2007) – Driver Learning Curve at Multilane Roundabout in Springfield, OR

Joerger (2007) evaluated driver learning curve at the City of Springfield, OR's first multilane roundabout. The roundabout was opened in 2006. Joerger (2007) evaluated driver behavior at the roundabout over the first six months of its operation. The multilane roundabout has five approaches; two of the approaches have right-turn yield-controlled by-pass lanes. Four of the five approaches also have two entry lanes into the circulating roadway.

Upon initial opening of the roundabout, Joerger (2007) observed two entries to the roundabout at which drivers were not choosing the correct lane resulting in behavior such as lane changes within the circulatory roadway or abrupt lane changes at the roundabout entry. Eight, 24-hour data observations were conducted over the first six months.

One entry had a through/left-turn lane and a through/right-turn lane. Joerger (2007) observed drivers making an abrupt lane change in the circulatory roadway from the through/right-turn lane to the inside lane to be able to stay in the roundabout (to presumably take the third exit). Initially, 40% of drivers observed made the abrupt lane change noted above. After six months, 3.3% of drivers observed made the abrupt lane change maneuver described above.

The other entry had a through/left-turn lane adjacent to a through-lane, which was separated from a right-turn bypass lane. Joerger (2007) observed drivers making a late lane change from the through/left-turn lane to the through lane on approach to the roundabout. Initially, following opening of the roundabout approximately 20% of drivers observed made the late lane change. After six months, 5% of drivers observed made the late lane change.

For both roundabout entries, Joerger (2007) fit the six-month period of observations to a logarithmic "learning curve".

3.6 OTHER INFORMATION

In addition to gathering information from agencies regarding existing available roundabout data, roundabouts that have been modified, and driver learning curve, the research team also discussed with agencies the challenges in roundabout design and implementation they encounter, their efforts related to driver education about roundabouts, coordination and education they undertake with law enforcement, and crash reporting practices.

3.6.1 Challenges Encountered in Roundabout Design and Implementation

The research team used the information about challenges encountered in roundabout design and implementation to help inform the selection of candidate roundabout configurations. This information is incorporated into Chapter 4, specifically Section 4.2.2. Clear areas of interest and challenges agencies encounter at roundabout include signing and pavement markings at multilane roundabouts, high-speed approaches (i.e., 45 mph or higher posted speed limits on one or more approaches), design of multilane roundabout entries, addressing the perception that roundabouts are too small for heavy vehicles, and design of compact urban roundabouts.

3.6.2 Driver Education

Most of the agencies the research team spoke with have roundabout education materials available to distribute to communities on a project-by-project basis. When a roundabout is proposed as potential project for an intersection and/or is moving through the project development process, agencies will distribute roundabout education materials such as pamphlets and brochures. Similarly, most agencies contacted had a half to two page discussion in the state's driver manual about how to drive through a roundabout.

Several agencies also reach beyond the project –based driver education. The City of Bend, Oregon created a driver education program in spring and summer of 2006 in anticipation of constructing and opening some of their first multilane roundabouts. Their education program included brochures focused by mode, website content, 2D-hands-on roundabout model, informational video, a series of large display boards, and a children's activity book. City staff used these materials in a variety of venues – presentation and interactions with students and parents at local schools, conversations and presentations to advocacy groups, presentations to local neighborhood groups, public service announcements through local television broadcasts, informational booths at community events, and other similar opportunities.

Washington State DOT has robust roundabout driver information on their website and has distributed a training video to driving schools and department of licensing offices. Wisconsin DOT has also done quite bit of general driver education specific to roundabouts including newspaper ads, on-going television informational campaign, DVDs distributed to government educational organizations, material in the driver's manual, and approximately four questions in the question bank for the state driver's license test.

3.6.3 Coordination with Law Enforcement and Crash Reporting Practices

Based on the agencies the research team reached, there is minimal coordination and education to law enforcement officials regarding how to enforce traffic rules at roundabouts and/or how to record crashes at roundabouts. Georgia DOT has offered to speak with law enforcement agencies; only a few have accepted the offer. Washington State DOT has coordinated through management levels regarding traffic enforcement and crash reporting specific to multilane roundabouts. This combined with enabling and encouraging local agencies with quite a few roundabouts, such as the City of Lacey, Washington, to communicate to state and other local law enforcement about their lessons learned and best practices specific to roundabouts has also

been helpful. This enables law enforcement agencies to help and share information among each other. Wisconsin DOT meets with local law enforcement and emergency responders on a project specific basis.

Each of the agencies the research team spoke with report crashes at roundabouts as they would at other intersections. They use the same crash types and error codes as they would at other intersection forms. Wisconsin DOT, Washington State DOT, and Iowa DOT have plans to revise and update their crash reporting forms within the next year or two. There is interest in revising the form to include roundabout-specific crash codes. Chapter 5 discusses roundabout-specific crash types and codes and a proposed approach from Project 17-70.

3.7 SUMMARY OF KEY FINDINGS

The following briefly highlights the key findings from the topic areas discussed above.

Existing Databases or Inventories: There are approximately eleven agencies shared roundabout inventory and other data they have available for use in Project 17-70.

Modified Roundabouts: There are a few roundabouts that have been modified since their initial construction. The known sample is too small to be able to definitively quantify the safety effects of those modifications; however, those specific roundabouts may be able to provide qualitative lessons-learned that practitioners could apply to roundabout design and implementation.

Driver Learning Curve: Driver learning curve at roundabouts in the U.S. has not been thoroughly researched to-date. Initial findings indicate that driver error or hesitancy at multilane roundabouts dissipates relatively quickly over the first six months of operation.

Challenges Designing and Implementing Roundabouts: Some of the common challenges agencies encounter specific to roundabouts includes multilane roundabout design; signing and pavement markings; perception of heavy vehicles at roundabouts; high speed approaches; and design of compact urban roundabouts.

Driver Education: Driver education across agencies include project-based informational campaigns, information in driver manuals, and an increasing number of agencies with larger general public information campaigns regarding how to drive and use a roundabout.

Coordination with Law Enforcement and Crash Reporting: Coordination activities with law enforcement agencies are relatively minimal with some focus on project-based outreach. Crash reporting at roundabouts currently is completed as it would be at any other intersection form.

3.8 REFERENCES

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Chapter 4: Candidate Roundabout Configurations

4.1 INTRODUCTION AND PURPOSE

A primary goal of Project 17-70 is to help practitioners make informed roundabout design decisions based on the crash potential of alternative configurations. To accomplish this, the roundabout crash prediction models developed by Project 17-70 need to address common roundabout configurations, including those that present key challenges to practitioners. The ability of the research team to develop crash prediction models for the desired set of configurations is based on data of sufficient depth and breadth being available. The discussion below presents the range of potential configurations and attributes the research team considered. The discussion also identifies data the research team knows is available based on reviewing the literature and discussion with state and local agencies.

4.2 POTENTIAL ROUNDABOUT CONFIGURATIONS FOR PROJECT 17-70 CONSIDERATION

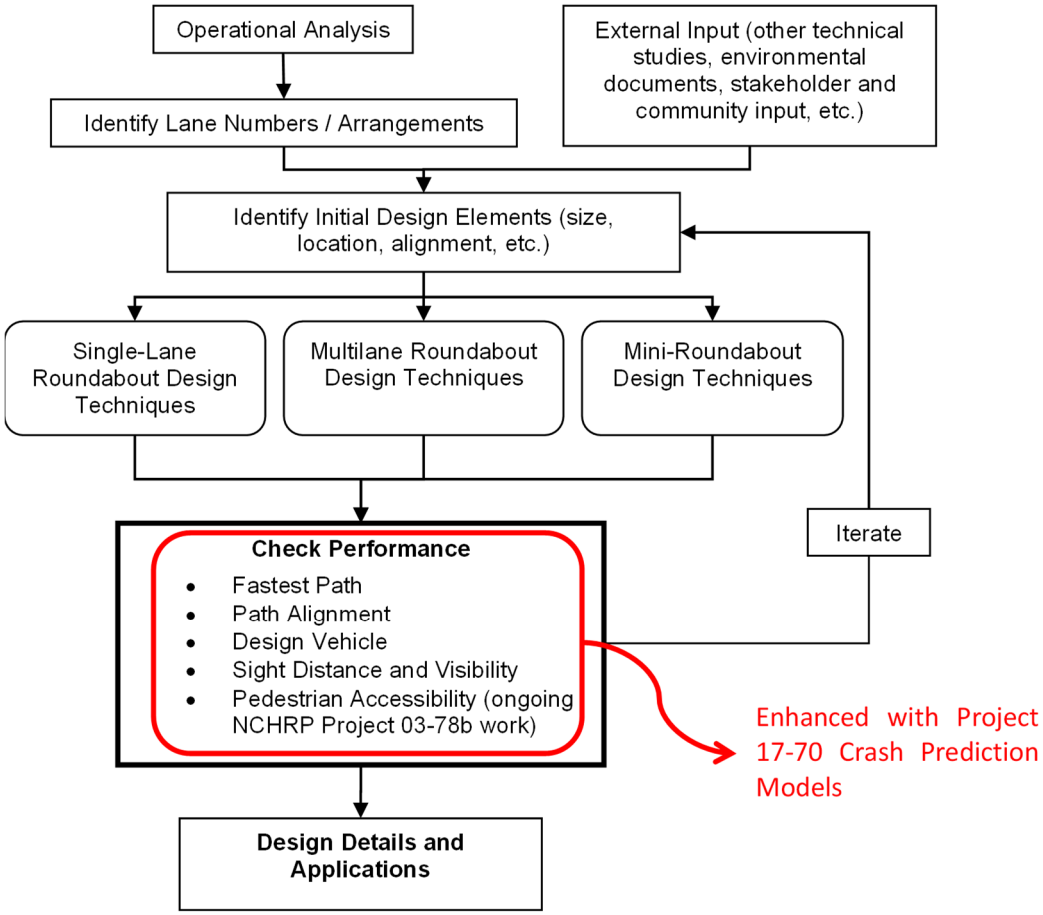
For Project 17-70 the research team identified roundabout configurations for intersection-level crash prediction models and approach level crash prediction models. Similar to Rodegerdts et al. (2007), the research team developed intersection-level crash prediction models for roundabouts for use in network screening activities and making planning level decisions about the type of traffic control desirable for an intersection. Project 17-70 also developed approach-level crash prediction models to inform design decisions practitioners encounter specific to roundabouts.

To identify potential roundabout configurations and attributes to consider within Project 17-70, the research team consulted a wide range of resources. The research team reviewed NCHRP *Report 672 Roundabouts: An Informational Guide, Second Edition*, which is the most current and national roundabout guidance for practitioners in the United States. The research team also gathered input from a wide range of local agencies regarding the design challenges they encounter specific to roundabouts. Finally, the research team also took into consideration the geometric and operational attributes identified in the critical review of the literature (previously discussed in Chapter 2). Findings from these activities are discussed in the subsections below.

4.2.1 NCHRP Report 672 Roundabouts: An Informational Guide, Second Edition

NCHRP Report 672, *Roundabouts: An Informational Guide, Second Edition* represents the current national guideline on roundabout design. The design process presented in Report 672 is a performance-based process, whereby a particular design is evaluated with respect to its operational performance, safety performance, accommodation of design users (automobiles, trucks, buses, bicycles, pedestrians, etc.), impacts to adjacent properties, and costs. This process is illustrated in Figure A4-1.

Figure A4-1. Roundabout Design Process



Source: Adapted from NCHRP Report 672 *Roundabouts: An Informational Guide, 2nd Edition* (Rodegerdts et al. 2010).

Figure A4-1 presents a simplified version of the design process, based on Exhibit 6-1 of NCHRP Report 672 *Roundabouts: An Informational Guide, Second Edition*. The process includes a step to check the performance characteristics of the roundabout, conducted after the initial design elements are selected but before the design details are established. Within these performance checks, the fastest path and path alignment checks are currently used as surrogates for safety performance. The crash prediction models from Project 17-70 could be used to augment and/or replace the fastest path and path alignment checks.

Some of the key design attributes discussed in NCHRP Report 672 that are believed to contribute most significantly to safety performance include the following:

- **Lane configuration**: Multilane roundabouts exhibit considerably higher crash frequencies than single-lane roundabouts because they accommodate higher traffic volumes and have a more complex configuration. The signing and pavement markings associated with establishing the appropriate lane configuration may be a contributing element.

- **Speed profiles:** The speed of vehicles on the approach, entry, circulatory roadway, and exit are all believed to contribute to safety performance. Higher speeds are believed to be associated with single-vehicle crashes and with injury crashes at the entry-circulating point. The speed profiles are influenced by a variety of geometric components, including entry width, entry angle, inscribed circle diameter, angle between legs, and others.
- **Alignment of vehicle:** For multilane roundabouts, the alignment of lanes is believed to contribute to side-swipe crashes. For accommodation of design vehicles at multilane roundabouts, a contributing factor to safety performance may include the decision on whether to allow trucks to straddle lanes or to keep them in separate lanes on entry and/or within the circulatory roadway.
- **Sight distance and visibility:** As with other types of intersections, stopping sight distance may be a contributing factor. For intersection sight distance, crash frequency is anticipated to be higher for cases where too little intersection sight distance is provided and where too much intersection sight distance is provided. The visibility of the intersection on approach may be a contributing factor for single-vehicle crashes, especially in environments with higher approach speeds. Nighttime illumination and signing may also be a contributing factor.
- **Location and design of pedestrian and bicycle facilities:** The presence/absence and configuration of pedestrian and bicycle facilities may be a contributing factor to pedestrian and bicycle crashes.

4.2.2 Outreach to Public Agencies

As discussed in Chapter 3, the research team reached out to 17 agencies at the state and local level to learn about their experiences with roundabouts and received feedback from 14 agencies. This effort included discussing challenges they have encountered in designing and implementing roundabouts in their jurisdiction. Based on these discussions, the research team identified the following as common challenges for which understanding the impact on crash frequency would be beneficial:

- Entry angle;
- Lane configurations at multilane roundabouts;
- Signing and striping used to communicate lane designations and vehicle paths at multilane roundabouts;
- Accommodating large trucks (i.e., when to provide a truck apron);
- High-speed approaches (i.e., roundabouts located on facilities with posted speed limits of 40 miles-per-hour or higher);
- Illumination;
- Compact urban roundabouts; and
- Central island landscaping with respect to fixed object crashes and sight distance.

4.2.3 Roundabout Features Influencing Crash Frequency and Severity

As presented in Chapter 2, trends in international research developing crash prediction models for roundabouts have identified various volume, basic roundabout configuration characteristics, specific geometric characteristics, speed related characteristics and other site related features that influence crashes. The summary of these collective features are below (repeated here from Chapter 2 for ease of reference).

- Volume Characteristics
  - Vehicle AADT (disaggregated by entering, circulating and exiting movements); and
  - Pedestrian, bicycle, and motorcycle volume data.
- Basic Configuration Characteristics
  - Number of approaching or entering lanes; and
  - Number of circulating lanes.
- Geometric Characteristics
  - Entry width;
  - Central island diameter;
  - Angle between approach legs;
  - Inscribed circle diameter;
  - Circulating roadway width; and
  - Approach lane width.
- Speed Related Characteristics:
  - Entering, exiting and circulating vehicle speed (e.g., free mean speed, 85th percentile speed); and
  - Variation in vehicle speed (e.g., between vehicles, for one vehicle through roundabout).
- Other Characteristics
  - Sight distance (from perspective of approaching vehicle).

4.2.4 Summary of Potential Roundabout Configurations

Collectively, the information above indicates a relatively long list of attributes for which it would be beneficial to practitioners to understand if those attributes consistently influence crash frequency and severity. And if the attributes do influence crashes, the degree to which they influence crash frequency and severity at roundabouts.

Based on the findings above, the intersection-level roundabout configurations the research team started to explore in Project 17-70 were:

- Urban/Suburban single-lane roundabouts (for posted speed limits <45 mph and ≥45 mph);
- Rural single-lane roundabouts (for posted speed limits <45 mph and ≥45 mph);

- Urban/Suburban multilane roundabouts (for posted speed limits <45 mph and ≥45 mph); and
- Rural multilane roundabouts (for posted speed limits <45 mph and ≥45 mph).

The approach-level roundabout configurations the research team started to explore in Project 17-70 were:

- Single-lane roundabout approaches (with and without right-turn bypass lanes).
- Multilane roundabouts with differing approaching lane configurations:
    - Two-lane entry vs. a single circulating lane;
    - Two-lane entry vs. two circulating lanes; and
    - Single-lane entry vs. two circulating lanes.

4.3 INFORMATION AVAILABLE IN ROUNDABOUT DATABASES

As part of the literature review effort, the research team compiled existing databases from using the information obtained through agency outreach activities and information from previously completed research projects (e.g., Rodegerdts et al., 2007). Table A4-1 summarizes the desired attributes/configurations and which databases contain information regarding those attributes.

Table A4-1. Summary of Database Attributes

Attributes	Databases									
	Modern Roundabout Website	Iowa DOT	Wisconsin DOT	Pennsylvania DOT	Georgia DOT	New York State DOT	NCHRP Report 572 (Rodegerdts et al., 2007)	NCHRP Report 705 (Srinivasan et al., 2011)	Florida DOT	North Carolina DOT
Number of Circulating Lanes	X[1]	X	X	X	X[1]	X[1]	X	X	X	X[1]
Inscribed Circle Diameter	X[2]	-	-	-	X[2]	-	X	X	-	-
Entry Angle	-	-	-	-	-	-	X	-	-	-
Number of Approaching Lanes	-	-	-	-	-	-	X	-	-	-
Number of Exiting Lanes					-	-	-	-	-	-
Truck Apron Presence	-	X	-	-	-	-	X	X	-	-
Illumination Presence	-	-	X	-	-	-	-	-	-	-
Vehicle AADT	-	X[2]	-	-	-	-	X[3]	X[5]	-	-
Bike/Ped/Motorcycle Volumes	-	-	-	-	-	-	-	-	-	-
Entry Width	-	-	-	-	-	-	X	-	-	-
Central Island Diameter	-	-	-	-	-	-	X	X	-	-
Angle between Approach Legs	-	-	-	-	-	-	X	-	-	-
Circulating Roadway Width	-	-	-	-	-	-	X	-	-	-
Lane Width	-	-	-	-	-	-	X	-	-	-
Entering Vehicle Speed	-	-	-	-	-	-	X	-	-	-
Variation in Vehicle Speed	-	-	-	-	-	-	X[4]	-	-	-
Entering Sight Distance	-	-	-	-	-	-	-	-	-	-
Bypass Lane	-	-	-	-	-	-	X	-	-	X
Land-use	-	-	-	-	-	-	X[6]	X[7]	-	-

Notes:
X = attribute information contained in database
- = attribute information is not contained in database
[1]Only denotes single vs. multi-lane
[2]Not available for all entries
[3]Includes total, major road, minor road, entering, exiting, and circulating AADT
[4]Includes differential between approach and entry; entry and circulating; and exit and circulating speeds
[5]Includes total, major road, and minor road AADT
[6]Includes urban, suburban, and rural locations
[7]Includes only urban and suburban locations

4.4 REFERENCES

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Chapter 5: Alternative and Initial Recommendation for Preferred Statistical Modeling Approaches

5.1 INTRODUCTION AND PURPOSE

The purpose of this chapter is to present the range of possible alternative crash prediction modeling approaches and identify the initial recommendations for the preferred statistical approaches to be used in Project 17-70. The Final Report for Project 17-70 documents the final modeling approach used.

5.2 EVOLUTION OF MODELING FOR THE HIGHWAY SAFETY MANUAL PREDICTIVE ALGORITHM

The modeling for the HSM predictive chapters began with the work of Vogt & Bared (1998) who estimated models for intersections and segments on two-lane rural roads and intersection models for certain intersection types on rural multilane roads. Negative binomial models were estimated for crash rates for road segments, assuming, as has since been shown to be invalid, that crash frequency is proportional traffic volume. Additional covariates based sometimes on specially collected data were included in the model when the direction of the implied effect was found to be in accord with intuition.

The Vogt and Bared models formed the basis of the predictive algorithm proposed by Harwood et al. (2000) that is currently prescribed in the Highway Safety Manual predictive chapters. In this, CMFs are applied to a base model prediction to adjust for conditions different from those assumed in the base model.

The base models for the two-lane rural highways chapter were derived by substituting base condition values in the multi covariate models estimated by Vogt and Bared. For example, for four-legged stop controlled intersection on two-lane rural roads, the multi covariate crash model is:

$$Crashes/year = exp(-9.34 + 0.60\ ln\ (Major\ Road\ ADT) + 0.61\ ln\ (Minor\ Road\ ADT) + 0.13(ND) - 0.0054\ SKEW$$

Where,

ND is the number of driveways within 76 m of the intersection on the major road; and SKEW is the intersection skew angle (= 0 for right angle intersections)

The base condition is no driveways, adequate sight distance, no turn lanes and no skew. For this condition, the base model is then derived as:

$$Crashes/year = exp(-9.34 + 0.60\ ln\ (Major\ Road\ ADT) + 0.61\ ln\ (Minor\ Road\ ADT)$$

Crash type models were not estimated. Rather the HSM approach specified crash type proportions that could be applied to the model predictions for total crashes.

Washington et al. (2005) proposed and used an alternative approach to developing base models in a validation of the Bared and Vogt models. In this they estimated some base models directly from subsets of data meeting base conditions.

The multilane rural roads chapter estimated both base models and multiple covariate models. The former, like Washington et al., were estimated directly from the subset if data that met base conditions, while the latter were also used for estimating a few CMFs. All models were estimated with a negative Binomial error structure, allowing the estimation of a dispersion parameter used for empirical Bayes prediction in the HSM. SPFs were directly estimated for total crashes of all types and severities as well as KAB only crashes. KABCO models were estimated from various crash types.

The urban and suburban arterials chapter in effect estimated models for average conditions of variables and specified these as base models to which CMFs externally determined. The approach estimated separate models for single and multiple vehicle crashes but specified crash type proportions to be applied to model predictions for total crashes to estimate crashes by type.

A safety prediction model for freeways was developed for the next edition of the HSM. The predictive models were based on multi covariate models. The covariates in the regression model were represented as CMFs. The base conditions were specified in the model when it was calibrated, with the values defined by the researchers based on typical conditions. Separate models were developed for single- and multiple-vehicle crashes. Crash type proportions are used with the predictive model to estimate the crash frequency for individual crash types. The researchers also developed a severity distribution function (SDF) to be used with the predictive model to estimate crash frequency for various severity categories. The SDF includes variables that define the influence of a change in the dimension of key geometric elements to a change in the proportion of crashes in each severity category.

5.3 CRASH PREDICTION MODELS FOR ROUNDABOUTS

For NCHRP Report 572, generalized linear modeling was used to estimate model coefficients assuming a negative binomial error distribution, consistent with the state of research in developing these models. Models were developed at the intersection level (using AADT, and number of lanes and legs) for total and injury crashes and at the approach level for approaching, entering-circulating and exiting-circulating crashes. For the latter, alternative models were estimated with different combinations of geometric variables. These models were used to imply some CMFs.

The report also reviewed models calibrated by others, finding in general that the approach to calibration of the better models was similar to that adopted in NCHRP Report 572.

5.4 MODELING APPROACHES AND ISSUES

Some content for the first two sub-sections is taken mainly from a recent series of papers that provide an excellent review of modeling approaches and issues (Lord and Mannering, 2010 and Savolainen et al., 2011). Some content is also based on a recent NCHRP 17-62 white paper that outlined the proposed methods for estimation of crash type and severity models under that project for the three predictive chapters currently in the HSM.

5.4.1 Crash Frequency Modeling

Lord and Mannering (2010) provide an excellent review of the advantages and disadvantages of methods for modeling crash frequency data. This is reproduced below as Figure A5-1.

Figure A5-1. Pros and cons of methods for modeling crash frequency data

Model type	Advantages	Disadvantages
Poisson	Most basic model; easy to estimate	Cannot handle over- and under-dispersion; negatively influenced by the low sample-mean and small sample size bias
Negative binomial/ Poisson-gamma	Easy to estimate can account for over-dispersion	Cannot handle under-dispersion; can be adversely influenced by the low sample-mean and small sample size bias
Poisson-lognormal	More flexible than the Poisson-gamma to handle over-dispersion	Cannot handle under-dispersion; can be adversely influenced by the low sample-mean and small sample size bias (less than the Poisson-gamma), cannot estimate a varying dispersion parameter
Zero-inflated Poisson and negative binomial	Handles datasets that have a large number of zero-crash observations	Can create theoretical inconsistencies; zero-inflated negative binomial can be adversely influenced by the low sample-mean and small sample size bias
Conway–Maxwell–Poisson	Can handle under- and over-dispersion or combination of both using a variable dispersion (scaling) parameter	Could be negatively influenced by the low sample-mean and small sample size bias; no multivariate extensions available to date
Gamma	Can handle under-dispersed data	Dual-state model with one state having a long-term mean equal to zero
Generalized estimating equation	Can handle temporal correlation	May need to determine or evaluate the type of temporal correlation a priori; results sensitive to missing values
Generalized additive	More flexible than the traditional generalized estimating equation models; allows non-linear variable interactions	Relatively complex to implement; may not be easily transferable to other datasets
Random-effects	Handles temporal and spatial correlation	May not be easily transferable to other datasets
Negative multinomial	Can account for over-dispersion and serial correlation; panel count data	Cannot handle under-dispersion; can be adversely influenced by the low sample-mean and small sample size bias
Random-parameters	More flexible than the traditional fixed parameter models in accounting for unobserved heterogeneity	Complex estimation process; may not be easily transferable to other datasets
Bivariate/multivariate	Can model different crash types simultaneously; more flexible functional form than the generalized estimating equation models (can use non-linear functions)	Complex estimation process; requires formulation of correlation matrix
Finite mixture/Markov switching	Can be used for analyzing sources of dispersion in the data	Complex estimation process; may not be easily transferable to other datasets
Duration	By considering the time between crashes (as opposed to crash frequency directly), allows for a very in-depth analysis of data and duration effects	Requires more detailed data than traditional crash-frequency models; time-varying explanatory variables are difficult to handle
Hierarchical/multilevel	Can handle temporal, spatial and other correlations among groups of observations	May not be easily transferable to other datasets; correlation results can be difficult to interpret
Neural network, Bayesian neural network, and support vector machine	Non-parametric approach does not require an assumption about distribution of data; flexible functional form; usually provides better statistical fit than traditional parametric models	Complex estimation process; may not be transferable to other datasets; work as black-boxes; may not have interpretable parameters

Source: Lord and Mannering, 2010

The authors also discuss what they term as formidable problems in terms of data characteristics, namely over-dispersion, under-dispersion, time-varying explanatory variables, low sample-means and size, crash-type correlation, under-reporting of crashes, omitted-variables bias, and issues related to functional form and fixed parameters). They identify some innovative

methodological approaches to resolve these issues, including random-parameter models, finite mixture models, Markov switching models among others.

5.4.2 Crash Severity Modeling

The approaches used for the HSM prediction methodology run the gamut from specifying severity proportions to be applied to model predictions for total crashes, to directly estimating SPFs for KAB and KABC crashes, to probability models used to estimate severity distribution functions in the freeways chapter. The "proportions' method is now regarded as obsolete, while the issues with the approach of estimating SPFs directly are the same as those for crash frequency models in general. The probabilistic approach has been preferred by the 17-62 team for re-estimating severity models for the existing HSM predictive chapters, so the review below will focus on that, with some content taken from a white paper developed by the NCHRP 17-62 project team.

The probabilistic approach estimates the probability of an injury of a given severity, given that a crash has occurred, as a function of roadway and traffic characteristics and, potentially, crash and person related variables. From these, severity distribution functions (SDFs) can be derived for estimating frequencies of crashes of given severities as a function of model variables. The NCHRP 17-45 project report (Bonneson et al., 2012) outlines the SDF estimation procedure in detail.

The probability models can be ordered or unordered. Ordered models reflect the ordinal nature of injury severity data, but some studies, according to the NCHRP 17-62 white paper, show that unobserved effects among adjacent injury categories can bias the parameter estimation (Savolainen and Mannering, 2007; Paleti et al., 2010; Savolainen et al, 2011). On the other hand, using unordered models to fit ordered data may result in a loss of efficiency (Amemiya, 1985). More details about limitations or these models are in Washington et al. (2011) and Savolainen et al. (2011).

Probability model formulations can be logit or probit. The error term of the Probit model is normally distributed across observations, while the error term of the Logit model is logistically distributed. Also, the link function of Probit and Logit models are different. In Probit models, the link function is the inverse normal cumulative distribution function, but in the Logit model, the link function is the Logit transform. However, there is usually no significant difference between the fitting results when the model uses either Logit or Probit functions (Chambers and Cox, 1967).

The partial proportional odds model (Mooradian et al., 2013) is a mixed version of ordered and multinomial logit. This preserves the ordered nature of the dependent variable but allows some covariates to have different slopes for each boundary.

Savolainen et al. (2011) provide an excellent review of the many variations of the probability models for injury severity, and discuss many of the data issues associated with this type of

modeling. The approaches reviewed listed below. The more popular ones (with 5 or more relevant publications) are identified in boldface.

- Artificial neural networks
- Bayesian hierarchical binomial logit
- Bayesian ordered probit
- **Binary logit and binary probit**
- Bivariate binary probit
- Bivariate ordered probit
- Classification and regression tree
- Generalized ordered logit
- Heterogeneous outcome model
- Heteroskedastic ordered logit/probit
- Log-linear model
- Markov switching multinomial logit
- Mixed generalized ordered logit
- Mixed joint binary logit-ordered logit
- **Multinomial logit**
- Multivariate probit
- **Nested logit**
- **Ordered logit and ordered probit**
- Partial proportional odds model
- **Random parameters (mixed) logit**
- Random parameters (mixed)
- ordered logit
- Random parameters ordered probit
- Sequential binary logit
- Sequential binary probit
- Sequential logit
- Simultaneous binary logit

Researchers for NCHRP Project 17-45 *Enhanced Safety Prediction Methodology and Analysis Tool for Freeways and Interchanges* (Bonneson at al., 2012) selected the standard multinomial logit (MNL) model as the basis for SDF development. Nested logit models were also developed to evaluate the Independence of Irrelevant Alternatives (IIA) limitation of the MNL. A test comparing the two models showed that the nested logit model is not different from the standard MNL model. A linear function was used to relate the crash severity with the geometric design features, traffic control features, and traffic characteristics.

5.4.3 Modeling for CMF Estimation

This section presents methods and statistical issues in CMF estimation, followed by a review of methods actually used or proposed for estimating CMFs for the HSM.

Methods and Statistical Issues

Gross et al. (2010) summarized the different methods available for CMF development, which is shown in Table A5-1.

Table A5-1. Summary of Methods for CMF Development

Method	Applications	Strengths	Weaknesses
Before-After with Comparison Group	Treatment is similar among treatment sites. Before and after data are available for both treated and untreated sites.	Simple Accounts for non-treatment related time trends and changes in traffic volume.	Difficult to account for regression-to-the-mean.
Before-After with empirical Bayes	Treatment is similar among treatment sites. Before and after data are available for both treated sites and an untreated reference group.	Account for : Regression-to-the-mean. Traffic volume changes over time. Non-treatment related time trends.	Relatively complex. Cannot consider spatial correlation.
Full Bayes	Useful for before-after or cross-section studies when: 1. There is a need to consider spatial correlation among sites. 2. Complex model forms are required. 3. Previous model estimates or CMF estimates are to be introduced in the modeling.	May require smaller sample sizes. Can include prior knowledge, spatial correlation, and complex model forms in the evaluation process.	Implementation requires a high degree of training.
Cross-Sectional	Useful when limited before-after data are available. Requires sufficient sites that are similar except for the treatment of interest.	Possible to develop crash modification functions. Allows estimation of CMFs when conversions are rare. Useful for predicting crashes.	CMFs may be inaccurate for a number of reasons: 1. Inappropriate functional form. 2. Omitted variable bias. 3. Correlation among variables.
Case-Control	Assess whether exposure to a potential treatment is disproportionately distributed between sites with and without the target crash.	Useful for studying rare events because the number of cases and controls is predetermined.	Can only investigate one outcome per sample.
	Indicates the likelihood of an actual treatment through the odds ratio.	Can investigate multiple treatments per sample.	Does not differentiate between locations with one crash or multiple crashes. Cannot demonstrate causality.

Source: Gross et al. (2010)

Table A5-1 continued. Summary of Methods for CMF Development

Method	Applications	Strengths	Weaknesses
Cohort	Used to estimate relative risk, which indicates the expected percent change in the probability of an outcome given a unit change in the treatment.	Useful for studying rare treatments because the sample is selected based on treatment status. Can demonstrate causality.	Only analyzes the time to the first crash. Large samples are often required.
Meta-Analysis	Combines knowledge on CMFs from multiple previous studies while considering the study quality in a systematic and quantitative way.	Can be used to develop CMFs when data are not available for recent installations and it is not feasible to install the strategy and collect data. Can combine knowledge from several jurisdictions and studies	Requires the identification of previous studies for a particular strategy. Requires a formal statistical process. All studies included should be similar in terms of data used, outcome measure, and study methodology.
Expert Panel	Expert panels are assembled to critically evaluate the findings of published and unpublished research. A CMF recommendation is made based on agreement among panel members.	Can be used to develop CMFs when data are not available for recent installations and it is not feasible to install the strategy and collect data. Can combine knowledge from several jurisdictions and studies. Does not require a formal statistical process.	Traditional expert panels do not systematically derive precision estimates of a CMF. Possible complications may arise from interactions and group dynamics. Possible forecasting bias.

Source: Gross et al. (2010)

There are several statistical issues associated with developing CMFs. Among them are:

- **Sample size** - The sample size may significantly impact the accuracy of CMF development. Generally, the standard error decreases as the sample size increases. Hauer (1997) provides a method for estimating required sample sizes assuming a comparison-group study is being conducted.
- **Potential Bias** - Besides the countermeasure of interest, the observed change in the safety performance may be due to other factors, such as regression-to-the-mean, traffic volume changes and secular changes in reported crash experience. Without accounting for the impacts of these factors, the CMFs derived from these data are usually unreliable. For cross-sectional studies the true relationship between a variable and crash risk may be misinterpreted by omitting important variables, correlation between explanatory variables and selecting an inappropriate functional form for the model.
- **Development of multiple CMFs** - Instead of assuming independence between individual CMFs, a CMF for a combination treatment should be developed directly from

an appropriate method (Gross et al. (2010)). Otherwise, if there is correlation between the effects of different treatments, the prediction based on the product of individual CMFs may be invalid.

- **Estimation context** - If the CMF was developed based on the data collected from a site with relatively high crash frequency, applying this CMF to other sites may result in incorrect prediction. For example, the CMF (0.764) for total crashes for the application of skid treatment was derived from the data collected at the road segments with a high frequency of wet weather crashes and low skid numbers. Therefore, Gross et al. (2010) suggested that it should not be expected that the same CMF will apply for resurfacing any road segment. In fact, Lyon and Persaud (2008) suggested that resurfacing can increase crashes at some locations.

CMF estimation for the HSM Chapters

CMFs have been estimated using three basic approaches, as were considered in NCHRP Project 17-29 (Prediction methodology for rural multilane roads). The review below is taken mainly from that project report (Lord et al., 2008).

The first approach is based on the before-after study framework. This method consists of estimating the safety effects of changes in geometric design features, traffic operations, or other characteristics by examining the increase or reduction in crash counts between the before and after periods. Three techniques have been proposed for this kind of study: 1) the simple or naive before-after study, 2) the before-after study with a control group, and 3) the before-after study using the EB method (Hauer 1997), which is considered state of the art. These techniques, including the limitations that apply mainly to the first two, have been well documented by others. A key resource is the FHWA Guidebook (Gross et al. 2010).

The second approach consists of estimating CMFs using the coefficients of multi covariate regression models. This method provides a simple way to estimate the effects of changes in geometric design features. However, although the variables are assumed to be independent, they may be correlated, which could affect the coefficients of the model, sometimes resulting in counterintuitive effects. This method was used as the basis for several CMFs in the Highway Safety Manual. Two CMFs for rural multilane roads were produced from the data used for NCHRP 17-29. NCHRP Report 572 also provides some CMFs estimated on this basis for approach level models. Table 8 in Chapter 2 presents the CMFs estimated in NCHRP Report 572.

The third approach estimates CMFs using baseline models and applying them to data that do not meet the baseline conditions. This method has been proposed by Washington et al. (2005) who thusly estimated some intersection related CMFs. For this method, the baseline model is first applied to sites not meeting all of the baseline conditions; then, the predicted and observed values per year are compared, and a linear relationship between these two values is estimated via a regression model to determine whether or not CMFs could be produced from its coefficients.

A variation of the second approach was applied for estimating some CMFs for the HSM freeways chapter (Bonneson et al, 2012). The researchers used a combination of previously estimated CMF's along with new CMFs estimated as part of the predictive SPFs, thereby taking advantage of existing road safety knowledge (contained in previously estimated CMF's) while also adding the information contained in the estimation data.

5.5 USING SURROGATES TO PREDICT CRASHES

Several researchers have attempted to analyze roundabout safety through surrogate measures. For the purpose of this review these efforts have been categorized as Speed based, Simulation-Conflict based or Other-Conflict based. The simulation-conflict approach has made use of the FHWA Surrogate Safety Assessment Model (SSAM) software to extract conflict data from the outputs of microsimulation software. Other-Conflict based approaches have used video data to extract a variety of conflict types.

For Project 17-70 the use of observed speeds, where available, or predicted speeds holds promise for exploring the use of speed as a surrogate in crash prediction models. While simulation and video based methods can be a powerful tool, the effort required for this type of approach is outside the scope of the project. Such methods however are promising avenues to pursue in future research. Key to their success will be the validation of accurately predicting conflicts and establishing the relationship between conflicts and real-world crashes.

5.5.1 Speed Based Prediction Approaches

Rodegerdts et al. (2007) documents an attempt to establish a speed-based approach-level safety SPF using measured mean vehicle speeds with the following structure:

$$Crashes / year = \exp(\text{intercept}) \cdot AADT^{b} \cdot \exp(cX)$$

Where,

AADT = average annual daily traffic

X = independent speed-related variable

Intercept, b, c = calibration parameters

The regression model was deemed inadequate on the basis of the weak effects of the speed variables so no speed-based SPF was recommended for use.

In addition to the attempt to model a speed-based SPF, the research documented in Rodegerdts et al. (2007) showed that SPFs for roundabouts typically do not include many geometric variables that would allow a designer to assess the safety implications of decisions in designing a roundabout. This is perhaps not surprising given that such functions are in fact difficult to estimate, given that roundabouts tend to have very few crashes and design features with little variation.

An approach for addressing this void was explored by Chen, Persaud and Lyon (2011). The premise behind this research is that if the safety performance of a roundabout can be related to vehicle speeds, and speed can be better estimated from traffic and geometric variables than crash experience can, then speed can be used indirectly as a surrogate in evaluating the safety implications of decisions in designing or re-designing a roundabout. In their work, four SPFs for predicting crashes were developed and compared based on the same sample of U.S. roundabouts used in Rodegerdts et al. (2007). The sample included data for the 33 individual approaches at 14 roundabouts which had measured average vehicle speeds. The four SPFs were a) approach level speed-based, b) approach level non speed-based, c) roundabout level speed-based, and d) roundabout level non speed-based. In developing the SPFs the average approach, entry, circulating and exiting speeds were considered. For the speed-based SPFs the best speed measure found for predicting crashes was the inside average speed (IAS) which averages the entry, upstream circulating and upstream exiting speeds for a given roundabout approach. For the roundabout level SPFs, the average of the IAS of each arm was used as the predictor speed variable. It was found that the SPFs including the IAS speed measures were superior to the SPFs lacking this variable in predicting crashes. The speed-based SPFs developed are shown below.

Speed-Based Approach Level Model

$Y = exp(-12.8046)(AADT)^{0.8075}exp(0.3388 \times IAS)$

Where,

Y = total crash frequency for specific approach per year

AADT = entering AADT on approach

IAS = average of the entry, upstream circulating and upstream exiting mean speeds in mph

Speed-Based Roundabout(i.e., Intersection) Level Model

$Y = exp(-15.0165)(AADT)^{1.0745}exp(0.3260 \times IASavg)$

Where,

Y = total crash frequency for entire roundabout

AADT = entering AADT for entire roundabout

IASavg = average of the IAS for each approach

The research further developed models for predicting roundabout speeds as a function of design features, with a view to using the speeds estimated from these models, along with the speed-based SPFs, to assess roundabout safety performance. With this approach, speed is in effect used as a surrogate safety measure. The model developed is:

$$IAS = (0.0253 \times ICD) + (0.1848 \times EW) + 9.51$$

Where,

ICD = inscribed circle diameter in feet

EW = entry width in feet

Building on the Chen, Persaud and Lyon (2011) work, Chen et al. (2013) developed geometric based models for predicting average vehicle speeds and then used these estimates to develop SPFs for roundabout approaches using a larger database of sites without in-field speed measurements. To develop the speed prediction models, the researchers used the same U.S. data as the previous study with additional data for roundabouts in Italy. The speed prediction model applied the IAS variable found in the earlier research to be the best predictor of roundabout crashes. The speed prediction model developed was:

$$IAS = 13.015958 - 3.088964 \times Cntry + 0.034074 \times Dav + 0.142936 \times Wav$$

Cntry = 1 for U.S. site, 0 for Italian site

Dav = average of the inscribed circle and central island diameters in feet

Wav = average of the entry, circulating and exiting width in feet

The speed prediction model was then applied to a larger database of 139 U.S. roundabout approaches to predict the IAS for each approach. The predicted speeds were then used to develop a speed-based SPF as shown below.

$$Crashes/year = exp(-16.3755)(AADT)^{0.5094}(IAS \times 4.3314)$$

The authors concluded that the developed SPF suggests that the predicted speed approach seems to be promising for indirectly estimating roundabout safety performance. This approach is conceptually more preferable than conventional observed speed based models for the advantage of expanding accessible sample size. The applicable sample size of U.S., for example, would be 33 if observed speed were applied to develop SPF. On the contrary, for predicted speed based modeling, sample size was 138.

Turner et al. (2009) developed a number of crash prediction models for urban roundabouts in New Zealand focusing on the relationship between crashes, speed, traffic volume and sight distance for various approach and circulating movements. Separate models were developed for the following crash types:

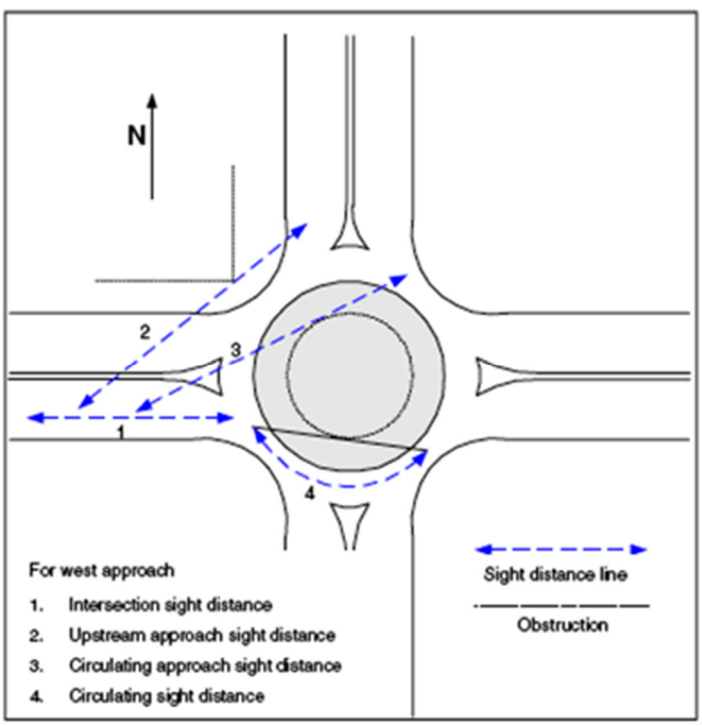
- Total crashes
- Types at urban roundabouts:
    - Entering vs circulating vehicles

- Rear-end
- Loss of control
- Other motor vehicles
- Pedestrian
- Entering vs circulating cyclist
- Other cyclist

Speed measures used were calculated from free-flowing vehicles. Variables used in the models included mean speed of vehicles on the circulating path, entering vehicle speed and the variation in entering vehicle speed. Results indicated that increased speeds and speed variance are associated with a high crash risk. Also developed were models predicting vehicle entering speed using sight distance from 10m behind the entry point to an upstream circulating vehicle and the inscribed circle diameter as explanatory variables. Entering speeds are associated with a larger diameter and larger visibility distance.

Zirekel et al. (2013) explored the relationship between crashes, operating speed and sight distance at low volume single-lane roundabouts in the United States. Sight distances were measured as shown in the Figure A5-2.

Figure A5-2. Sight Distance Measurements from Zirekel et al. (2013)



Data for 72 roundabout approaches were collected and split into two groups, one with a posted speed limit of 40 km/hr or greater and the other less than 40 km/hr. In general it was observed that once a required sight distance is met, larger sight distances were associated with a larger speed differentials between conflicting vehicles and also with total and rear-end entry crashes.

Crash prediction models using AADT and upstream approach sight distance explained more variation in crash counts than a model with AADT as the only explanatory variable. Given the observed relationship between sight distance and speed it is reasonable to assume that vehicle speeds would also have been a good predictor of crashes.

5.5.2 Simulation-Conflict Based Prediction Approach

In a paper submitted for the 2015 Transportation Research Board annual meeting (Saulino et al., 2014), the microsimulation software VISSIM combined with the Surrogate Safety Assessment Model (SSAM) software was used to estimate the number of peak hour conflicts for roundabout approaches. Conflicts defined by time-to-collision (1.5s) and post-encroachment time (5s) were considered. Conflict prediction models and approach level crash prediction models were calibrated suggesting that simulated conflicts can be applied for predicting crashes at roundabouts. The explanatory variables for the conflict models include peak hour vehicle volumes and either observed or predicted speeds with speed being the average of entering, exiting and circulating at the approach. The crash prediction models using the predicted number of conflicts as the explanatory variable proved to be a better predictor of multiple-vehicle crashes then models using AADT as the explanatory variable.

Vasconcelos et al. (2014) also used the SSAM software to calculate surrogate measures of safety using trajectory files generated by microscopic simulation for a four-leg single lane roundabout and a five-leg two lane roundabout. The time-to-collision was used for defining conflicts (less than 1.5 seconds being a conflict) and the relative speed at the time of minimum time-to-collision between vehicles as a proxy for crash severity. The generated conflicts at various levels of assumed AADT were compared to the predicted number of injury crashes from various available crash prediction models for the single lane roundabout. A satisfactory correlation between generated conflicts and predicted roundabout crashes was observed. At both the single and two lane roundabouts, the generated conflicts were compared to observed conflicts. It was found that the simulation predicts entering-circulating and weaving conflicts where they were observed in the field but did under predict the number of conflicts observed. However, if a larger value of time-to-collision had been used of course more conflicts would have been generated. In the field a conflict was recorded whenever a driver had to change their behavior as a cause of another vehicle's action.

Al-Ghandour et al. (2011), using the VISSIM simulation and SSAM software, analyzed conflict patterns at single-lane roundabouts with and without slip lanes for various scenarios of traffic movements. Three different slip lane exit control scenarios were also considered (yield, stop and free-flow merge). Conflict models were then developed using a Poisson log-linear regression model. Conflicts in various zones in the roundabout and slip lane were modeled as a function of approach entry, circulating and slip lane traffic flows and were found to be sensitive to the slip lane exit type. The addition of slip lanes was found to reduce overall conflicts. A time to collision threshold of less than 1.5 seconds was used to define conflicts. Predicted conflicts were used to predict crashes using the relationship from (FHWA, 2008):

$$Crashes/year=0.119(Conflicts/hr)^{1.419}$$

The predicted crashes correlated well to some of the existing crash prediction models. Observed data from ten single lane roundabouts in Indiana was then used to model conflicts in VISSIM and SSAM and a model calibrated to predict observed crashes from the estimated conflict frequencies:

Crashes/year = 0.796 × exp(0.0486 × Conflicts/hr)

5.5.3 Other-Conflict Based Surrogate Approaches

Richfield et al. (2014) collected 216 hours of video before and after changes were made to striping and signage along with accompanying education campaign and traffic enforcement at a two-lane roundabout that had been experiencing a high number of crashes. The video was used to record driver violations including incorrect turn (turning movement is not allowed from lane in which vehicle is driving) or failing to yield (not yielding to a circulating vehicle upon entering) and lane change violations (changing lanes within the circulating roadway). Other measured violations included wrong way violations, stop violations and incorrect entrance lane choice. Video was collected for six days each for three distinct periods, before, immediately after and one year after the changes were made. Traffic volumes were also recorded in order to normalize the violation data with respect to exposure by dividing the violation count by the traffic volume. Lasting improvements in most violation types was observed following the changes.

St.-Aubin et al. (2014) used video data to extract road user trajectories and then predicted conflicts defined by time-to-collision. The observed road user trajectories were used to build models that predict the probability of a road user's location at any time based upon its previous position, speed, travel lane, road type and other observable variables. These probabilities were then used to predict the probability of a conflict at all points within the area of the roundabout being modeled.

Zheng et al. (2013) examined the relationship between driver behavior and crash patterns using video data from two multilane roundabouts and quantifying 12 types of improper movements. Field videos were reviewed to identify instances of improper maneuvers at one quadrant of each roundabout. A conflict rate was determined by dividing the count of the improper movements in the observed time interval by the volume of vehicles in that time interval that enter the quadrant from the same lane where the undesired movement started. The rates were then used to derive expected crash percentages of crash types and compared to the observed crash data at one of the roundabouts studied. The results did not pass a statistical test of significance to accept the hypothesis that the expected crash pattern predicted by exposure rates could predict the actual crash pattern although the results for some crash types were accurate and the approach does show some promise.

Guido et al. (2011) used data obtained by video recording the operations at an urban roundabout in Italy focusing on rear-end conflicts. Five measures of safety were recorded, including: maximum deceleration rate to avoid a crash (DRAC), time to collision (TTC), proportion of stopping distance (PSD), time integrated time to collision (TIT), and crash

potential index (CPI). A vehicle is considered in conflict if the value of DRAC exceeds a given limit, 3.35 m/s2 in this paper. For TTC the measure assumes a lead vehicle maintains a constant speed and a conflict for TTC is assumed to be the minimum perception or reaction time of 1.5s. The PSD is measured as the distance remaining until the point of collision divided by the minimum acceptable stopping distance, which is based on the maximum deceleration rate. The TIT measures the difference between the time to collision and the time interval where a collision would be unavoidable and is integrated over the period it takes a vehicle to pass a given road segment. The CPI measures the proportion of time that the DRAC exceeds the braking capability for a given vehicle and road conditions. The video was analyzed for seven different vehicle paths through the roundabout. The authors conclude that safety performance is very dependent on how it is measured and different measures may highlight different geometric or operational characteristics of a roundabout as being of a safety concern.

5.6 INITIAL RECOMMENDATION FOR PREFERRED STATISTICAL MODELING APPROACHES

In assessing alternative statistical modeling approaches, and recommending the most appropriate ones for Project 17-70, it seems of interest to be consistent with what is being done in NCHRP Project 17-62 *Improved Prediction Models for Crash Types and Crash Severities* for the other HSM Chapters. Modeling approaches assessed include those for base models, safety performance functions, crash modification functions, crash modification factors, and crash type and severity models.

5.6.1 HSM base model-CMF approach vs SPFs with Multiple Covariates that Represent CMF Effects

According to a draft white paper for NCHRP Project 17-62, the "base condition-CMF" approach used currently in the HSM predictive chapter is better than an "SPF with multiple covariates" approach, where the latter approach includes representing variables of interest in design applications instead of CMFs. This assertion is based on the following reasons a) SPFs with multiple covariates can be unreliable in terms of transferability to a time or location other than what pertained for the data used for calibration and b) SPFs with multiple covariates typically will not contain all covariates of interest because of data availability and statistical estimation issues. The base condition-CMF approach avoids these issues because SPFs including only exposure as an explanatory variable are more transferable between time and location. What are required however are reliable CMFs for variables impacting crash frequency that are also transferable.

One key disadvantage of the base condition-CMF approach, according to that white paper, is that independence of safety effects is assumed when multiple CMFs are applied. This issue is being addressed in NCHRP Project 17-63 *Guidance for the Development and Application of Crash Modification Factors*, which involves Project 17-70 research team members. Close coordination with that project will ensure that the recommended prediction methodology for will reflect the results.

5.6.2 Estimating Crash Frequency SPFs

The recommendation is to estimate models with multiple covariates and, from these, derive base condition models by substituting base conditions for all variables other than exposure. How to define a base condition should consider what CMFs are available and the typical condition. This recommendation is because it is highly unlikely that a sufficiently large sample of roundabouts with identical values of base condition variables can be obtained for direct estimation of base models.

The models with multiple variable covariates, according to the NCHRP Project 17-62 white paper, directly incorporate the effects of traffic, geometric and other factors that are associated with, or impact, safety. The model intercept applies to average conditions of variables that could not be included in the model because of data availability of statistical estimation issues. These models were used to develop the base-condition models for the current HSM two-lane rural highways chapter. For some crash and site types, it is possible that AADT may be the only variable that could be included. This was in fact the case for several HSM models for urban and suburban arterials and rural multilane roads.

The Negative Binomial model, also called the Poisson-Gamma model is preferred for crash count modeling because it models a count distribution and can handle the issue of over-dispersion in crash count data, where the variance exceeds the mean as is required for the Poisson distribution. Thus it is the recommended primary model of choice for roundabout crash prediction.

Then other main reason for recommending NB model estimation over others is that a dispersion parameter that is fundamental to the empirical Bayes estimation procedure for sites with a crash history that are being redesigned can be estimated. In the HSM, the dispersion parameter is assumed to be constant, except for rural multilane roads, for which the dispersion is inversely proportional to length. Exploration of the calibration of a dispersion parameter that varies with other factors (e.g., total AADT entering a roundabout) is recommended.

5.6.3 Estimating SPFs for Different Crash Types and Severities

The approach currently employed in the HSM for prediction of crashes by type and severity for several facility types is to first predict total crashes using an SPF, then predict crashes by type and/or severity using aggregate proportions estimated from application data. However, as noted in the NCHRP 17-62 white paper, these proportions may depend on variables such as traffic volume and, in effect, the variable coefficients for a crash type or severity SPF can be quite different from those for an SPF based on all crashes.

To overcome this difficulty, and following the plan for NCHRP Project 17-62, two alternative approaches will be considered before selecting the best one for each roundabout crash type/severity situation.

1. Develop SPFs for estimating frequency of total crashes and crashes of each type, using NB modeling as suggested above, and then predict severity using a probabilistic

approach that employs discrete choice modeling to estimate severity distribution functions (SDFs) as was done for the proposed HSM freeways chapter (Bonneson et al. 2012).

2. Develop SPFs directly for each crash type/severity combination using NB modeling as suggested above.

In estimating the probability models the research team will consider the three basic modeling approaches planned for NCHRP Project 17-62 that were discussed in Section 5.4.2. One approach is based on the use of ordered discrete outcome models. The second approach is based on the use of unordered multinomial discrete outcome models. The third approach is based on the use of partial proportional odds models. As noted in Section 5.4.2, researchers for NCHRP Project 17-45 *Enhanced Safety Prediction Methodology and Analysis Tool for Freeways and Interchanges* (Bonneson at al., 2012) selected the standard multinomial logit (MNL) model as the basis for SDF development.

It is possible that data will be limited for some combinations of site type, crash type, and severity. In these situations, it may still be necessary to fall back to the current HSM approach of using proportions, and developing these proportions for each crash type.

5.6.4 Estimating CMFs and CMFunctions

The coefficients of models with multiple covariates will be used to develop the SPFs for Project 17-70. These models can also be used to derive some CMFs for the base-model-CMF prediction. Given the dearth of CMFs for roundabout features, it is recommended to explore the development of new CMFs using these models. However, it is recognized that they may not generally be appropriate for this purpose due to problems with endogeneity, correlation amongst predictor variables and omitted variable bias. These limitations of estimating CMFs using such models can be overcome by corroborating the results with intuitive reasoning, results of other studies, including the application of the other two approaches where possible. It is expected that the application of each approach will be limited due to data availability constraints, but that pursuing them would still be worthwhile if they can serve to corroborate each other.

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Chapter 6: Roundabout Related Crash Definitions

6.1 INTRODUCTION AND PURPOSE

One of the key objectives of this research was to connect how geometric and operational features influence crashes at roundabouts. Therefore, the research team established a clear definitions for crashes at roundabouts. This included two key considerations: 1) Identifying which crashes occur as a result of the presence of a roundabout; and 2) Defining crash types for the roundabout intersection. These two considerations are discussed below.

While the types of crashes that occur at roundabouts are the same types of at other intersection forms, the typology used to categorize those crashes may need to be unique to roundabouts to make the crash descriptions more useful in practice. This includes, for example, differentiating between side-swipe crashes by location in the roundabout (e.g., entry-circulating, exit-circulating). The tradeoff to developing a typology specific to roundabout intersections is that this approach requires researchers and practitioners to post-process crash data and redefine (based on their judgment and information available in the crash data) the initially reported crash type.

6.2 IDENTIFYING ROUNDABOUT CRASHES

The research team reviewed several resources to explore how best to determine whether or not a crash occurred at or due to a roundabout. The research team considered the Highway Safety Manual's definition of an intersection crash, common crash reporting trends by public agencies in the U.S.

The Highway Safety Manual defines crashes as those that occurred at the intersection proper as well as those occurring on approach to the intersection that are considered intersection-related. Intersection-related is defined to mean the crashes that would not have occurred if an intersection had not been present.

In many states, crashes at intersections are defined as those occurring at the intersection proper, those entered by the responding police officer as intersection-related, and/or crashes that occurred within a specific distance of an intersection (e.g., 250 feet).

For crash model prediction development by Rodegerdts et al. (2007), roundabout crashes were identified as those occurring at the intersection proper and provided by the jurisdictions as related to the roundabout. There was no common definition.

Based on the above practices, for Project 17-70, the research team identified roundabout crashes as those that occur at the intersection proper, coded by the responding police officer as intersection-related, and crashes occurring from yield line back approximately 250 feet.

6.3 CRASH TYPE DEFINITIONS FOR CRASHES AT ROUNDABOUTS

Based on reviewing the literature and outreach to public agencies in the U.S. with roundabouts, the research team has found that researchers tend to develop roundabout-specific crash definitions, while public agencies in the U.S. generally use the same crash definitions for roundabouts as they do at other intersections.

6.3.1 Research Definitions for Crashes at Roundabouts

Crash types at roundabouts have been defined by previous research efforts to develop crash prediction models in the U.S., United Kingdom, Australia, Sweden, and New Zealand. These are discussed below.

United States - NCHRP Report 572 Roundabouts in the United States (Rodegerdts et al., 2007)

NCHRP Report 572 *Roundabouts in the United States*, produced planning-level crash prediction models to predict overall safety performance at the intersection level and design-level crash prediction models that take into account different design characteristics to predict crashes at the approach level. For the intersection-level models, the NCHRP Report 572 team analyzed total crashes and injury crashes (i.e. those involving a fatality or definite injury, but not a possible injury or property damage only). For the approach-level models, the project used the following crash definitions:

- Entering-Circulating: Crashes occurring between an entering vehicle and a circulating vehicle
- Exit-Circulating: Crashes occurring between an exiting vehicle and a circulating vehicle
- Approaching: A crash on an approach to the roundabout

The project also categorized approaching crashes as either rear-end or loss of control in summarizing existing data. Pedestrian and bicycle crashes were also included in the existing data summary, but were not included as specific types in the models.

These crash definitions were used because they are consistent with previous international efforts, described below, and they provide a better connection to roundabout-specific design features, helping practitioners understand where design deficiencies may be leading to crashes.

International Research Efforts

The crash definitions used in international research efforts are similar to those used by Rodegerdts et al. (2007). Based on reviewing several significant international research efforts, using crash definitions specific to roundabouts was a relatively direct use of the crash data available for those intersections. Crash data in other countries with a longer history of circular intersections appear to have adjusted crash reporting to fit the circular geometry of the intersections.

For example, Turner et al. (2009), in describing the crash data used to develop the crash prediction models for roundabouts in New Zealand, notes that the crash models were developed from the major crash types extracted from the Ministry of Transport's Crash

Analysis System. The major crash types used in the crash modeling exercises for Turner et al. (2009) were:

- Entering vs. circulating (motor vehicle only)
- Rear-end (motor vehicle only)
- Loss of control (motor vehicle only)
- Other (motor vehicle only)
- Pedestrian [involved]
- Entering vs. circulating bicyclist
- Other motor vehicle vs. bicyclist

Similar crash types were used by Maycock and Hall (1984), Arndt (1994 & 1998), and Brude and Larsson (2000) in their development of crash prediction models for roundabouts in the United Kingdom, Australia and Sweden, respectively. Table A6-1 summarizes the crash types used in each of the studies.

Table A6-1. Summary of Crash Types Explored from Past Research Studies

Crash Type	Study		
	Maycock and Hall (1984)	Arndt (1994 and 1998)	Brude and Larsson (2000)
Single-Vehicle	X	X	
Approaching	X		
Entering-circulating	X	X	
Other	X		
Pedestrian	X		X
Rear-end on Approach Lanes		X	
Exiting-circulating		X	
Sideswipe		X	
All Crashes			X
Bicyclist			X

Overlapping crash types between the international research studies summarized above and U.S.-based Rodegerdts et al. (2007) appear to have mostly consistent definitions. One difference is the loss-of-control crash type in Rodegerdts et al. (2007) is restricted to single-vehicle crashes on the approach to the roundabout whereas the loss-of-control crash type used by Turner et al. (2009) and single-vehicle crash definitions used by Maycock and Hall (1984) and Arndt (1994 and 1998) include single-vehicle crashes on approach to and within the circulatory roadway.

Agency Definitions for Crashes at Roundabouts

As part of the literature review activities, the research team reached out to 17 agencies at the state and local level to learn about their experiences with roundabouts. One of the pieces of information the research team collected was how roundabout crashes were coded in their crash

databases. The research team received feedback from nine agencies and learned that this sampling of state departments of transportation (DOTs) and local agencies use the same crash types at roundabouts as they do at other intersections (e.g. angle, sideswipe). The Alaska Department of Transportation and Public Facilities indicated that they have a field code on the crash report that allows responding officers to code the crash as being at a roundabout. Other agencies rely on the crash location to determine if the crash occurred at or near an intersection and the type of control present at the intersection. The Washington State Department of Transportation did indicate they were exploring opportunities to revise crash reporting at roundabouts to be more consistent with the crash types used in the research by Rodegerdts et al. (2007) and reflected in NCHRP Report 672 *Roundabouts: An Informational Guide, Second Edition*.

6.3.2 Proposed Definitions for Roundabout Crashes for Project 17-70

The purpose and objectives of Project 17-70 is to develop crash prediction models that practitioners can use to understand and evaluate the tradeoffs between different geometric and operational features of a roundabout. The research team has identified two alternative approaches for defining crash types at roundabouts that achieve the above objective. One approach uses the crash type definitions traditionally reflected in crash databases within the U.S. (e.g., rear-end, sideswipe). The second approach uses crash type definitions unique to circular intersections (e.g., entering-circulating, exiting-circulating). These two approaches are discussed below.

Traditional Crash Type Definitions

Traditional crash type definitions reported in most crash databases include crash types such as rear-end, turning, angle, single-vehicle, fixed-object, sideswipe and others that describe the type of collision without reference to where it occurred on the roadway segment or within the intersection.

The advantages of using these crash type definitions in Project 17-70 include: 1) Less time intensive for the research team to assemble databases and develop crash prediction models; 2) Easier for practitioners to apply the crash prediction models at existing roundabouts and apply the empirical-Bayes method (i.e., integrate roundabout crash history into the crash prediction to improve its predictive power); and 3) Helps create and maintain consistency for practitioners evaluating and comparing multiple intersections with different types of traffic control.

The potential disadvantages of this approach include: 1) Deviating from international research and previous U.S.-based research may make it more difficult to compare Project 17-70 crash prediction models to previous work; and 2) Less of a clear connection between crash type and the location of those crashes within the roundabout (e.g., rear-end crashes could occur on approach to or at the entry of the roundabout vs. entering-circulating crash clearly occurred at the roundabout entry).

Roundabout Specific Crash Types

Roundabout specific crash types have been used in previous research in an attempt to more clearly tie geometric and operational features of a roundabout to crash frequency. Potential roundabout specific crash types under consideration for Project 17-70 include the following.

- Entering-Circulating: Crashes occurring between an entering vehicle and a circulating vehicle.
- Exiting-Circulating: Crashes occurring between an exiting vehicle and a circulating vehicle.
- Rear-end on Approach Lanes: A rear-end crash involving vehicles on the approach to the roundabout.
- Loss of Control: A single-vehicle crash on approach to the roundabout or in the circulatory roadway.
- Pedestrian: A crash involving a pedestrian.
- Bicyclist: A crash involving a bicyclist.

The advantages of the above crash type definitions include: 1) They capture the unique aspects of roundabouts in a way that traditional crash types do not thereby helping to connect geometric and operational features of the roundabout to crashes; and 2) Allow for relatively easy comparison to previously developed U.S.-based roundabout crash prediction models and international roundabout crash prediction models.

The potential disadvantages of using the above crash definitions include: 1) More time intensive for the research team to develop the crash prediction models because each crash will need to be re-defined to fit the above categories; 2) More time intensive for practitioners to apply the crash prediction models at existing roundabouts because they would need to re-define historic crashes to fit the above categories in order to apply the empirical-Bayes method; 3) Inconsistent crash types relative to those used at other intersection forms and relative to those used in the *Highway Safety Manual*. Of these potential disadvantages, the research team is most concerned about items and 1) and 2) noted above. Rodegerdts et al. (2007) used roundabout specific crash type definitions similar to those noted above. The effort to redefine and recode each crash for each roundabout site would limit (more than if the traditional crash type definitions were used) the total sample size the research team could use in developing the crash prediction models. When less data is used to develop and validate the models, the inherent risk is that the final crash prediction models are of lower quality with less applicability than if larger sample sizes are used. Similarly, the effort to redefine and recode roundabout crashes by practitioners for use of the models is also a concern with respect to developing readily applicable prediction tools.

6.4 SUMMARY OF KEY FINDINGS

With respect to identifying crashes that are roundabout-related crashes (i.e., more generally intersection-related crashes), the research team proposes using criteria similar to the *Highway Safety Manual* and crash reporting practices in most jurisdictions. Therefore, for Project 17-70 roundabout crashes are as those that occur at the intersection proper, coded by the responding

police officer as intersection-related, and crashes occurring from yield line back approximately 250 feet.

With respect to crash type definitions, Project 17-70 used the traditional crash type definitions for the intersection-level models and the roundabout-specific crash types for the leg-level models.

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Appendix B – Data Reduction Procedures Guide

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Chapter 1: Introduction

1.1 PURPOSE AND SCOPE OF DATA REDUCTION PROCEDURES GUIDE

The purpose of the Data Reduction Procedures Guide was to create consistency in how data attributes were collected for Project 17-70. The Data Reduction Procedures Guide focused on the process used to collect the roadway inventory and speed data attributes shown in Table B1-1.

Table B1-1 Summary of Roadway Inventory and Speed Data Attributes

Data Attribute	Anticipated Source and Tools
Area Type (urban/suburban, rural)	US Census Data
Opening Year	Public Agency Data or Google Earth Historical Imagery
Ramp Terminal Intersection	Google Earth Aerial Imagery
Number of Circulating Lanes	Google Earth Aerial Imagery
Number of Legs	Google Earth Aerial Imagery
Number of Entering Lanes per Leg	Google Earth Aerial Imagery
Number of Exiting Lanes per Leg	Google Earth Aerial Imagery
Presence of Right-Turn By-Pass Lane	Google Earth Aerial Imagery
Presence and Type of Lane Use Markings	Google Earth Aerial Imagery
Number of Luminaires within 250 ft of Roundabout	Google Earth Aerial Imagery and AutoCAD
Entry Width (ft) – Measured at Yield Line	Google Earth Aerial Imagery and AutoCAD
Angle to Next Leg (deg)	Google Earth Aerial Imagery and AutoCAD
Inscribed Circle Diameter (ft)	Google Earth Aerial Imagery and AutoCAD
Circulating Width (ft)	Google Earth Aerial Imagery and AutoCAD
Approach Half-Width (ft)	Google Earth Aerial Imagery and AutoCAD
Number of Access Points (i.e., Driveways) within 250 feet of Roundabout	Google Earth Aerial Imagery and AutoCAD
Posted Speed (mph)	Google Earth Street View

Data Attribute	Anticipated Source and Tools
Entering Vehicle Speed (mph)	Google Earth Aerial Imagery, NCHRP Report 672 Methodology and AutoCAD Software
Circulating Vehicle Speed (mph)	

A companion document titled Roadway Safety Database Organization documents the database framework that was used to develop the Project 17-70 databases. The Roadway Safety Database Organization (see Appendix C) includes the variable names corresponding each data attribute collected for Project 17-70. It also includes a brief description of each data attribute. Therefore, the Roadway Safety Database Oganization document repeats in brevity the detailed information regarding the roadway inventory and speed data contained in the Data Reduction Procedures Guide and provides additional information about variables to be used to record attributes such as roundabout location, traffic volume, crash data, and start/end of evaluation periods.

The Data Reduction Procedures Guide should be used to understand how to collect and, in some instances measure, the roadway inventory and speed data collected for Project 17-70. The Roadway Safety Database Organization should be used to understand how data are coded into the databases, including roadway inventory, speed, traffic volume, and crash data. These are important companion documents.

1.2 SCOPE OF DATA COLLECTION

Project 17-70 data collection focused on roundabout sites with three or four legs and one to two circulating lanes. Roundabout sites with more than four legs were identified and/or more than two circulating lanes will be noted. The research team, with the Panel, determined additional data collection for roundabouts with more than four legs and/or more than two circulating lanes was not appropriate given their limited representation in the data set.

1.3 LEG NAMING CONVENTION

Throughout the Data Reductions Procedures Guide and the Roadway Safety Database Organization the legs of each roundabout were named as follows:

- N = Will be used to denote the northern leg;
- E = Will be used to denote the eastern leg;
- S = Will be used to denote the southern leg; and
- W = Will be used to denote the western leg.

Other compass directions—e.g., SW for southwest—were used on a case-by-case basis to clarify unusual geometry in cases where the above notations may be ambiguous.

Chapter 2: Roadway Inventory Data

2.1 INTRODUCTION

This chapter presents guidance for gathering and measuring the roadway inventory data attributes collected at roundabout sites for Project 17-70. These data are summarized in Table B2-1 alongside the source of the data.

Table B2-1 Summary of Roadway Inventory Data Attributes

Data Attribute	Anticipated Source and Tools	Relevant Section of Chapter
Area Type	US Census Data	2.2
Opening Year	Public Agency Data or Google Earth Historical Imagery	2.3
Ramp Terminal Intersection	Google Earth Aerial Imagery	2.4
Number of Circulating Lanes	Google Earth Aerial Imagery	2.5
Number of Legs	Google Earth Aerial Imagery	2.6
Number of Entering Lanes per Leg	Google Earth Aerial Imagery	2.7
Number of Exiting Lanes per Leg	Google Earth Aerial Imagery	2.8
Presence of Right-Turn By-Pass Lane	Google Earth Aerial Imagery	2.9
Presence and Type of Lane Use Markings	Google Earth Aerial Imagery	2.10
Number of Luminaires within 250 ft of Roundabout	Google Earth Aerial Imagery and AutoCAD	2.11
Entry Width (ft) – Measured at Yield Line	Google Earth Aerial Imagery and AutoCAD	2.12
Angle to Next Leg (deg)	Google Earth Aerial Imagery and AutoCAD	2.13
Inscribed Circle Diameter (ft)	Google Earth Aerial Imagery and AutoCAD	2.14

Table B2-1 continued Summary of Roadway Inventory Data Attributes

Data Attribute	Anticipated Source and Tools	Relevant Section of Chapter
Circulating Width (ft)	Google Earth Aerial Imagery and AutoCAD	2.15
Approach Half-Width (ft)	Google Earth Aerial Imagery and AutoCAD	2.16
Number of Access Points (i.e., Driveways) within 250 feet of Roundabout	Google Earth Aerial Imagery and AutoCAD	2.17

The following sub-sections present the data reduction guidance organized per data attribute.

2.2 AREA TYPE

Area type associates each roundabout with a basic land use type of either: 1)Urban/Suburban; or 2) Rural. A roundabout characterized as urban/suburban is located within an incorporated area or is in an unincorporated area defined as a Census Designated Place (CDP) where the population is greater than 5,000 people. A roundabout characterized as rural is located in unincorporated area where the population is less than 5,000 people. The CDP serves as the spatial limit on the unincorporated areas. This definition is consistent with the definition in the Highway Safety Manual; specificity has been added (i.e., use of Census Designated Place) to explain how Project 17-70 defined unincorporated areas.

To designate each roundabout site as urban/suburban or rural:

- Use the latitude and longitude information about the roundabout intersection location to map each roundabout site in GIS;
- Obtain GIS shapefiles of CDP boundaries within each focus state from the US Census Bureau;
- Obtain the U.S. population table (2008-2012 ACS 5 Year Estimates) from the the US Census Bureau;
- Join the CDP boundary GIS-shape files with the population tables based on the "GEOID" field, a unique ID for each CDP defined by US Census Bureau, in each field;
- Identify the CDPs where the population is greater than 5,000 people and label them as "urban/suburban";
- Identify the roundabout sites within urban/suburban areas identified above and associate the roundabouts sites as urban/suburban;
- Associate the rest of roundabout sites as rural; and

- Transfer the urban/suburban and rural classifications for each roundabout site to the database.
    - Area Type should be recorded under the column "AreaType". Enter a "U" for urban/suburban areas and enter a "R" for "rural" areas.

The area type definitions provided above are consistent with those used in the Highway Safety Manual (HSM, 2010).

2.3 OPENING DATE

The opening date for each roundabout will be used for two purposes. One purpose is to screen roundabout sites so the candidate site database includes roundabouts with a minium of three years of crash data. The research team assumes that states will be able to provide complete crash data for the roundabout sites through the year 2013. Therefore, the candidate site database will include roundabouts with an opening year of 2009 or earlier (e.g., 2008, 2007). A roundabout opening in 2009 has the potential for at least three to four years of after data (i.e., January 1, 2010 through December 31, 2012 or December 31, 2013), depending on the respective jurisdictions state of crash data record keeping. The second purpose of the opening year is to use a subset of the roundabout sites in analysis aimed at quantifying or at least qualitatively better understanding the effect of driver learning curve on crashes at roundabouts.

The sources of the opening year information are: 1) Modern roundabout database (which is itself largely based on a combination of state roundabout inventories, Google Earth historic aerial imagery, and online articles about the subject roundabout); 2) State roundabout inventories received as part of Task 1 within Project 17-70; and 3) Google Earth historic aerial imagery. The latter two items will be used for verification where gaps may exist in the database information.

2.3.1 Opening Date to Screen Roundabout Sites

For the purpose of screening roundabout sites to ensure a minimum of three of years of crash data, Google Earth historic aerial imagery will be used to check and supplement the opening date information contained in the modern roundabout database and state inventories. This exercise will be a relatively quick review of the Google Earth historic aerial imagery nearest to the specified opening year to confirm the roundabouts' completion. If the Google Earth historic aerial imagery contains an incomplete roundabout or another intersection form, the opening year for the site will be revised to the year in which the Google Earth aerial imagery contains a completed roundabout. If an agency can provide updated and more accurate proof the opening date, the year will be revised accordingly.

If an opening date is not known, the Google Earth historic aerial imagery will be used to identify the year in which the roundabout first appears completed and open to traffic. This approach will ensure that the sites and crash data used per site reflect the intersection's crash performance with a roundabout present.

2.3.2 Opening Date to Investigate Driver Learning Curve

A precise and accurate opening date for each roundabout site used by the research team to evaluate the effect of driver learning curve on crashes at roundabouts is critical. Therefore, the research team will use the subset of roundabout sites for which the opening date (day, month, and year or at least month and year) is known and well-established. For this subset of roundabout sites, the opening date will be obtained through the modern roundabout database and cross-referenced with the information in the state roundabout inventories. If there are discrepancies between these two resources, Nick Foster will contact the appropriate agency to obtain additional details to confirm when the roundabout opened to traffic. If an opening date or an opening year cannot be determined with confidence, then the specific site will not be used to investigate driver learning curve.

2.3.3 Recording Opening Date

Opening date data will be recorded as described below.

OpenDate = If known, input the year, month and day in the format YYYY-MM-DD. Where YYYY is the year the site was opened. Use "0000" if year is unknown or not known with confidence. MM is the number of the month (e.g., December = 12). Use "00" if month is unknown or not known with confidence. DD is the day of the month (e.g., 09 for the 9th). Use "00" if the day is unknown or not known with confidence.

2.4 RAMP TERMINAL INTERSECTION

For Project 17-70, a roundabout is considered a ramp terminal intersection if one or more of the legs to the roundabout is a ramp leading from or to a controlled access facility, such as a freeway. To determine whether or not a roundabout site is a ramp terminal intersection, use a Google Earth image of the roundabout site's location and review each leg to determine if it is a ramp leading from or to a controlled access facility.

The data regarding ramp terminal intersection should be recorded in the related columns as described below.

RampTerm = If one or more of the legs is a ramp to or from a controlled access facility, enter "Y" for "yes". If it is not, enter "N" for "no".

2.5 NUMBER OF CIRCULATING LANES

The circulating lane (or lanes) circle the central island of the roundabout. For a single-lane roundabout, there is one circulating lane around the central island. For multilane roundabouts, the number of circulating lanes can vary. Some portions of the same roundabout can have one circulating lane while others have two or more. Some multilane roundabouts have circulatory roadways that are partially or completely unstriped, even though multilane entries feed it. This section describes how to identify and record the number of circulating lanes for the purpose of Project 17-70 research.

1. Via Google Earth zoom or search to the roundabout site of interest.
2. Count the number of lanes circulating around the central island; lanes are often separated by dashed or solid lines.
3. If the number of circulating lanes is constant around the central island, then record that number under the comlumn labled "CirculatingLanes". For example, a single-lane roundabout shown in Figure B2-1 would be recorded with a "1" within the "CirculatingLanes" column.

Figure B2-1. Single Lane Roundabout Example

Image Source: Google Earth, 2014.

4. If the number of circulating lanes varies around the central island, then record the greatest number of circulating lanes in the column labled "CirculatingLanes". In the column labled, "Hybrid" record "Y" for "yes", and enter the number of circulating lanes per leg in the following columns:
    - CirculatingLanes_N = For the northern leg to the roundabout, enter the number of circulating lanes adjacent to the splitter island for that leg (conflicting with the entry).
    - CirculatingLanes_E = For the eastern leg to the roundabout, enter the number of circulating lanes in front of the entry for that leg (conflicting with the entry).
    - CirculatingLanes_S = For the southern leg to the roundabout, enter the number of circulating lanes in front of the entry for that leg (conflicting with the entry).
    - CirculatingLanes_W = For the western leg to the roundabout, enter the number of circulating lanes in front of the entry for that leg(conflicting with the entry).

    For example, the entries for the roundabout in Figure B2-2 would be:
    - CirculatingLanes = 2
    - Hybrid = Y

- CirculatingLanes_N = 2
- CirculatingLanes_E = 2
- CirculatingLanes_S = 1
- CirculatingLanes_W = 2

Figure B2-2. Multilane Hybrid Roundabout Example

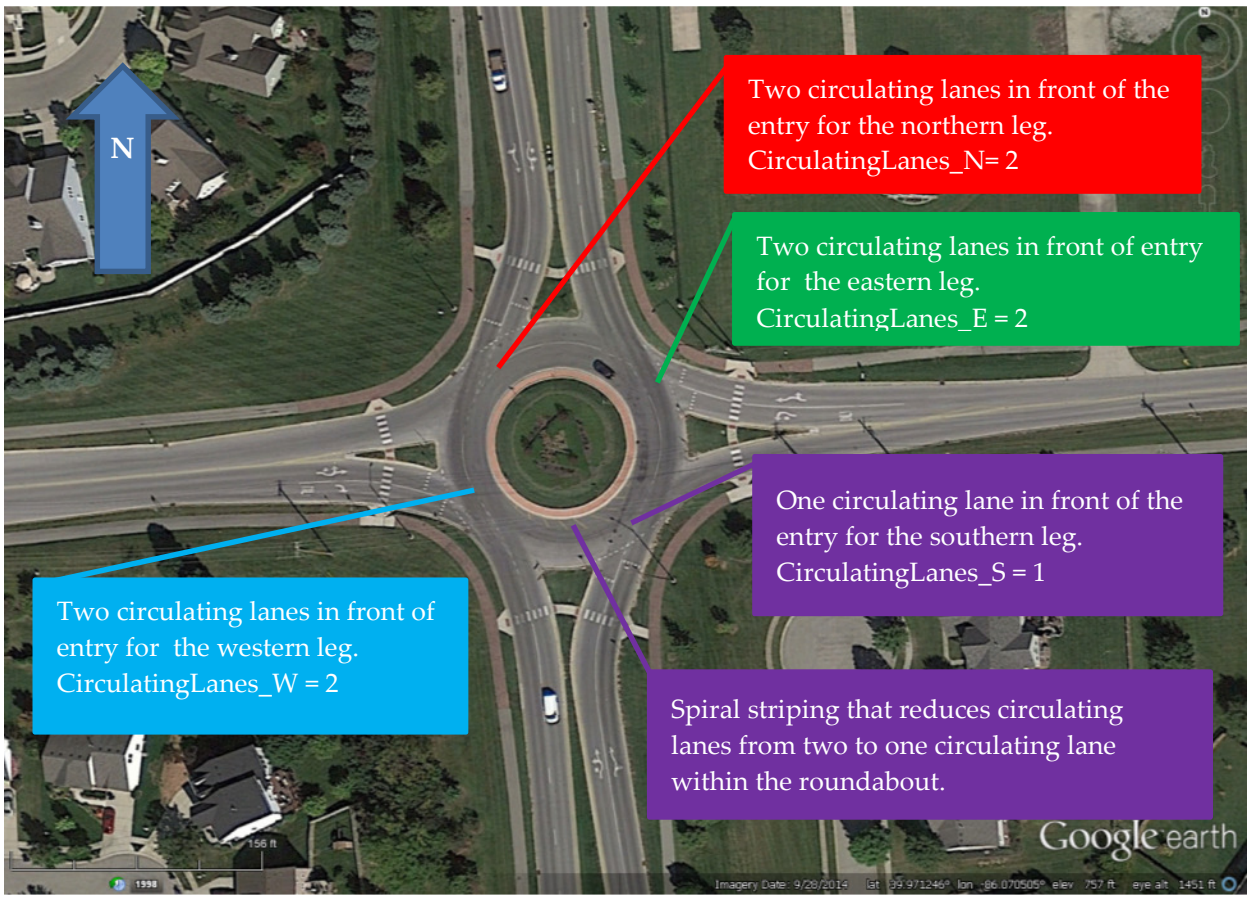


Image Source: Google Earth, 2014.

Figure B2-3 is another example of a hybrid roundabout that would have the following entries.

- CirculatingLanes = 2
- Hybrid = Y
- CirculatingLanes_N = 2
- CirculatingLanes_E = 1
- CirculatingLanes_S = 2
- CirculatingLanes_W = 1

Figure B2-3. Supplemental Mulilane Hybrid Roundabout Example

Image Source: Google Earth, 2014.

5. For roundabouts with a constant number of circulating lanes throughout the roundabout (e.g., Figure B2-1), entries should be:
    - CirculatingLanes = 1
    - Hybrid = N
    - CirculatingLanes_N = 1
    - CirculatingLanes_E = 1
    - CirculatingLanes_S = 1
    - CirculatingLanes_W = 1

If a leg does not exist, enter "0" for the corresponding variable.

2.6 NUMBER OF LEGS

Use Google Earth aerial imagery to confirm and/or record the number of legs at each roundabout site. The initial roundabout database may include information on the number of legs at a given roundabout. This should be confirmed for each site using Google Earth aerial imagery and revised, as needed.

For Project 17-70, a leg of a roundabout should be counted as such it is a public roadway (i.e., not a private driveway) A leg does not need to accommodate two-way traffic. A leg can, but does not need to, separate opposing traffic with a physical or painted splitter island. A private driveway with access directly to the roundabout will be recorded as an Access Point; identifying and recording access points is discussed in Section 2.17 below. Figure B2-4 provides an example of a roundabout with three legs and one access point. Three of the legs have splitter islands and are public streets while there is also a private driveway that would be recorded as an access point. For Project 17-70, the number of legs at the roundabout in Figure B2-4 would be recorded as "3" and entered into the column "NumberLegs".

Figure B2-4. Identifying Number of Legs

Image Source: Google Earth, 2014.

2.7 NUMBER OF ENTERING LANES PER LEG

Use Google Earth aerial imagery to confirm and/or record the number of entering lanes at the entry line per leg at each roundabout site. The initial roundabout database may include information on the number of entering lanes at a given roundabout. This should be confirmed and/or determined for each site using Google Earth aerial imagery and revised, as needed.

The number of entering lanes per roundabout leg should be recorded as follows.

- EnteringLanes_N = Number of entering lanes on the northern leg

- EnteringLanes_E = Number of entering lanes on the eastern leg
- EnteringLanes_S = Number of entering lanes on the southern leg
- EnteringLanes_W = Number of entering lanes on the western leg

Using the convention above, the entries for the roundabout in Figure B2-5 would be as follows:

- EnteringLanes_N= 2;
- EnteringLanes_E = 1;
- EnteringLanes_S = 2; and
- EnteringLanes_S = 1.

Figure B2-5. Number of Entering Lanes per Leg

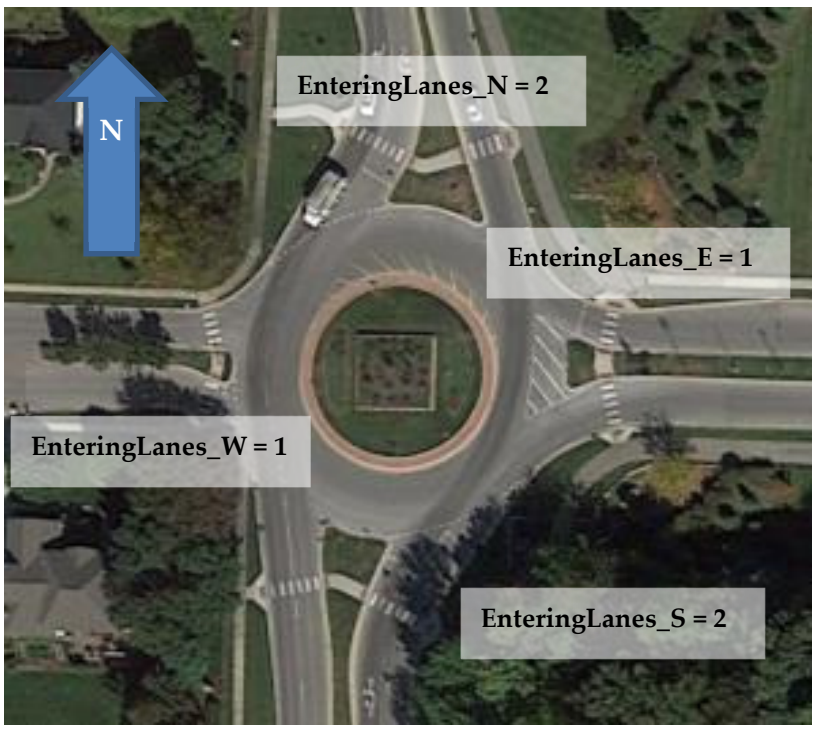


Image Source: Google Earth, 2014.

If a roundabout site has three legs, "0" should be entered for leg that does not exist. For example, the roundabout in Figure B2-6 would have the following entries:

- EnteringLanes_N = 0;
- EnteringLanes_E = 1;
- EnteringLanes_S = 1; and
- EnteringLanes_W = 1.

Figure B2-6 Number of Entering Lanes for a Roundabout with Three Legs

Image Source: Google Earth, 2014.

The number of entering lanes per leg does not include right-turn bypass lanes on the leg. Right-turn bypass lanes are recorded as a separate data attribute. For Project 17-70, a right-turn bypass lane is a lane, typically for right turns but sometimes for through movements at T-intersections, that is separated from the adjacent entry lane either by a channelized island or pavement markings used to create physical, horizontal separation between the bypass lane and the adjacent roundabout entry lanes. Figure B2-7 illustrates different types of bypass lanes. Right-turn bypass lanes should not be included in the count of number of entering lanes on a leg.

Figure B2-7. Right-Turn Bypass Lane Examples

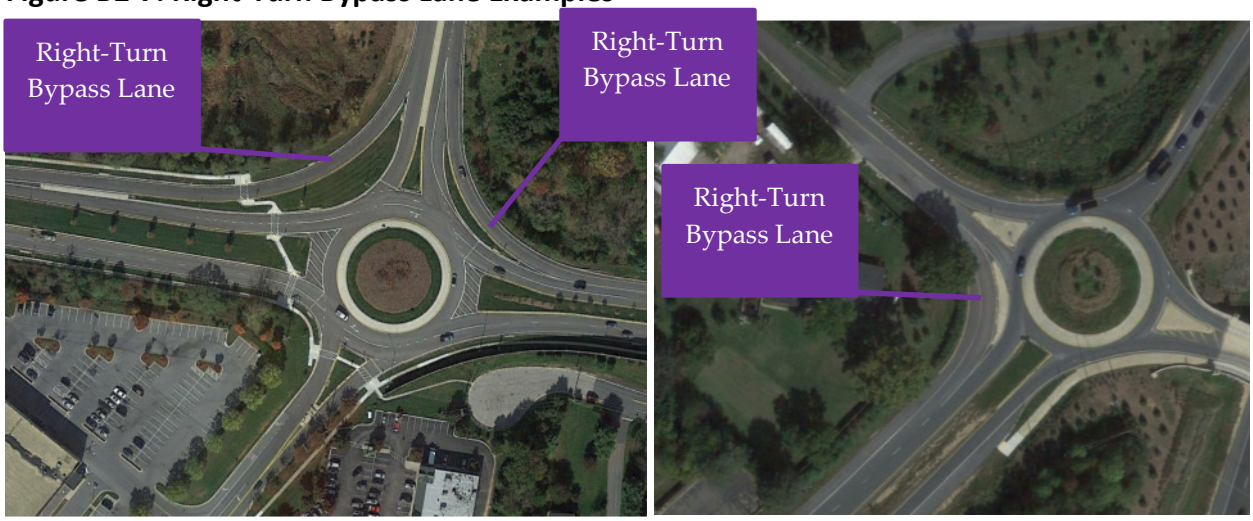


Image Source: Google Earth, 2014.

2.8 NUMBER OF EXIT LANES PER LEG

Use Google Earth aerial imagery to confirm and/or record the number of exiting lanes per leg at each roundabout site. The initial roundabout database may include information on the number of exiting lanes at a given roundabout. This should be confirmed and/or determined for each site using Google Earth aerial imagery and revised, as needed.

The number of exiting lanes per roundabout leg should be recorded as follows.

- ExitingLanes_N = Number of exiting lanes on the northern leg
- ExitingLanes_E = Number of exiting lanes on the eastern leg
- ExitingLanes_S = Number of exiting lanes on the southern leg
- ExitingLanes_W = Number of exiting lanes on the western leg

Using the convention above, the exit lanes for the roundabout in Figure B2-8 would be as follows:

- ExitingLanes_N = 1;
- ExitingLanes_E = 2;
- ExitingLanes_S = 1; and
- ExitingLanes_W = 2.

Figure B2-8. Number of Exit Lanes Example

Image Source: Google Earth, 2014.

If a roundabout site has three legs, "0" should be entered for a leg that does not exist. For example, the roundabout in Figure B2-9 would have the following recorded:

- ExitingLanes_N = 0;
- ExitingLanes_E = 1;
- ExitingLanes_S = 1; and
- ExitingLanes_W = 1.

Figure B2-9. Supplemental Example for Number of Exit Lanes

Image Source: Google Earth, 2014.

The number of exit lanes per leg does not include any lanes that may be formed from a bypass lane. Figure B2-10 illustrates a roundabout with a right-turn bypass lane and denotes the lane that should not be counted as an exit lane.

Figure B2-10. Right-Turn Bypass Lanes Not to be Counted as Exit Lanes

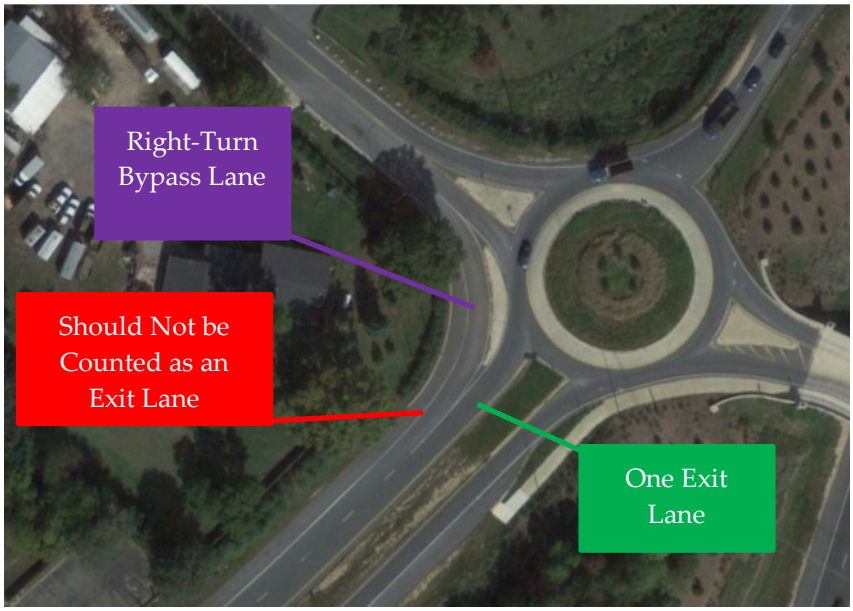


Image Source: Google Earth, 2014.

2.9 PRESENCE OF A BYPASS LANE

Use Google Earth aerial imagery to identify if a roundabout site has a bypass lane present and record the legs for which the bypass lane is present. See section 2.8 for the definition of a bypass lane used for this project. Figure B2-7 and Figure B2-10 above illustrate examples of right-turn bypass lanes.

The data to be recorded for right-turn bypass lanes is summarized below.

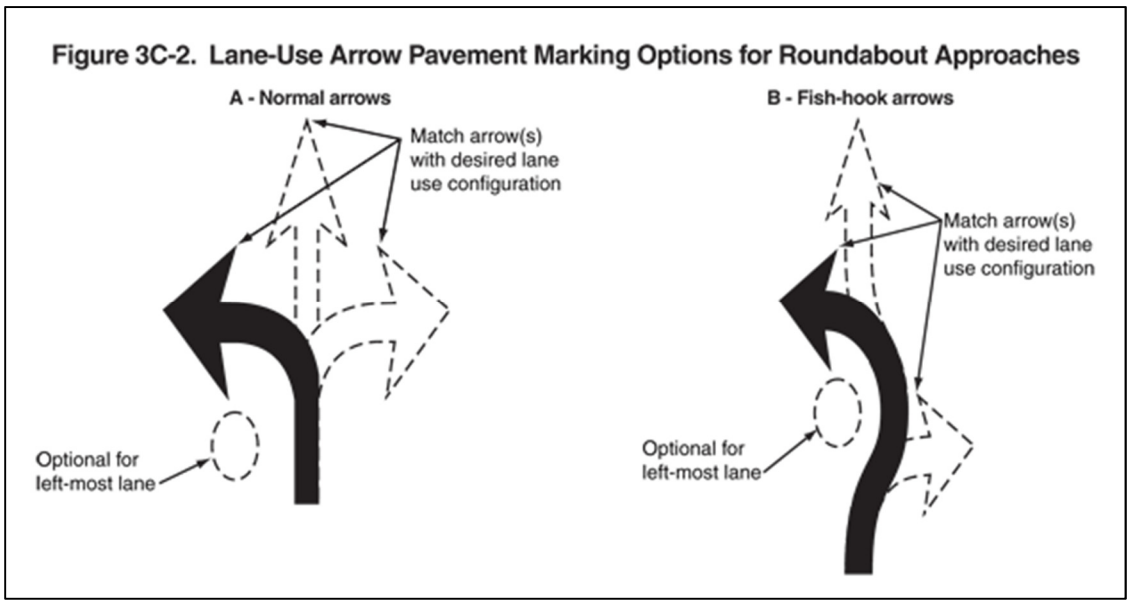
- Bypass_N = If the northern leg to the roundabout has a bypass lane adjacent to the entering lanes, enter "1". If it does not, enter "0".
- Bypass_E = If the eastern leg to the roundabout has a bypass lane adjacent to the entering lanes, enter "1". If it does not, enter "0".
- Bypass_S = If the southern leg to the roundabout has a bypass lane adjacent to the entering lanes, enter "1". If it does not, enter "0".
- Bypass_W = If the western leg to the roundabout has a bypass lane adjacent to the entering lanes, enter "1". If it does not, enter "0".

If one of the legs above does not exist at the roundabout, enter "0" for the leg.

2.10 PRESENCE AND TYPE OF LANE USE MARKINGS

Use Google Earth aerial imagery to identify the presence and type of lane use markings present on each leg of the roundabout site of interest. If lane use markings are present there are two basic types that are most likely to appear, these are shown in Figure B2-11 below.

Figure B2-11. Lane Use Markings at Roundabouts



Source: MUTCD, 2009.

The presence and type of lane use markings per leg at each roundabout should be recorded as described below.

- LaneMarkings_N = If lane use markings are not present on the northern leg, enter "B". If they are present and the lane use markings are Type A – Normal Arrows, enter "N". If they are present and the lane use markings are Type B – Fish-hook Arrows, enter "F". If they are present and not either Normal or Fish-hook Arrows, enter the letter "O".
- LaneMarkings_E = If lane use markings are not present on the eastern leg, enter "B". If they are present and the lane use markings are Type A – Normal Arrows, enter "N". If

they are present and the lane use markings are Type B – Fish-hook Arrows, enter "F". If they are present and not either Normal or Fish-hook Arrows, enter the letter "O".

- LaneMarkings_S = If lane use markings are not present on the southern leg, enter "B". If they are present and the lane use markings are Type A – Normal Arrows, enter "N". If they are present and the lane use markings are Type B – Fish-hook Arrows, enter "F". If they are present and not either Normal or Fish-hook Arrows, enter the letter "O".
- LaneMarkings_W = If lane use markings are not present on the western leg, enter "B". If they are present and the lane use markings are Type A – Normal Arrows, enter "N". If they are present and the lane use markings are Type B – Fish-hook Arrows, enter "F". If they are present and not either Normal or Fish-hook Arrows, enter the letter "O".

If one of the legs above does not exist at the roundabout, enter "0" for the leg.

Use Google Earth historical imagery (if available for the site) to identify the presence and type of lane use markings at the roundabout for the first and last year of crash data for the site. In some cases, the first and last year of crash data may not match the historical Google Earth aerial imagery; use the closest year available. If the presency and type of lane markings have changed over the course of the crash data for the site, enter "V" for varies for the appropriate approaches.

2.11 NUMBER OF LUMINARES WITHIN 250 FEET OF ROUNDABOUT

For Project 17-70, a luminare is a light fixture that appears to has been designed to illuminate the roadway right-of-way along the legs of the roundabout and through a roundabout. Luminares do not include adjacent lighting that is oriented to illuminate roadside buildings or features like billboards or other advertising signs. The process to identify the number of luminars within 250 feet of the roundabout is described below.

- Locate the roundabout using Google Earth Pro. Zoom in to 1:100 scale. Save image at high resolution (1488X944).
- Insert the image into AutoCAD model space at 43.2943 scale (this makes the image full scale; the 100ft scale bar becomes 100ft in model space).
- Estimate the inscribed circle diameter (ICD) as described in Section 2.14 below.
- Use the OFFSET command in AutoCAD to copy and offset a circle 250 feet back from the estimated ICD.
- Through visual inspection of the aerial, count the number of luminares within the offset circle that denotes the 250 foot buffer from the roundabout site of interest. As needed, supplement visual inspection of the aerial with Google Earth street view imagery.

The number of luminares will be recorded per leg as indicated below.

- Luminares_N = Number of luminares on the northern leg.
- Luminares_E = Number of luminares on the eastern leg.
- Luminares_S = Number of luminares on the southern leg.
- Luminares_W = Number of luminares on the western leg.

If one of the legs above does not exist at the roundabout, enter "0" for the leg.

2.12 ENTRY WIDTH

The entry width should be measured per leg for each roundabout site. The entry width is measured at the entrance line (i.e., the continuation of the inscribed circle diameter) and is the perpendicular width of the lane relative to a vehicle entering the circulatory roadway. The width is measured from the splitter island curb to the curb face at the outside edge of the travel way. In the absence of a curb on the outside edge of travel way, the width is measured from the spliter island curb face to the paved edge of travel way. The process used to make the entry width measurement on each leg is described below.

- Locate the roundabout using Google Earth Pro. Zoom in to 1:100 scale. Save image at high resolution (1488X944).
- Insert the image into AutoCAD model space at 43.2943 scale (this makes the image full scale; the 100ft scale bar becomes 100ft in model space).
- The entry width (ft) is measured by creating polylines (PLINE) at the curb (or edge of pavement) or lane lines on both sides of the entry lane.
- The entry width can then be measured from the point where the left side of the entry lane intersects the circle representing the Inscribed Circle Diameter (discussed in Section 1.14) to a point on the right side of the entry lane perpendicular to the first.
- The width is annotated in the drawing using an aligned dimension (DIMALIGNED).

Figure B2-12 illustrates the entry width measurement on a leg of a roundabout; as indicated the width is 23 feet.

Figure B2-12. Entry Width Measurement

Image Source: Google Earth Pro, 2014.

The entry width dimensions should be recorded as indicated below. Measurements should be entered in feet and rounded to the nearest foot.

- EntryWidth_N = Measured entry width for the northern leg.
- EntryWidth_E = Measured entry width for the eastern leg.
- EntryWidth_S = Measured entry width for the southern leg.
- EntryWidth_W = Measured entry width for the western leg.

If one of the legs above does not exist at the roundabout, enter “0” for the leg.

2.13 ANGLE TO NEXT LEG

The angle to next leg is used to measure the relative intersection skew; it is measured in degrees. It is measured for each leg to the roundabout using the process described below.

- Locate the roundabout using Google Earth Pro. Zoom in to 1:100 scale. Save image at high resolution (1488X944).
- Insert the image into AutoCAD model space at 43.2943 scale (this makes the image full scale; the 100ft scale bar becomes 100ft in model space).
- Create a circle with a radius of 100 feet radiating from the center of the Inscribed Circle Diameter.
- At each leg, draw a line starting at the intersection of the right curb (or edge of pavement) and a perpendicular point on the left curb (or edge of pavement).
- At each leg, draw a line form the midpoint of the above line and the center of the 100 foot circle.
- A Radial Dimension (DIMRADIUS) will then be added between each radial line for annotative purposes.

Figure B2-13 illustrates the above measurement.

Figure B2-13. Measuring Angle to Next Leg

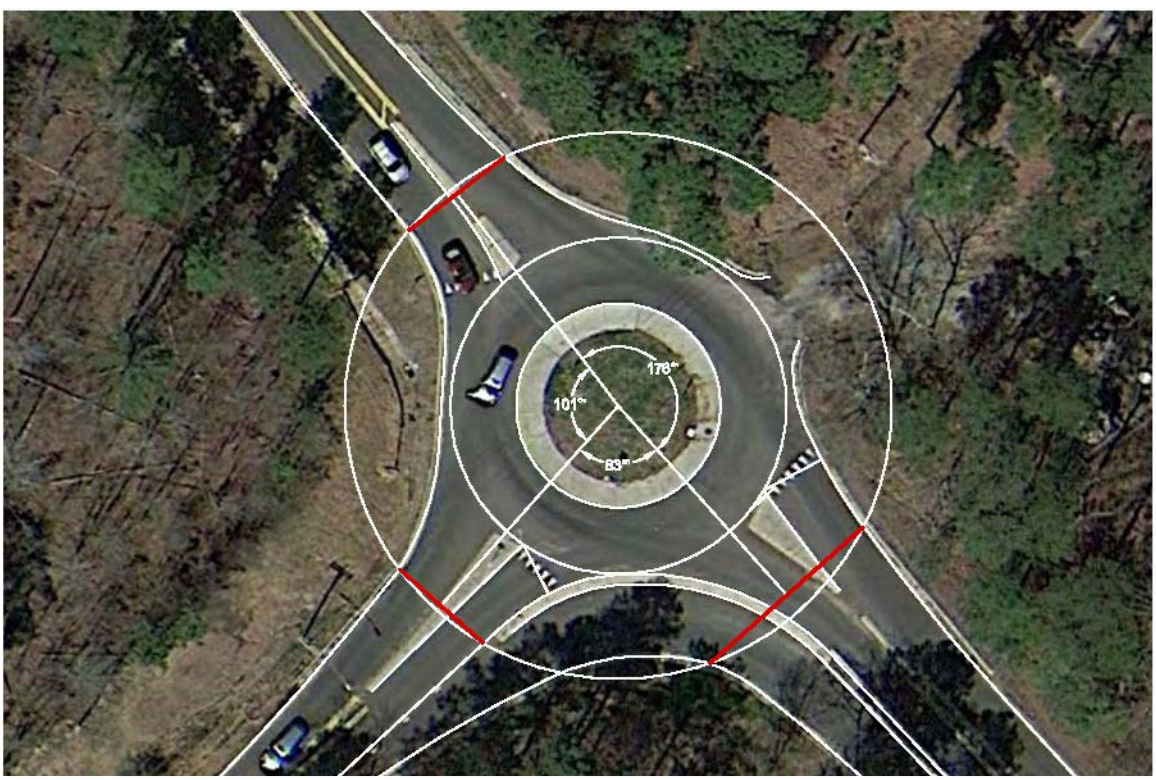


Image Source: Google Earth Pro, 2014.

The measurements are recorded for each leg as described below.

- Angle_N = Numerical angle rounded to the nearest degree for the northern leg (i.e., the angle between the northern and western legs).
- Angle_E = Numerical angle rounded to the nearest degree for the eastern leg (i.e., the angle between the eastern and northern legs).
- Angle_S = Numerical angle rounded to the nearest degree for the southern leg (i.e., the angle between the southern and eastern legs).
- Angle_W = Numerical angle rounded to the nearest degree for the western leg (i.e., the angle between the western and southern legs).

If one of the legs above does not exist at the roundabout, enter "0" for the leg.

2.14 INSCRIBED CIRCLE DIAMETER

The inscribed circle diameter (ICD) is the parameter used to discuss the general size of a roundabout. The ICD will be measured using the process described below. It will be measured in feet and rounded to the nearest integer foot per roundabout site.

- Locate the roundabout using Google Earth Pro. Zoom in to 1:100 scale. Save image at high resolution (1488X944).
- Insert the image into AutoCAD model space at 43.2943 scale (this makes the image full scale; the 100ft scale bar becomes 100ft in model space).
- In order to find the ICD, create a circle (enter CIRCLE command, select the 3 point circle option [3P], and select three points at the curb line (or edge of pavement if there is no curb). This traces the curb line (or edge of pavement).
- In the event that a roundabout does not have a uniformly circular shape, two (2) points at its largest diameter will be selected.
- The ICD will then be measured by offsetting from the circle representing the central island diameter to the nearest curb or splitter island.
- A Diametric Dimension (DIMDIAMETER) will then be added for annotative purposes.

Figure B2-14 illustrates the process used to measure the ICD for the roundabout sites. The measurement is recorded per roundabout in the column labeled "icd". The measurement is taken in feet and rounded to the nearest integer.

Figure B2-14. Measuring Roundabout Inscribed Circle Diameter

Image Source: Google Earth Pro, 2014.

2.15 CIRCULATING WIDTH

Circulating width is the width of the circulatory roadway within the roundabout. It is measured in feet and recorded as the nearest integer foot. The process to measure the circulating width is presented below.

- Locate the roundabout using Google Earth Pro. Zoom in to 1:100 scale. Save image at high resolution (1488X944).
- Insert the image into AutoCAD model space at 43.2943 scale (this makes the image full scale; the 100ft scale bar becomes 100ft in model space).
- Circulating width is the measurement between the circle representing the central island and the circle representing the ICD (see Section 1.14).
- An Aligned Dimension (DIMALIGNED) is added to measure between a point on the circle representing the central island diameter and a point on the circle representing the ICD which is perpendicular to each other.

Figure B2-15 illustrates the measurement of the circulating width at roundabout site where the width is consistent throughout the roundabout.

Figure B2-15. Measuring Consistent Circulating Width

Source: Google Earth Pro, 2014.

Roundabouts in which the number of circulating lanes varies and/or roundabouts that are not circular (e.g., oblong shaped) may have circulating widths that vary. In these instances, the aligned dimension will need to be used to measure the circulating width in front of the splitter island on each leg. The circulating width is then recorded per leg as described below.

- CirculatingWidth_N = Circulating roadway width (in feet, rounded to the nearest integer) in front of the northern leg using the same point as was used to measure the entry width.
- CirculatingWidth_E = Circulating roadway width (in feet, rounded to the nearest integer) in front of the eastern leg using the same point as was used to measure the entry width.
- CirculatingWidth_S = Circulating roadway width (in feet, rounded to the nearest integer) in front of the southern leg using the same point as was used to measure the entry width.
- CirculatingWidth_W = Circulating roadway width (in feet, rounded to the nearest integer) in front of the western leg using the same point as was used to measure the entry width.

For roundabout sites with a consistent circulating roadway width, the above data attributes will be the same number. If a leg does not exist for a roundabout site, enter “0”.

2.16 APPROACH HALF-WIDTH

The approach half-width is the width of the leg divided in two for each roundabout leg. To provide a consistent means for measuring the approach half-width, Project 17-70 is measuring the half-width up-stream of the roundabout prior to widening for the roundabout. The process for taking this measurement is described below.

- Locate the roundabout using Google Earth Pro. Zoom in to 1:100 scale. Save image at high resolution (1488X944).
- Insert the image into AutoCAD model space at 43.2943 scale (this makes the image full scale; the 100ft scale bar becomes 100ft in model space).
- Create a circle with a radius of 300 feet radiating from the center of the circle describing the inscribed circle diameter. Confirm from visual inspection that widening of the roadway has not occurred for the roundabout. If it has, move the measurement further upstream of the roundabout.
- Draw a polyline starting at the intersection of the right curb (or edge of pavement) and ending at a perpendicular point on the left curb (or edge of pavement).
- An Aligned Dimension (DIMALIGNED) is added to measure between the center point of the line representing the full width of the approach and the right curb (or edge of pavement). This measurement is the Approach Half-Width.

Figure B2-16 illustrates the approach half-width measurement.

Figure B2-16. Measuring Approach Half-Width

Source: Google Earth Pro, 2014.

The approach half-width is recorded as described below.

- HalfWidth_N = Half width measured in feet, rounded to the nearest integer for the northern leg.
- HalfWidth_E = Half width measured in feet, rounded to the nearest integer for the eastern leg.
- HalfWidth_S = Half width measured in feet, rounded to the nearest integer for the southern leg.
- HalfWidth_W = Half width measured in feet, rounded to the nearest integer for the western leg.

If a leg does not exist for a roundabout site, enter "0".

2.17 ACCESS POINTS WITHIN 250 FEET OF ROUNDABOUT

For Project 17-70, an access point is a driveway (not a public roadway) located along one of the legs of the roundabout of interest. The process to identify the number of access points within 250 feet of the roundabout is described below.

- Locate the roundabout using Google Earth Pro. Zoom in to 1:100 scale. Save image at high resolution (1488X944).
- Insert the image into AutoCAD model space at 43.2943 scale (this makes the image full scale; the 100ft scale bar becomes 100ft in model space).
- Estimate the inscribed circle diameter (ICD) as describedin Section 2.14 above.
- Use the OFFSET command in AutoCAD to copy and offset a circle 250 feet back from the estimated ICD.
- Through visual inspection of the aerial, count the number of access points within the offset circle that denotes the 250 foot buffer from the roundabout site of interest. As needed, supplement visual inspection of the aerial with Google Earth street view imagery.

The number of access points will be recorded per leg as indicated below.

- NumberAccess_N = Number of access points on the northern leg.
- NumberAccess_E = Number of access points on the eastern leg.
- NumberAccess _S = Number of access points on the southern leg.
- NumberAccess _W = Number of access points on the western leg.

If one of the legs above does not exist at the roundabout, enter "0" for the leg.

Chapter 3: Speed Data

3.1 INTRODUCTION

This chapter presents guidance for gathering speed data collected at roundabout sites for Project 17-70. These data are summarized in Table B3-1 alongside the anticipated source of the data.

Table B3-1 Summary of Speed Data Attributes

Data Attribute	Anticipated Source and Tools	Relevant Section of Chapter
Posted Speed per Leg (mph)	Google Earth Street View	3.2
Entering Vehicle Speed		
Vehicle Path Construction and Radii Measurements	Google Earth Aerial Imagery and NCHRP Report 672 Methodology	3.3.1
Calculations to Convert Radii into Entering Vehicle Speed (mph)	NCHRP Report 672 Methodology	3.3.2
Circulating Vehicle Speed		
Vehicle Path Construction and Radii Measurements	Google Earth Aerial Imagery and NCHRP Report 672 Methodology	3.3.1
Calculations to Convert Radii into Entering Vehicle Speed (mph)	NCHRP Report 672 Methodology	3.3.2

3.2 POSTED SPEED

The posted speed for each leg to the roundabout site was gathered using Google Earth street view. On each leg to the roundabout, use Google Earth street view to drive the leg of the roundabout in the direction of a vehicle approaching the roundabout (i.e., identify upstream posted speed limit ideally within 0.5 miles of the roundabout). Record the posted (regulatory) speed in miles per hour (mph) as described below. Advisory speed placards (posted on yellow warning signs) should not be recorded.

- PostedSpeed_N = Posted speed for the northern leg.
- PostedSpeed_E = Posted speed for the eastern leg.
- PostedSpeed_S = Posted speed for the southern leg.
- PostedSpeed_W = Posted speed for the western leg.

If the posted speed is not found on one of the legs via Google Earth street view, enter "0" for the applicable leg. If a leg does not exist, enter "0" for that data.

Use the variable SpeedType_X (where X is N, E, S, or W depending on the leg) to record the type of posted speed data recorded. Where, SpeedType_X = "U" if posted speed was not found; "P" if posted speed was identified within 0.5 mile of the roundabout site where the distance is measured from the center of the roundabout intersection; or "E" if the posted speed was estimated based on the characteristics of the roadway. A posted speed may be estimated for a leg if the posted speed was found on other legs of the roundabout making it reasonable to assume that an adjacent leg with similar characteristics would have the same posted speed.

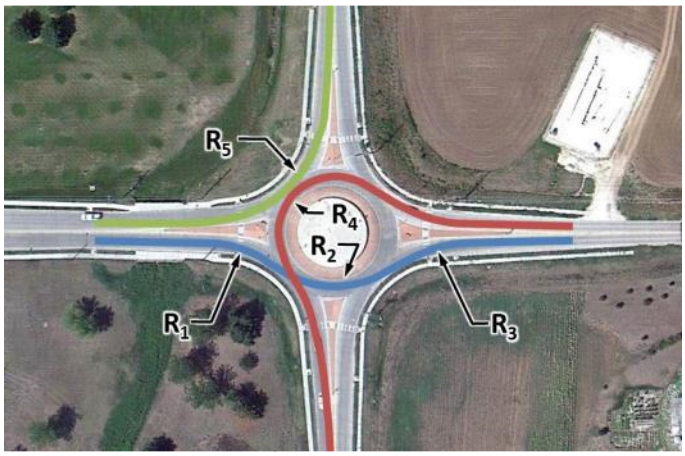
3.3 ENTERING AND CIRCULATING VEHICLE SPEED

Entering and circulating vehicle speed was estimated using NCHRP Report 672 *Roundabouts: An Informational Guide, Second Edition* Fastest Path methodology.The Fastest Path methodology is used by designers to estimate the theoretical maximum entry, circulating and exit speeds for motorists traveling through the roundabout. The methodology is sensitive to the roundabout specific geometric elements like ICD and approach curvature which are used to create deflection to help slow vehicle speeds. Section 3.3.1 describes the methodology that used to consistently draw the fastest paths for vehicles traveling through a roundabout and then to measure the corresponding radii of curves within the path. Section 3.3.2 describes the methodology that used to convert the curve radii to an estimated entering vehicle speed in miles per hour (mph).

3.3.1 Vehicle Path Construction and Radii Measurement

The fastest path is drawn for a motorist traveling through the entry, around the central island and out an exit. In designing a roundabout, the path is drawn for each vehicle movement on each leg. For Project 17-70, to estimate entering and circulating vehicle speed on each leg, a path will be developed on each leg for a vehicle traveling through the roundabout and a vehicle turning right at the roundabout. Figure B3-1 illustrates the different vehicle movement paths through a roundabout. The blue path (with the R_1, R_2, and R_3 labels), red path (with R_4 path labeled) and the green path (with the R_5 label) will be drawn per leg as part of the data collection effort for Project 17-70.

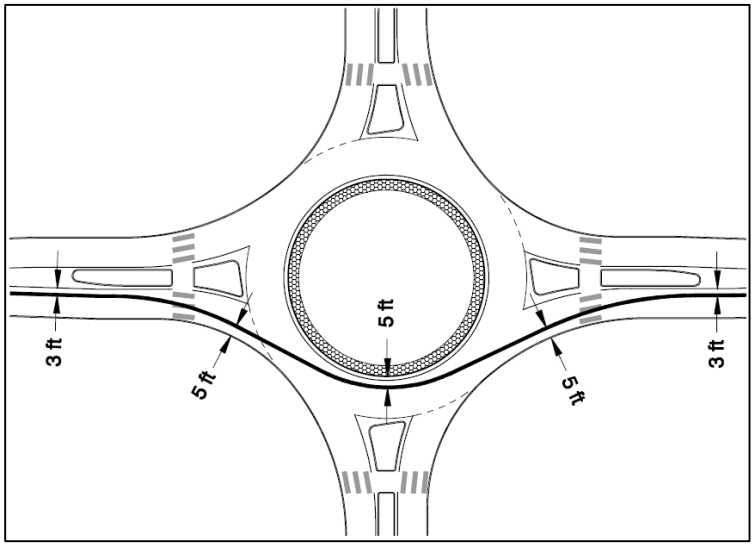
Figure B3-1. Fastest Path Concept



Source: Kansas Roundabout Guide, Second Edition (Exhibit 6-2)

Section 6.7.1 Fastest Path of NCHRP Report 672 provides detailed instructions for drawing the fastest vehicle paths for all movements at a roundabout, including guidance on where to draw paths in relation to curbs and paint lines. Fastest paths should be drawn with an offset of 5 feet from curbs and centerline stripes and an offset of 3 feet from other stripes (such as pavement markings denoting a striped median area). Figure B3-2 illustrates the fastest vehicle path through a single-lane roundabout. As seen in figure, the fastest path is drawn to maintain the appropriate offsets from curbs and striping and includes short lengths of tangents between consecutive curbs to account for the time it takes a driver to turn the steering wheel.

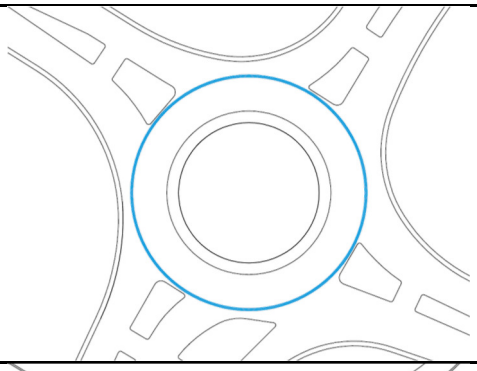
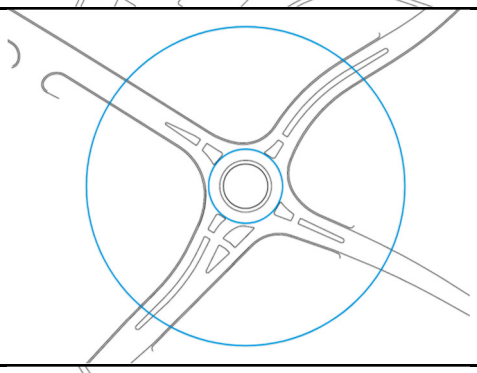
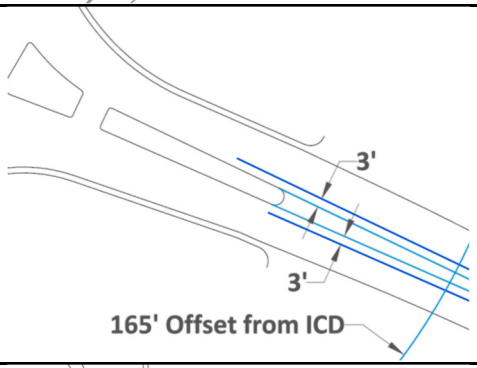
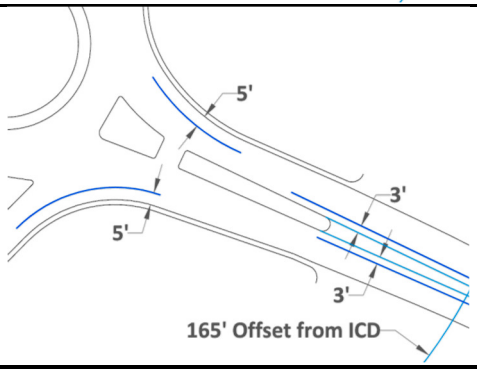
Figure B3-2 Fastest Path Example of Through Movement



Source: Kansas Roundabout Guide, Section Edition (Exhibit 6-3, Adapted from NCHRP Report 672, Exhibit 6-48*)*

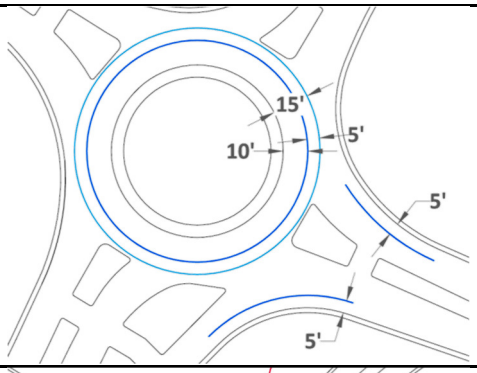
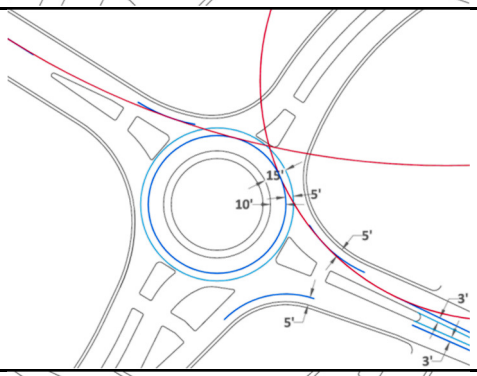
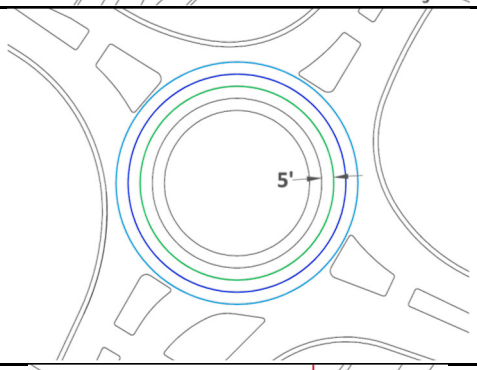
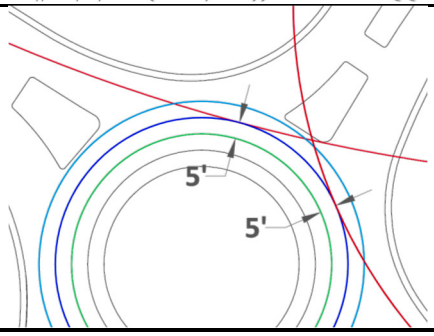
For consistency in Project 17-70 data collection, fastest paths should be drawn using AutoCAD software following the process described in Table B3-2.

Table B3-2. Steps for Vehicle Path Construction

Steps	Illustration of Steps
1. Locate the roundabout using Google Earth Pro. Zoom in to 1:100 scale. Save image at high resolution (1488X944). 2. Insert the image into AutoCAD model space at 43.2943 scale (this makes the image full scale; the 100ft scale bar becomes 100ft in model space). 3. Draw the ICD (see Section 1.14 for additional Guidance).	
4. Offset a circle 165 feet beyond the ICD.	
5. Approximately 165 feet beyond the ICD, draw lines several car lengths in length that are offset 5 feet from curbs and centerline stripes or 3 feet from other stripes, on both the entries and exits. In the example to the right, the splitter island does not extend past the 165-foot circle, so only a 3-foot offset is used.	
6. Draw an arc that is offset 5 feet from the outside entry- and exit-curve curbs.	

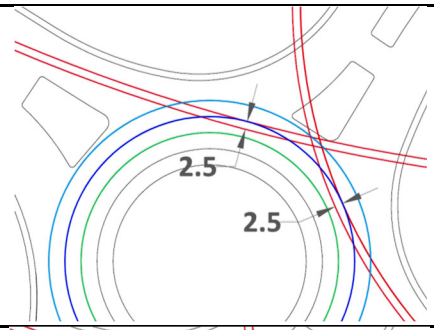
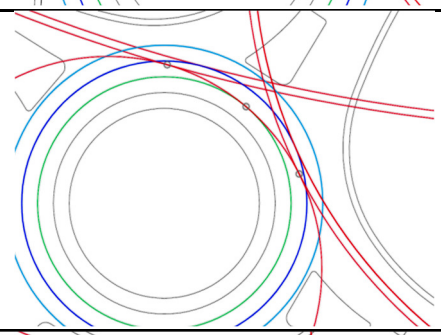
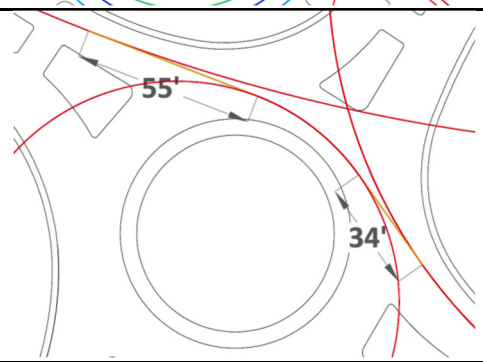
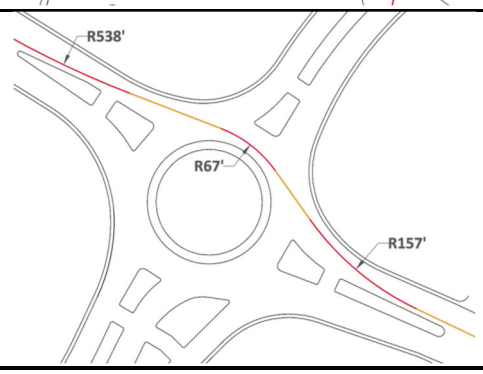
Source: Adapted from Kansas Roundabout Guide, Second Edition (Exhibit 6-4)

Table B3-2 continued. Steps for Vehicle Path Construction

Steps	Illustration of Steps
7. Within the circulatory roadway, draw a circle that is approximately two thirds of the distance from the inside edge of the circulatory roadway to the outside edge. For example, if the circulatory roadway is 15 feet wide, the circle would be offset 10 feet from the truck apron/inside of circulatory roadway and 5 feet from the outside of the circulatory roadway or ICD.	
8. For both the entry and exit, draw a three-point circle. The three points are tangent to the lines created in steps 3, 4, and 5. This creates the R1 and R3 curves.	
9. Draw a circle that is offset 5 feet beyond the truck apron.	
10. Measure the shortest distance between the truck apron offset (circle created in step 9) and the R1 and R3 curves (circles created in step 8).	

Source: Adapted from Kansas Roundabout Guide, Second Edition (Exhibit 6-4)

Table B3-2 continued. Steps for Vehicle Path Construction

Steps	Illustration of Steps
11. Draw circles offset from the R1 and R3 curves that are offset half of the distance measured in step 10.	
12. Draw a three-point curve with points tangent to the truck apron offset (circle created in step 9) and the circles created in step 11. This creates the R2 curve.	
13. Draw two lines that are tangent to the R2 curve (created in step 12) and the R1 and R3 curves (created in step 8). Check that the lines are at least 30 feet long. Clean up the construction lines.	
14. Trim the R1, R2 and R3 curves and measure the R1, R2 and R3 radii	

Source: Adapted from Kansas Roundabout Guide, Second Edition (Exhibit 6-4)

The R_4 measurement representing the radius of the left-turn movement of a vehicle at a roundabout is the radius of a circle offset 5 feet from the curb of the central island. This is often the slowest movement through a roundabout.

Constructing the fastest path for the right-turn movement (measurement R_5) is not as straightforward as the construction of R_1, R_2, and R_3 paths. In constructing the R_5 path, remember the basic principles associated with the construction of fastest-path curves:

- The fastest path curves are the smoothest, flattest path possible for a single vehicle.
- The fastest path ignores all lane markings (with the exception of lane markings separating opposing traffic).
- The fastest path will either consist of back-to-back spiral curves, or back-to-back curves with a short tangent section.
- The fastest path is drawn assuming the centerline of the vehicle will not get closer than the following distances to the particular geometric features:
    - 5 ft from a concrete curb
    - 5 ft from a roadway centerline
    - 3 ft from a roadway edge line (or edge of pavement if no edge line is present)
    - 3 feet from the striped extension of splitter island

The R_5 fastest path results in the largest possible radius that can be developed while adhering to the principles and offsets identified above.

The construction of an R_5 fastest path radius is typically done through a trial and error method. When using AutoCAD software, the offsets identified above are created, and a three-point arc is developed connecting three tangent points on the offsets. If the curve infringes on one of the offsets, the fastest path radius is redrawn, typically using the offset that was infringed upon as one of the tangents for the new three-point curve. Occasionally, the geometry of the roundabout does not allow the development of a logical three-point arc. In this case, a manual method can be used, where radii are drawn in five-foot increments until the largest radius adhering to the principles above is found.

The radius measurements should be recorded as described below – measurements should be entered in feet and rounded to the nearest integer.

- For the northern leg of the roundabout:
    - R1_N = Enter the radius for the R_1 curve in the fastest path.
    - R2_N = Enter the radius for the R_2 curve in the fastest path.
    - R3_N = Enter the radius for the R_3 curve in the fastest path.
    - R4_N = Enter the radius for the R_4 curve in the fastest path.
    - R5_N = Enter the radius for the R_5 curve in the fastest path.
- For the eastern leg at the roundabout:
    - R1_E = Enter the radius for the R_1 curve in the fastest path.
    - R2_E = Enter the radius for the R_2 curve in the fastest path.
    - R3_E = Enter the radius for the R_3 curve in the fastest path.
    - R4_E = Enter the radius for the R_4 curve in the fastest path.

    - R5_E = Enter the radius for the R_5 curve in the fastest path.
- For the southern leg at the roundabout:
    - R1_S = Enter the radius for the R_1 curve in the fastest path.
    - R2_S = Enter the radius for the R_2 curve in the fastest path.
    - R3_S = Enter the radius for the R_3 curve in the fastest path.
    - R4_S = Enter the radius for the R_4 curve in the fastest path.
    - R5_S = Enter the radius for the R_5 curve in the fastest path.
- For the western leg at the roundabout:
    - R1_W = Enter the radius for the R_1 curve in the fastest path.
    - R2_W = Enter the radius for the R_2 curve in the fastest path.
    - R3_W = Enter the radius for the R_3 curve in the fastest path.
    - R4_W = Enter the radius for the R_4 curve in the fastest path.
    - R5_W = Enter the radius for the R_5 curve in the fastest path.

If a leg does not exist, enter "0" for that data entry.

3.3.2 Converting Radii to Entering Vehicle Speed

The correlation between the radii of horizontal curvature and travel speed is documented in AASHTO's *Policy on Geometric Design of Streets and Highways*. "Both superelevation and the side friction factor affect the speed of a vehicle. Side friction varies with vehicle speed and can be determined in accordance with AASHTO guidelines. The most common superelevation values encountered are +0.02 and −0.02, corresponding to a 2% cross slope"(NCHRP Report 672). Equation 2-1 and Equation 2-2 (from NCHRP Report 672) provide a simplified relationship between speed and radius for these two common superelevation rates that incorporates the AASHTO relationship and side friction factors. The speed-radius relationship is displayed graphically in Figure B3-3. This graph can be used to determine the speed associated with the theoretical fastest paths through the roundabout.

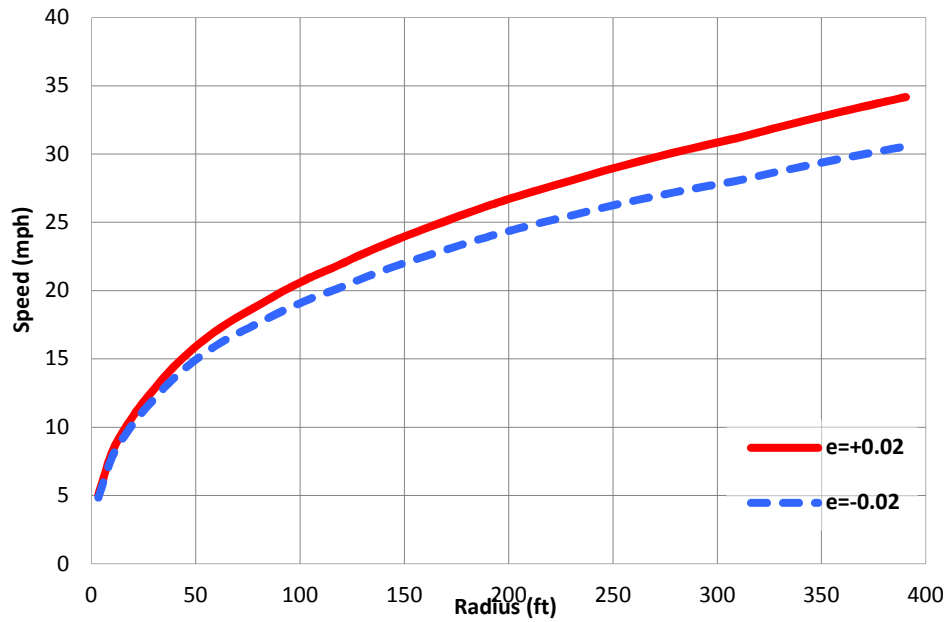
Equation B3-1: $V = 3.4415R^{0.3861}$, for $e = +0.02$

Equation B3-2: $V = 3.4614R^{0.3673}$, for $e = -0.02$

Where,

$V =$ predicted speed, mph;
$R =$ radius of curve, ft; and
$e =$ superelevation, ft/ft.

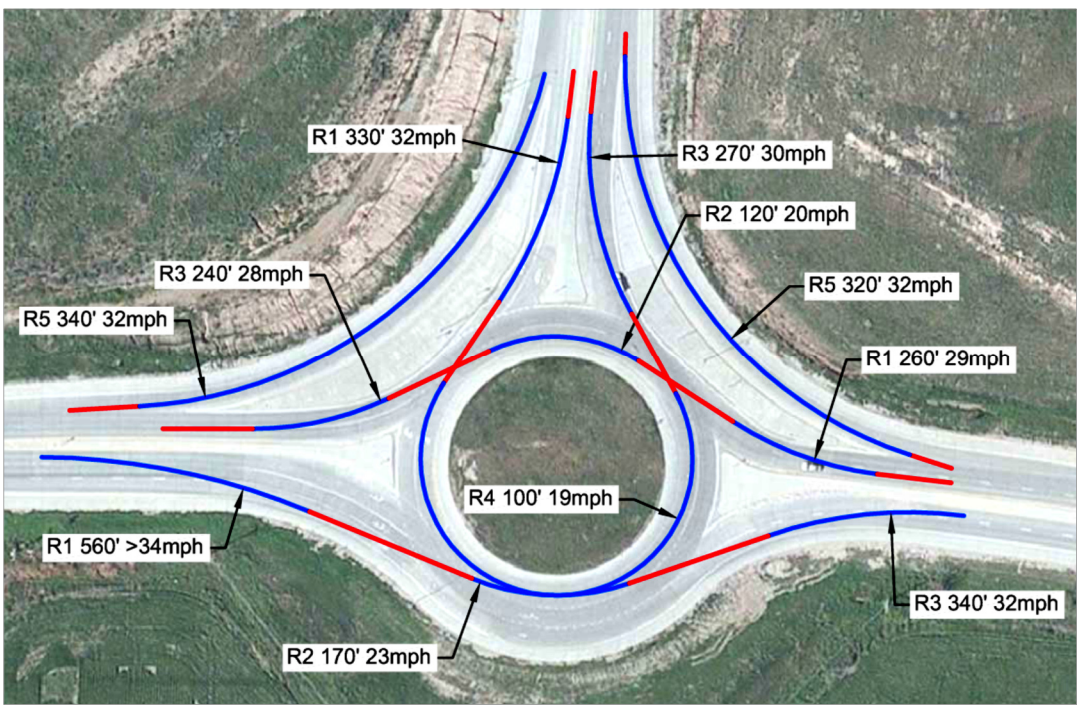
Figure B3-3. Speed-Radius Relationship



Source: Kansas Roundabout Guide, Second Edition (Exhibit 6-5; Adapted from NCHRP Report 672, Exhibit 6-52*)*

Figure B3-4 is an annotated example of converting the radii measured in the theoretical fastest paths to vehicle speed in miles per hour.

Figure B3-4. Fastest Path to Vehicle Speed Example



Source: Kansas Roundabout Guide, Second Edition (Exhibit 6-6).

In Project 17-70, the process described was used to convert the fastest path radii measured per leg to vehicle entering and circulating speeds. Exceptions for unusual geometric cases may be needed.

- Theoretical Maximum Entering Vehicle Speed, V_{emx}
    - Convert R_1 measurement to vehicle speed (V_1) using Equation B3-1 above.
    - Convert R_5 measurement to vehicle speed (V_5) using Equation B3-1 above.
    - If $V_1 > V_5$, $V_{emx} = V_1$.
    - If $V_5 > V_1$, $V_{emx} = V_5$.
- Theoretical Maximum Circulating Vehicle Speed
    - Convert R_4 measurement to vehicle speed (V_4) using Equation B3-2 above.

Record the entering and circulating vehicle speeds as described below – speeds should be recorded in mph to the nearest integer.

- For the northern leg at the roundabout:
    - V1_N = Vehicle speed, V_1 calculated based on R_1 measurement.
    - VCirc_N = Vehicle speed, V_4 calculated based on R_4 measurement.
    - V5_N = Vehicle speed, V_5 calculated based on R_5 measurement.
    - VEntMax_N = Maximum theoretical entering speed (V_{emx}).
- For the eastern leg at the roundabout:
    - V1_E = Vehicle speed, V_1 calculated based on R_1 measurement.
    - VCirc _E = Vehicle speed, V_4 calculated based on R_4 measurement.
    - V5_E = Vehicle speed, V_5 calculated based on R_5 measurement.
    - VEntMax_E = Maximum theoretical entering speed (V_{emx}).
- For the southern leg at the roundabout:
    - V1_S = Vehicle speed, V_1 calculated based on R_1 measurement.
    - VCirc _S = Vehicle speed, V_4 calculated based on R_4 measurement.
    - V5_S = Vehicle speed, V_5 calculated based on R_5 measurement.
    - VEntMax_S = Enter maximum theoretical entering speed (V_{emx}).
- For the western leg at the roundabout:
    - V1_W = Vehicle speed, V_1 calculated based on R_1 measurement.
    - VCirc _W = Vehicle speed, V_4 calculated based on R_4 measurement.
    - V5_W = Vehicle speed, V_5 calculated based on R_5 measurement.
    - VEntMax_W = Maximum theoretical entering speed (V_{emx}).

If a leg does not exist, enter "0" for that data entry.

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INTRODUCTION

This document describes the database assembled for roundabout safety prediction model development. Specifically, it describes the individual data attributes in the database. Procedures are described in a separate document for collecting (i.e., acquiring or measuring) these data from various sources and processing (i.e., combining or reducing) them to obtain the data corresponding to input variables considered in the development of a predictive model.

This document is divided into two sections. The first section describes the data needed for the development of intersection-level safety prediction models. Many of these data will also be used for the development of approach-level safety prediction models. The second section describes the additional data needed for the development of approach-level safety prediction models. The data in the second section will only be recorded for a subset of sites.

The data are recorded in an Excel workbook. There are two worksheets in this workbook. One worksheet is used describe the road inventory, speed, and traffic data associated with each roundabout. One row of data in this worksheet corresponds to the data for one roundabout. The second worksheet is used to describe the crash data associated with each site. One row of data in this worksheet corresponds to one crash record. A unique identification number is used to link the crash data to the roundabout at which the crash occurred. A second identifier is used to link the crash data to a specific leg for those roundabouts selected for developing leg-level SPFs. In both worksheets, one column is allocated to each data attribute.

INTERSECTION-LEVEL DATA

General Guidance

Do not make any calculations by hand or with a hand calculator. Enter every measured variable in the Excel sheet. If calculations are needed to convert the measured variables to the needed quantity, then enter the equation in the Excel cell associated with the variable of interest.

Intersection Boundary

The leg numbering convention is identified in Figure 1.

Figure 1. Leg naming convention: (figure needs to be drafted)

```
  N          N    E        North is up in both configurations
W + E          ×
  S          W    S
```

Definitions

Roundabout-Related Crash. A roundabout crash satisfies one or more of the following criteria:

1. Indicated as occurring at the intersection on the crash report,
2. Occurring within 250 feet of the yield line and indicated as "intersection-related" on the crash report, or
3. If crash data does <u>not include</u> an "intersection-related" category, a crash occurring on a roundabout leg between the yield line and a point approximately 250 feet back from the yield line – excluding crashes that are driveway related where these can be identified from the crash reports.

Area Type. In the HSM, the definition of "urban" and "rural" areas is based on Federal Highway Administration (FHWA) guidelines which classify "urban" areas as places inside urban boundaries where the population is greater than 5,000 persons. "Rural" areas are defined as places outside urban areas with populations greater than 5,000 persons. The HSM uses the term "suburban" to refer to outlying portions of an urban area; the predictive method does not distinguish between urban and suburban portions of a developed area.

Evaluation Period. The time period that starts with the opening date of the roundabout (but no earlier than 2008) and extends for at least three years (but no more than five years and no later than 2013). One exception to this are the subset of sites used to evaluate the effect of driver learning curve on roundabout crash performance. Each study site is confirmed to have had no changes to its geometry during the evaluation period. For example, a site with an opening date of January 2007 would have January 2007 as the starting date for the evaluation period. The ending date of the evaluation period for this site would be December 2011 (i.e., the evaluation period is 1/2007-12/2007, 1/2008-12/2008, 1/2009-12/2009, 1/2010-12/2010, 1/2011-12/2011).

A site with an opening date of March 2009 would have March 2009 as the starting date for the evaluation period. The ending date of the evaluation period for this site would be February 2013 (i.e., the evaluation period is 3/2009-2/2010, 3/2010-2/2011, 3/2011-2/2012, 3/2012-2/2013). If the research team is able to identify a distinct time period for a learning curve for motorists at new roundabouts, the evaluation period will be adjusted to exclude the crashes recorded during the learning curve period.

DESCRIPTIVE DATA

Problem Flag.

Variable name: ProblemFlag
Format: XX
Type: integer
Variable values:

This variable contains a one-digit integer that describes the acceptability of the subject site for data collection. If a number greater than 1 is recorded, do not populate the remaining data fields for this site.

1. no problems found.
2. roundabout under construction during the evaluation period.
3. aerial photo of acceptable quality is not available within the evaluation period.
4. distance to the nearest intersection with all-way stop control, signal control, or roundabout operation is less than 700 ft (measured from center of subject roundabout to center of intersection)
5. five or more legs at the roundabout.
6. three or more legs serving only one travel direction.
7. three or more circulating lanes <u>at any point</u> within the roundabout.
8. three or more entering lanes on any leg.
9. multilane roundabout with multiple circulatory lanes but no pavement markings within the circulatory roadway.
10. oblong shaped roundabout (i.e., inscribed circle diameter measurements vary by more than 15%)
11. one or more approaches has a type of control for the through movement that is other than Yield control (i.e., study sites should only include those roundabouts where the through movement on each approach is Yield controlled).
12. roundabout sites that are ramp terminal intersections in a tear-drop or dog-bone configuration.
13. other problem(s) which are described in the Notes column of the database.

Opening Date

Variable name: OpenDate
Format: YYYY-MM-DD
Type: character
Variable values:
This variable identifies the date that the site was opened to traffic and all evidence of construction removed. The format is:
DD – number of day that the site was opened (e.g., 09 for 9th day of month). Use "00" if day is unknown or not known with confidence.
MM – number of month that site was opened (e.g., '01' – January, '02' – February, etc.). Use "00" if the month is unknown or not known with confidence.
YYYY – year that site was opened. Use "0000" if the year is unknown or not known with confidence.

Starting Date of Evaluation Period

Variable name: StartDate
Format: YYYY-MM
Type: character
Variable values:
This variable identifies the starting date of the evaluation period. The format is:
MM – number of month (e.g., '01' – January, '02' – February, etc.).
YYYY – year.

Ending Date of Evaluation Period

Variable name: EndDate
Format: YYYY-MM
Type: character
Variable values:
This variable identifies the ending date of the evaluation period. The format is:
MM – number of month (e.g., '01' – January, '02' – February, etc.).
YYYY – year.

Roundabout Identification Number

Variable name: Link_ID.
Format. SSYYYY
Type: character, upper case if alpha
Variable values:
A variable that uniquely identifies each site. It is also used in the crash data base. The format is:
SS – U.S. Post Office two-letter abbreviation for a state.
YYYY – four digit number, preceded by "0" if ≤ 999, "00" if ≤ 99, and "000" if ≤ 9

Example: Site number 2 in Oregon. Link_ID = 'OR0002'

Street Name or Route Number

Variable name: Name_Leg_X.

Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

Name_Leg_N, Name_Leg_E, Name_Leg_S, Name_Leg_W

Format:

If route is numbered, format: NN-YYYY

If route is not numbered, format: ZZZZZZZZ, eight characters or less

Type: character, upper case if alpha

Variable values:

A. If route is numbered,

NN is the route designation for leg X.

NN - US, SH, IH, CO, FM;

where SH = state highway, IH = interstate highway, CO = county road, FM = farm-to-market road.

YYYY is the signed route number for leg X. This number is preceded by "0" if ≤ 999, "00" if ≤ 99, and "000" if ≤ 9

B. If route is not numbered,

ZZZZZZZZ is the first eight characters of the street name for leg X.

C. If leg does not exist: leave blank

This variable is also used in the Crash Data worksheet.

Milepost of Intersection Center

This variable is used with the crash data if that data are assigned a location by linear reference system. This variable should be defined when the crash database is populated.

Variable name: Milepost_Int_X

Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

Milepost_Int_N, Milepost_Int_E, Milepost_Int_S, Milepost_Int_W

It is likely that the variable value will be the same for legs N and S, and the same for legs E and W. However, the values may be different if a route "turns" direction at the roundabout.

The specified milepost for Milepost_Int_X is in reference to the alignment identified using the Name_Leg_X variable for a common leg number.

Format: varies, 1000th of a mile precision (e.g., 12.345) when such precision is available in the data provided.

Type: real

Variable values:

The milepost location of the intersection's center point, as defined by the milepost sequence established for leg X (i.e., the point where the center line of the crossing alignment intersects with that for leg X). The

unit of this variable is "miles." Its value will be determined by the operating agency based on the linear referencing system used to locate crashes.

Use "-1.000" if the agency operating the subject site does not locate crashes using a linear referencing system. Use "-2.000" if leg does not exist.

City or County Name

Variable name: City_County
Format: YYYYYYYY
Type: character, upper case if alpha
Variable values:
The first eight letters of the city in which the site is located. If the site is not located in a city, then use the first eight letters of the county in which the site is located.

Area Type

Variable name: AreaType
Format: Y
Type: character, upper case
Variable values:
U – Urban/Suburban
R – Rural

This variable generally describes the environment in the site is located. The definitions of urban/suburban, and rural are provided in the General Guidance section.

Latitude and Longitude of Center of Intersection

Variable name: Lat_Long
Format: varies, DDD°MM'SS.SS"N_ DDD°MM'SS.SS"W
First variable in format is latitude in degrees, minutes, seconds (to 100th of degree), and cardinal direction. Second variable is longitude in degrees, minutes, seconds (to 100th of degree), and cardinal direction. The two variables are separated by a space.

Type: text, upper case if alpha
Variable values:
Geocoordinates of the center of the intersection. Obtained from the Placemark feature in Google Earth.

Ramp Terminal Intersection

Variable name: RampTerm
Format: X
Type: character
Variable values:
Y – One or more of the roundabout legs is a ramp to or from a controlled access facility.
N – otherwise.

ROAD INVENTORY DATA

All road inventory data describe the site geometry during the evaluation period, as defined previously in the section titled General Guidance.

Number of Legs at the Roundabout

Variable name: NumberLegs
Format: X
Type: integer
Variable values:
3 – three legs at the roundabout
4 – four legs at the roundabout.

Number of Circulating Lanes

Variable name: CirculatingLanes
Format: X
Type: integer
Variable values:
1 – one circulating lane throughout the roundabout (no variation in lane count).
2 – two circulating lanes throughout the roundabout (no variation in lane count) (record "N" for variable Hybrid).
2 – number of circulating lanes varies (record "Y" for variable Hybrid).

Variable name: Hybrid
Format: X
Type: character
Variable values:
Y – combination of one and two circulating lanes within the roundabout.
N – consistent number of circulating lanes within the roundabout.

Variable name: CirculatingLanes_X
Format: X
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.
CirculatingLanes_N, CirculatingLanes_E, CirculatingLanes_S, CirculatingLanes_W
0 – leg does not exist
1 – one circulating lane in front of entry lanes at leg X
2 – two circulating lanes in front of entry lanes at leg X

Number of Entering Lanes

Variable name: EnteringLanes_X
Format: X
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.
EnteringLanes_N, EnteringLanes_E, EnteringLanes_S, EnteringLanes_W
0 – no entering lanes on leg X (or leg X does not exist).
1 – one entering lane on leg X (excluding any right-turn by-pass lane that may be present).
2 – two entering lanes on leg X (excluding any right-turn by-pass lane that may be present).

Number of Exiting Lanes

Variable name: ExitingLanes_X
Format: X
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

ExitingLanes_N, ExitingLanes_E, ExitingLanes_S, ExitingLanes_W

0 – no exiting lanes on leg X (or leg X does not exist).
1 – one exiting lane on leg X.
2 – two exiting lanes on leg X.

Presence of Right-Turn By-Pass Lane

Variable name: Bypass_X
Format: X
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

Bypass_N, Bypass_E, Bypass_S, Bypass_W

0 – no right-turn bypass lane on leg X (or leg X does not exist).
1 – one right-turn bypass lane on leg X.

Presence of Lane-Use Pavement Marking Arrows

Variable name: LaneMarkings_X
Format: X
Type: character
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

LaneMarkings_N, LaneMarkings_E, LaneMarkings_S, LaneMarkings_W

B – markings not present on leg X (or leg X does not exist).
N – markings are present on leg X and use the "normal" arrow form.
F – markings are present on leg X and use the "fish hook" arrow form.
O – markings are present on leg X but do not use the "normal" or "fish hook" arrow form.
V – markings vary (in type or their presence) over the duration of the evaluation period.
X – leg does not exist.

Number of Luminaires within 250 Feet of Roundabout

Variable name: Luminaires_X
Format: X
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

Luminaires_N, Luminaires _E, Luminaires _S, Luminaires _W
0 – no luminaireson leg X (or leg X does not exist).
1..9 – number of luminaires on leg X.

Entry Width

Variable name: EntryWidth_X
Format: XX, 1 foot precision (e.g, 22)
Type: real
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.
EntryWidth_N, EntryWidth_E, EntryWidth_S, EntryWidth_W
0 – leg X does not exist, or does not have entry lanes (just exit lanes).

Angle to Next Leg

Variable name: Angle_X
Format: XXX, 1 degree precision (e.g., 22)
Type: real
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.
Angle_N, Angle_E, Angle_S, Angle_W
Angle is measured in counterclockwise direction from subject leg centerline to next leg centerline.
0 – leg X does not exist.

Inscribed Circle Diameter

Variable name: ICD
Format: XXX, one foot precision (e.g., 102) when such precision is available in the data provided.
Type: integer
Variable values:
The effective average diameter of the outside edge of traveled way for the outside circulating lane.

Circulating Width

Variable name: CirculatingWidth_X
Format: XX, 1 foot precision (e.g, 22).
Type: real
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.
CirculatingWidth_N, CirculatingWidth_E, CirculatingWidth_S, CirculatingWidth_W
0 – leg X does not exist.

Approach Half-Width

Variable name: HalfWidth_X
Format: XX, 1 foot precision (e.g, 22)

Type: real
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

HalfWidth_N, HalfWidth_E, HalfWidth_S, HalfWidth_W

0 – leg X does not exist, or does not have entry lanes (just exit lanes).

Access Points within 250 Feet of Roundabout

Variable name: NumberAccess_X
Format: X
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

NumberAccess_N, NumberAccess_E, NumberAccess_S, NumberAccess_W

0, …, 4 – number of access points on leg X. (Use "0" if leg does not exist).

This variable represents a two-way count along the subject leg between the yield line and a point approximately 250 feet back from the yield line. That is, it represents the count of individual driveway along the entry lanes and exit lanes. Each leg is counted once.

SPEED DATA

All speed data describe the site characteristics during the evaluation period, as defined previously in the section titled General Guidance.

Posted Speed

Variable name: PostedSpeed_X
Format: XX, speed limit in miles per hour.
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

PostedSpeed_N, PostedSpeed_E, PostedSpeed_S, PostedSpeed_W

Posted speed applicable to vehicles traveling <u>toward</u> the subject roundabout.

0 – unknown speed limit on approach associated with leg X (or leg X does not exist).

25,…, 65 – speed limit on approach associated with leg X.

Speed Limit Type

Variable name: SpeedType_X
Format: X
Type: character
Variable values
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

SpeedType_N, SpeedType_E, SpeedType_S, SpeedType_W

Source of data describing the speed limit for vehicles traveling toward the subject roundabout.
U – unknown. Used if "0" is used for PostedSpeed variable for approach on leg X.
P – posted speed limit sign on leg X within 0.5 miles of the subject roundabout.
E – an estimate of the posted speed based on other sources of information (e.g., basic speed law, agency feedback, posted speed limit in direction of travel away from the subject roundabout, judgment indicates that a posted speed more than 0.5 miles from the roundabout is still applicable).

TRAFFIC DATA

All traffic data describe the site characteristics during the evaluation period, as defined previously in the section titled General Guidance.

AADT's represent one calendar year (i.e., January to December). If an evaluation period starts in any portion of a calendar year, then the AADT for that year must be provided. For this reason, the allowance for an evaluation period up to five years in duration will requires the specification of up to six AADT values (to account for the case where the evaluation period starts after January).

Each AADT described in this section represents the total flow on the associated leg. If the leg supports travel in two directions, then the AADT is represents a two-way total flow.

AADT for Year 1 of Evaluation Period

Variable name: AADT_Year1_X
Format: XXXXX, nearest vehicle precision (e.g., 15023) when such precision is available in the data provided.
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

AADT_Year1_N, AADT_Year1_E, AADT_Year1_S, AADT_Year1_W

0 – leg X does not exist or Year 1 AADT for leg X is unknown.
X – value of Year 1 AADT for leg X.

Year 1 Date

Variable name: Year1_X
Format: YYYY
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg, as identified in Figure 1.

Year1_N, Year1_E, Year1_S, Year1_W

This variable identifies the year of the corresponding AADT. The format is:
0 – leg X does not exist or Year 1 AADT for leg X is unknown.
Y – year.

AADT for Year 2 of Evaluation Period

AADT_Year2_N, AADT_Year2_E, AADT_Year2_S, AADT_Year2_W

Year 2 Date

Year2_N, Year2_E, Year2_S, Year2_W

AADT for Year 3 of Evaluation Period

AADT_Year3_N, AADT_Year3_E, AADT_Year3_S, AADT_Year3_W

Year 3 Date

Year3_N, Year3_E, Year3_S, Year3_W

AADT for Year 4 of Evaluation Period

AADT_Year4_N, AADT_Year4_S, AADT_Year4_E, AADT_Year4_W

Year 4 Date

Year4_N, Year4_E, Year4_S, Year4_W

AADT for Year 5 of Evaluation Period

AADT_Year5_N, AADT_Year5_E, AADT_Year5_S, AADT_Year5_W

Year 5 Date

Year5_N, Year5_E, Year5_S, Year5_W

AADT for Year 6 of Evaluation Period

AADT_Year6_N, AADT_Year6_E, AADT_Year6_S, AADT_Year6_W

Year 6 Date

Year6_N, Year6_E, Year6_S, Year6_W

CRASH DATA

All crash data describe the crash characteristics during the evaluation period, as defined previously in the section titled General Guidance. Each observation in this database describes one crash. The original electronic crash data are to be kept in separate spreadsheets. The crash data should be entered into the safety database using the variables below. The original crash variables will be used to meet our variable definitions. The exact methods may differe between jurisdictions depending on how their crash data are coded.

Roundabout Identification Number

Variable name: Link_ID.
Format. SSYYYY
Type: character, upper case if alpha
Variable values:
A variable that uniquely identifies each site. It is also used in the combined road inventory, speed, and traffic database. The format is:
SS – U.S. Post Office two-letter abbreviation for a state.
YYYY – four digit number, preceded by “0” if ≤ 999, “00” if ≤ 99, and “000” if ≤ 9

Example: Site number 2 in Oregon. Link_ID = 'OR0002'

Data Source

Variable name: CrashSource
Format: XX
Type: character
Variable values:
CI – city agency
CO – county agency
ST – state agency
HS – HSIS database from FHWA
OT – other agency.

Street Name or Route Number of Crash

Variable name: Name_Leg.
Format:
If route is numbered, format: NN-YYYY
If route is not numbered, format: ZZZZZZZZ

Type: character, upper case if alpha

Variable values:
A. If route is numbered,
NN is the route designation for leg X.
NN - US, SH, IH, CO, FM;
where SH = state highway, IH = interstate highway, CO = county road, FM = farm-to-market road.

YYYY is the signed route number for leg X. This number is preceded by "0" if ≤ 999, "00" if ≤ 99, and "000" if ≤ 9

B. If route is not numbered,
ZZZZZZZZ is the first eight characters of the street name for leg X.

C. If leg does not exist: leave blank

This variable describes the roadway on which the vehicles involved in the crash were traveling just before the crash. This variable is also used in the combined road inventory, speed, and traffic database.County of Crash

Variable name: CrashCounty
Format: XXXXXXXX, eight characters or less
Type: character
Variable values:
County in which the crash occurred.
X – name of county

Year of Crash

Variable name: CrashYear
Format: XXXX
Type: integer
Variable values:
This variable identifies the year in which the crash occurred. The format is:
X – year

Month of Crash

Variable name: CrashMonth
Format: XX
Type: integer
Variable values:
This variable identifies the month of year in which the crash occurred. The format is:
X – number of month (1, 2, …, 12)

Day of Crash

Variable name: CrashDay
Format: XX
Type: integer
Variable values:
This variable identifies the day of month in which the crash occurred. The format is:
X – number of day of month (1, 2, …, 31)

Hour of Crash

Variable name: CrashHour
Format: XX
Type: integer
Variable values:
This variable identifies the hour of day in which the crash occurred. The format is:
X – number of hour of day, military time (0=midnight, 1, …, 23)

Minute of Crash

Variable name: CrashMinute
Format: XX
Type: integer
Variable values:
This variable identifies the minute of the hour in which the crash occurred. The format is:
X – number of minute of hour (0, 1, …, 59)

Milepost of Crash

Milepost location of the first harmful event (e.g., collision, overturn, fire, cargo loss, etc.) associated with the crash. This variable is used if crashes are assigned a location by linear reference system. The specified milepost is in reference to the alignment identified using the Name_Leg_X variable.
Variable name: CrashMilepost

Format: varies, 1000th of a mile precision (e.g., 12.345) when such precision is available in the data provided.

Type: real

Variable values:

The unit of this variable is "miles." Its value will be determined by the operating agency based on the linear referencing system used to locate crashes.

Use "-1.000" if the agency operating the subject site does not locate crashes using a linear referencing system.

Distance to Intersection

This variable is used if the crash location is not specified using a linear referencing system.

Variable name: CrashDistance

Format: XXX, one foot precision (e.g., 102) when such precision is available in the data provided.

Type: integer

Variable values:

The distance between the reported point of collision and the center of the roundabout, as measured along the leg centerline. The center of the roundabout is determined by the crossing point of the centerlines of the two intersecting alignments. If the crash location is offset from the leg centerline, then the offset is ignored and the crash location is projected onto the leg centerline (along a line perpendicular to the centerline) for distance measurement.

Use -1 if the agency operating the subject site locates crashes using a linear referencing system and milepost data have been entered for the CrashMilepost variable.

Crash Location

Variable name: CrashLocation

Format: X

Type: integer

Variable values:

This variable describes the relationship of the crash to the roundabout. The value entered is obtained from the crash report, as determined by the person reporting the crash. Note that whether a crash is considered to be "roundabout-related" will be determined at a later time based on consideration of this variable and other variables in the crash database.

0 – other specified location.

1 – at intersection/roundabout.

2 – intersection/roundabout related.

3 – driveway related.

9 – location not specified.

Crash Type

Variable name: CrashType

Format: XX

Type: character

Variable values:

This variable describes the manner of collision. The value entered is obtained from the crash report, as determined by the person reporting the crash.

HO – head-on.

RA – right-angle.

RE – rear-end
SS – sideswipe same direction
OM – other multiple vehicle
AN – animal
FO – fixed object
OO – other object
PV – parked vehicle
PD – pedestrian
BK – bicycle
OS – other single vehicle

Crash Severity

Variable name: CrashSeverity
Format: X
Type: character
Variable values:
This variable indicates the most severe level of injury for all persons involved in the collision. The value entered is obtained from the crash report, as determined by the person reporting the crash.
K – fatal
A – incapacitating injury, severe visible injury, disabling injury
B – non-incapacitating injury, not severe visible injury, evident injury
C – possible injury, complaint of pain
O – property damage only.
U – unknown or unreported.

APPROACH-LEVEL DATA

The variables in this section will only be recorded for the subset of sites identified for developing approach-level SPFs.

For approach-level crash data, we will need to review the officer narrative and drawing on the crash reports.

Leg Level Crash Type

Variable name: CrashTypeLeg
Format: X
Type: character
Variable values:
This variable describes the type of crash for the approach-level SPFs. The value entered is obtained after reviewing the crash report narrative and drawing.

ENT/CIRC – The first harmful event involves one vehicle entering the roundabout and one vehicle circulating at the point of entry. The location of the crash should be coded to the leg from with the entering vehicle originates for the variable Leg ID X.

EXT/CIRC – The first harmful event involves one exiting the roundabout and one vehicle circulating at the point of exit. The location of the crash should be coded to the leg from with the exiting vehicle is departing the roundabout for the variable Leg ID X.

APP – The first harmful event involves one or more vehicles on the roundabout leg with at least one of the vehicles advancing towards the roundabout and prior to advancing beyond the yield line. The location of the crash should be coded to the leg from with the approaching vehicle is originating for the variable Leg ID X.

DWNSTRM – The first harmful event involves two or more vehicles, occuring on the leg (i.e. not within the roundabout), with all vehicles advancing away from the roundabout. The location of the crash should be coded to the leg on which the exiting vehicles are travelling for the variable Leg ID X.

OTHERCRASH – Any crash that does not fit into the above crash categories is coded as OTHERCRASH and Leg ID X is entered as 0.

Leg-Level Crash Location

Variable name: Leg_ID X.
Format. SSYYYYX
Type: character, upper case if alpha
Variable values:

Four variables are described by this attribute, one variable for each roundabout leg. The first part of the variable is the same as the Link_ID for the intersection-level data. Thus for the leg-level crash data one will add the leg identifier 'X' to the LINK ID variable. The letter "X" is used in the variable name to designate the subject roundabout leg number, as identified in Figure 1.

Leg ID_N, Leg ID_E, Leg ID_S, Leg ID_W

If the leg-level crash type is 'Other' then enter the value 0. Details on how to define a 'Other' crash are discussed for the variable CrashTypeLeg.

Example: Site number 2 in Oregon on the North leg. Link_ID = 'OR00002N'

General Note Regarding AADT per Movement per Leg

The methodology for estimating entering, exiting and circulating AADT is to be determined. It will be dependent on what data can be provided by jurisdictions. The research team will also review and consider the method from NCHRP Project 3-65 *Applying Roundabouts in the United States*.

Entering AADT for Years 1 to 6 of Evaluation Period

Variable name: Entering AADT_Year1_X
Format: XXXXX, nearest vehicle precision (e.g., 15023) when such precision is available in the data provided.
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg number, as identified in Figure 1. Data is entered separately for years 1 to 6.

Entering AADT_Year1_N, Entering AADT_Year1_E, Entering AADT_Year1_S, Entering AADT_Year1_W

0 – leg X does not exist or Year 1 Entering AADT for leg X is unknown.

X – value of Year 1 Entering AADT for leg X.

Exiting AADT for Years 1 to 6 of Evaluation Period

Variable name: Exiting AADT_Year1_X

Format: XXXXX, nearest vehicle precision (e.g., 15023) when such precision is available in the data provided.

Type: integer

Variable values:

Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg number, as identified in Figure 1. Data is entered separately for years 1 to 6.

Exiting AADT_Year1_N, Exiting AADT_Year1_E, Exiting AADT_Year1_S, Exiting AADT_Year1_W

0 – leg X does not exist or Year 1 Exiting AADT for leg X is unknown.

X – value of Year 1 Exiting AADT for leg X.

Circulating AADT for Years 1 to 6 of Evaluation Period

Variable name: Circulating AADT_Year1_X

Format: XXXXX, nearest vehicle precision (e.g., 15023) when such precision is available in the data provided.

Type: integer

Variable values:

Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg number, as identified in Figure 1. Data is entered separately for years 1 to 6.

Circulating AADT_Year1_N, Circulating AADT_Year1_E, Circulating AADT_Year1_S, Circulating AADT_Year1_W

0 – leg X does not exist or Year 1 Circulating AADT for leg X is unknown.

X – value of Year 1 Circulating AADT for leg X.

Entering Speed

Variable name: VEntMax_X

Format: XX, fastest estimated entering speed in mph using the fastest path methodology.

Type: integer

Variable values:

Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg number, as identified in Figure 1.

VEntMax_N, VEntMax _E, VEntMax _S, VEntMax _W

If a leg does not exist, enter "0" for that data entry.

Circulating Speed

Variable name: VCirc_X
Format: XX, fastest estimated circulating speed in mph using the fastest path methodology.
Type: integer
Variable values:
Four variables are described by this attribute, one variable for each roundabout leg. The letter "X" is used in the variable name to designate the subject roundabout leg number, as identified in Figure 1.

VCirc _N, VCirc _E, VCirc _S, VCirc _W

If a leg does not exist, enter "0" for that data entry.

Note that the measurement is for a vehicle from the specificed a leg making an left-turn movement through the roundabout. Therefore, the VCirc_N would be the estimated circulating speed for a vehicle making the southbound left-turn movement (i.e., the left-turn movement from the north leg).

Link ID	North Leg	East Leg	South Leg	West Leg	City	State
CA002	OldRiverRd_Leg_N	LakeportHopland175Rd_Leg_E	MainSt_Leg_S		Old Hopland	CA
CA004	CA89_Leg_N	HennessRd_Leg_E	CA89_Leg_S	DonnerPassRd_Leg_W	Truckee	CA
CA008	50thSt_Leg_N	PalmdaleBlvd_Leg_E	47thSt_Leg_S	CA138_Leg_W	Palmdale	CA
CA009	CA89_Leg_N	I80WBofframp_Leg_E	CA89_Leg_S	I80WBonramp_Leg_W	Truckee	CA
FL0001	NW23RDTE_Leg_N	NW31STAV_Leg_E	NW23RDTE_Leg_S	GLENSPRI_Leg_W	Gainesville	FL
FL0002	CAINBLVD_Leg_N	LAKESATB_Leg_E	CAINBLVD_Leg_S	LAKESATB_Leg_W	(unincorporated)	FL
FL0003	SE7THST_Leg_N	SE4THAVE_Leg_E	SE7THST_Leg_S	SE4THAVE_Leg_W	Gainesville	FL
FL0004		HOLLYWOO_Leg_E	DOOLITTL_Leg_S	HOLLYWOO_Leg_W	Fort Walton Beach	FL
FL0005	SHAMROCK_Leg_N		SHAMROCK_Leg_S	KILLARNE_Leg_W	Tallahassee	FL
FL0006	GULFDRS_Leg_N	BRIDGEST_Leg_E	GULFDRS_Leg_S		Bradenton Beach	FL
FL0007	JUANARD_Leg_N	SW18THST_Leg_E	JUANARD_Leg_S	SW18THST_Leg_W	Boca Raton	FL
FL0014	COMMERCE_Leg_N	RESERVEB_Leg_E	COMMERCE_Leg_S	RESERVEB_Leg_W	Port St. Lucie	FL
FL0015	INDIANBA_Leg_N	COMMONSD_Leg_E	INDIANBA_Leg_S	COMMONSD_Leg_W	Destin	FL
FL0016	NBLVD_Leg_N	WCOUNTRY_Leg_E	NBLVD_Leg_S	WCOUNTRY_Leg_W	Tampa	FL
FL0017	FULTONAV_Leg_N	FAIRMONT_Leg_E	FULTONAV_Leg_S	FAIRMONT_Leg_W	Clearwater	FL
FL0018	HILLAVEN_Leg_N	HOLLYWOO_Leg_E		HOLLYWOO_Leg_W	Mary Ester	FL
FL0020	JACARAND_Leg_N	EVENICEA_Leg_E	JACARAND_Leg_S	EVENICEA_Leg_W	Venice	FL
FL0023	NMARTINL_Leg_N	PALMETTO_Leg_E	NMARTINL_Leg_S	PALMETTO_Leg_W	Clearwater	FL
FL0024	FLEISCHM_Leg_N	MICCOSUK_Leg_E		MICCOSUK_Leg_W	Tallahassee	FL
FL0025	N40THST_Leg_N	EHANNAAV_Leg_E	N40THST_Leg_S	EHANNAAV_Leg_W	Tampa	FL
FL0026	N40THST_Leg_N	ERIVERHI_Leg_E	N40THST_Leg_S	ERIVERHI_Leg_W	Tampa	FL
FL0027	NMALCOLM_Leg_N	EYUKONST_Leg_E	NMALCOLM_Leg_S	EYUKONST_Leg_W	Tampa	FL
FL0028	NBEACHRD_Leg_N	BEACHRD_Leg_E	GULFBLVD_Leg_S		Englewood Beach	FL
FL0029	CINDERIA_Leg_N	NLAKEORL_Leg_E		NLAKEORL_Leg_W	Orlando	FL
FL0031	SH-0A1A_Leg_N	NECAUSEW_Leg_E	SH-0A1A_Leg_S	NECAUSEW_Leg_W	Jensen Beach	FL
FL0033	NWCALIFO_Leg_N	NWPEACOC_Leg_E	NWCALIFO_Leg_S	NWPEACOC_Leg_W	Port St. Lucie	FL
FL0036	CR-0707_Leg_N	AVEA_Leg_E	CR-0707_Leg_S	AVEA_Leg_W	Fort Pierce	FL
FL0037	SCOLORAD_Leg_N	SEOSCEOL_Leg_E	SCOLORAD_Leg_S	SWOSCEOL_Leg_W	Stuart	FL
FL0039		LAKEBREE_Leg_E	SLAKEORL_Leg_S	LAKEBREE_Leg_W	Orlando	FL
FL0040	SEPINEVA_Leg_N	SEWESTMO_Leg_E	SEPINEVA_Leg_S	SEWESTMO_Leg_W	Port St. Lucie	FL
FL0043		TOUCHTON_Leg_E	WALLACED_Leg_S	TOUCHTON_Leg_W	Jacksonville	FL
FL0044	HAGENRAN_Leg_N	PALMISLE_Leg_E	HAGENRAN_Leg_S	MAJESTIC_Leg_W	Boynton Beach	FL
FL0046	NLOISAVE_Leg_N	WTAMPABA_Leg_E	NLOISAVE_Leg_S	TAMPABAY_Leg_W	Tampa	FL
FL0047	HAGENRAN_Leg_N	TREVISOL_Leg_E	HAGENRAN_Leg_S	TREVISOL_Leg_W	Boynton Beach	FL
FL0048	SH-0A1A_Leg_N	BEACHLAG_Leg_E	SH-0A1A_Leg_S	BEACHLAG_Leg_W	Amelia Isl/	FL
FL0052	HAGENRAN_Leg_N	CHARLEST_Leg_E	HAGENRAN_Leg_S	CHARLEST_Leg_W	Boynton Beach	FL

Link ID	North Leg	East Leg	South Leg	West Leg	City	State
FL0053	HAGENRAN_Leg_N	GATEWAYB_Leg_E	HAGENRAN_Leg_S	ABERDEEN_Leg_W	Boynton Beach	FL
FL0054	HAGENRAN_Leg_N	LECHALET_Leg_E	HAGENRAN_Leg_S	LECHALET_Leg_W	Boynton Beach	FL
FL0055	VIAFLORA_Leg_N	LAKEIDAR_Leg_E	VIAFLORA_Leg_S	LAKEIDAR_Leg_W	Delray Beach	FL
FL0057		E4THAVE_Leg_E	CHANNELS_Leg_S	E3RDAVE_Leg_W	Tampa	FL
FL0058	BUENAVIS_Leg_N		BUENAVIS_Leg_S	CO-0472_Leg_W	The Villages	FL
FL0060	CASLERAV_Leg_N	PALMETTO_Leg_E	CASLERAV_Leg_S	PALMETTO_Leg_W	Clearwater	FL
FL0064	BUENAVIS_Leg_N	ELCAMINO_Leg_E	BUENAVIS_Leg_S	GLENVIEW_Leg_W	The Villages	FL
FL0067	ELCAMINO_Leg_N	MORSEBLV_Leg_E	PAIGEPL_Leg_S	MORSEBLV_Leg_W	The Villages	FL
FL0076	SE24THAV_Leg_N	BIRKDALE_Leg_E	SE24THAV_Leg_S	VISCAYAP_Leg_W	Cape Coral	FL
FL0077		INDIANCR_Leg_E	CENTRALB_Leg_S	INDIANCR_Leg_W	Jupiter	FL
FL0078	CENTRALB_Leg_N	FREDERIC_Leg_E	CENTRALB_Leg_S	FREDERIC_Leg_W	Jupiter	FL
FL0081	NEINDIAN_Leg_N	NECAUSEW_Leg_E	NEINDIAN_Leg_S	NEPINEAP_Leg_W	Jensen Beach	FL
FL0082	SEGOWIN_Leg_N	SEWESTMO_Leg_E		SEWESTMO_Leg_W	Port St. Lucie	FL
FL0083	BAKERSFI_Leg_N	SEWESTMO_Leg_E	SEBAKERS_Leg_S	SEWESTMO_Leg_W	Port St. Lucie	FL
FL0084	SEWESTMO_Leg_N	SEMORNIN_Leg_E	SEWESTMO_Leg_S	SEMORNIN_Leg_W	Port St. Lucie	FL
FL0089	NLAKEDR_Leg_N	CLEVELAN_Leg_E	SLAKEDR_Leg_S	CLEVELAN_Leg_W	Clearwater	FL
FL0090	NCORONAA_Leg_N	CLEVELAN_Leg_E	SCORONAA_Leg_S	CLEVELAN_Leg_W	Clearwater	FL
FL0091	NSATURNA_Leg_N	CLEVELAN_Leg_E	SSATURNA_Leg_S	CLEVELAN_Leg_W	Clearwater	FL
FL0098	NAURORAA_Leg_N	CLEVELAN_Leg_E	SAURORAA_Leg_S	CLEVELAN_Leg_W	Clearwater	FL
FL0102		TRAFALGA_Leg_E	SURFSIDE_Leg_S	SANDOVAL_Leg_W	Cape Coral	FL
FL0103	75THSTW_Leg_N	53RDAVEW_Leg_E	ELCONQUI_Leg_S		(unincorporated)	FL
FL0104	NE9THST_Leg_N	NE8THAVE_Leg_E	NE9THST_Leg_S	NE8THAVE_Leg_W	Gainesville	FL
FL0105	SW91STST_Leg_N	SW24THAV_Leg_E	SW91STST_Leg_S	SW24THAV_Leg_W	Gainesville	FL
FL0107	SFLETCHE_Leg_N	BEACHACC_Leg_E	SFLETCHE_Leg_S	SADLERRD_Leg_W	Amelia Isl/	FL
FL0108		19THST_Leg_E	BALBOAAV_Leg_S	19THST_Leg_W	Panama City	FL
FL0109	JIMLEERD_Leg_N	EORANGEA_Leg_E	JIMLEERD_Leg_S	EORANGEA_Leg_W	Tallahassee	FL
FL0112	BENNINGD_Leg_N	MOUNTAIN_Leg_E	BENNINGD_Leg_S	MOUNTAIN_Leg_W	Destin	FL
FL0114	PARKSIDE_Leg_N	HOLMBERG_Leg_E		HOLMBERG_Leg_W	(unincorporated)	FL
FL0115		HOLMBERG_Leg_E	RIVERSID_Leg_S	HOLMBERG_Leg_W	(unincorporated)	FL
FL0116	A1A_Leg_N	SECOVERD_Leg_E	A1A_Leg_S	SECOVERD_Leg_W	Port Salerno	FL
FL0117	A1A_Leg_N	SESTLUCI_Leg_E	SEMANATE_Leg_S	A1A_Leg_W	Port Salerno	FL
FL0118	ECOUNTRY_Leg_N	WHIDDENV_Leg_E	ECOUNTRY_Leg_S		Highl/ Beach	FL
FL0119	HOODROAD_Leg_N		GRANDECO_Leg_S	HOODROAD_Leg_W	(unincorporated)	FL
FL0121	LINDELLB_Leg_N	DOTTEREL_Leg_E	LINDELLB_Leg_S		Delray Beach	FL
FL0122	LYONSRD_Leg_N	DILLMANR_Leg_E	LYONSRD_Leg_S	DILLMANR_Leg_W	(unincorporated)	FL
FL0123	JOGROAD_Leg_N	ACCESS_Leg_E	MIRASOLD_Leg_S	JOGROAD_Leg_W	Palm Beach Gardens	FL

Link ID	North Leg	East Leg	South Leg	West Leg	City	State
FL0124	MIRASOLB_Leg_N	JOGROAD_Leg_E	JOGROAD_Leg_S		Palm Beach Gardens	FL
FL0128		PIERSONR_Leg_E	120THAVE_Leg_S	PIERSONR_Leg_W	Wellington	FL
FL0135	NWCASHME_Leg_N	NWCASHME_Leg_E	OLDINLET_Leg_S	NWPEACOC_Leg_W	(unincorporated)	FL
FL0137	SYCAMORE_Leg_N	WESTCLIF_Leg_E	SWCOMMUN_Leg_S	WESTCLIF_Leg_W	(unincorporated)	FL
FL0138	SWCOMMUN_Leg_N	TRADITIO_Leg_E	SWCOMMUN_Leg_S	TRADITIO_Leg_W	(unincorporated)	FL
FL0139	SWVILLAG_Leg_N	CLARIDGE_Leg_E	SWVILLAG_Leg_S	WESTCLIF_Leg_W	(unincorporated)	FL
FL0142	AVALONPA_Leg_N	GOLDENRA_Leg_E	AVALONPA_Leg_S	REDMANGO_Leg_W	Orlando	FL
KS003	RiceRd_Leg_N	CyprusDr_Leg_E	RiceRd_Leg_S	I70EBRamps_Leg_W	Topeka	KS
KS004	RiceRd_Leg_N	SycamoreDr_Leg_E	RiceRd_Leg_S	I70WBRamps_Leg_W	Topeka	KS
KS005	US75Ramps_Leg_N	NW46thSt_Leg_E	US75Ramps_Leg_S	NW46thSt_Leg_W	Topeka	KS
KS006	WanamakerRd_Leg_N	SW37thSt_Leg_E	WanamakerRd_Leg_S	SW37thSt_Leg_W	Topeka	KS
KS007	WanamakerRd_Leg_N		WanamakerRd_Leg_S	SW41stSt_Leg_W	Topeka	KS
KS013	4thSt_Leg_N	BluemontAve_Leg_E	4thSt_Leg_S	BluemontAve_Leg_W	Manhattan	KS
MI001	Riverview Dr_Leg_N	W Main St_Leg_E	Riverview Dr_Leg_S	W Main St_Leg_W	Berrien	MI
MI002	5th St_Leg_N	E Main St_Leg_E	5th St_Leg_S	E Main St_Leg_W	Berrien	MI
MI003	US127_Leg_N		US127_Leg_S	N Mission Rd_Leg_W	Clare	MI
MI004	N Canal Rd_Leg_N	Willow Hwy_Leg_E	N Canal Rd_Leg_S	Willow Hwy_Leg_W	Eaton	MI
MI005	Hulett Rd_Leg_N	Bennett Rd_Leg_E	Hulett Rd_Leg_S	Bennett Rd_Leg_W	Ingham	MI
MI006	Chamberlain Dr_Leg_N	Lake Lansing Rd_Leg_E		Lake Lansing Rd_Leg_W	Ingham	MI
MI007	Wood St_Leg_N	Sams Way_Leg_E	Wood St_Leg_S	Sams Way_Leg_W	Ingham	MI
MI008	N Washington Ave_Leg_N	E Michigan Ave_Leg_E	S Washington Ave_Leg_S	W Michigan Ave_Leg_W	Ingham	MI
MI009	Cedar St_Leg_N		Cedar St_Leg_S	Holbrook Dr_Leg_W	Ingham	MI
MI010	N Main St_Leg_N	Mosher St_Leg_E	N Main St_Leg_S	Mosher St_Leg_W	Isabella	MI
MI011	Rankin St_Leg_N		Knollwood Ave_Leg_S	W Michigan Ave_Leg_W	Kalamazoo	MI
MI012	Jefferson Ave SE_Leg_N	Cherry St SE_Leg_E	Jefferson Ave SE_Leg_S	Cherry St SE_Leg_W	Kent	MI
MI013	Lafayette Ave SE_Leg_N	Wealthy St SE_Leg_E	Lafayette Ave SE_Leg_S	Wealthy St SE_Leg_W	Kent	MI
MI014	Jefferson Ave SE_Leg_N	Wealthy St SE_Leg_E	Jefferson Ave SE_Leg_S	Wealthy St SE_Leg_W	Kent	MI
MI015	Brewer Ave NE_Leg_N	7 Mile Rd NE_Leg_E	Brewer Ave NE_Leg_S	7 Mile Rd NE_Leg_W	Kent	MI
MI016	Hamburg Rd_Leg_N	Winans Lake Rd_Leg_E	Hamburg Rd_Leg_S		Livingston	MI
MI017	Kensington Rd_Leg_N	Kensington Rd_Leg_E		Jacoby Rd_Leg_W	Livingston	MI
MI018	Village Place Blvd_Leg_N	Village Place Blvd_Leg_E	Village Place Blvd_Leg_S	Green Oak Ave_Leg_W	Livingston	MI
MI020	N 3rd St_Leg_N	W Main St_Leg_E	S 3rd St_Leg_S	W Main St_Leg_W	Livingston	MI
MI021	Hayes Rd_Leg_N	25 Mile Rd_Leg_E	Hayes Rd_Leg_S	25 Mile Rd_Leg_W	Macomb	MI
MI022	Romeo Plank Rd_Leg_N	Riverside Village Apt_Leg_E	Romeo Plank Rd_Leg_S	19 Mile Rd_Leg_W	Macomb	MI
MI023	Romeo Plank Rd_Leg_N	Cass Ave_Leg_E	Romeo Plank Rd_Leg_S		Macomb	MI
MI024		Utica Rd_Leg_E	Dodge Park Rd_Leg_S	Utica Rd_Leg_W	Macomb	MI

Link ID	North Leg	East Leg	South Leg	West Leg	City	State
MI025	W Vergote Dr_Leg_N	Waterside Dr_Leg_E		Waterside Dr_Leg_W	Macomb	MI
MI026	M53 SB Ramp_Leg_N	26 Mile Rd_Leg_E	M53 SB Ramp_Leg_S	26 Mile Rd_Leg_W	Macomb	MI
MI027	M53 NB Ramp_Leg_N	26 Mile Rd_Leg_E	M53 NB Ramp_Leg_S	26 Mile Rd_Leg_W	Macomb	MI
MI028	3rd St_Leg_N	W Western Ave_Leg_E	3rd St_Leg_S	W Western Ave_Leg_W	Muskegon	MI
MI029	S Walker Rd_Leg_N	Chesapeake Dr_Leg_E	S Walker Rd_Leg_S	Chesapeake Dr_Leg_W	Muskegon	MI
MI032	Commerce Crossings Rd_Leg_N	Commerce Crossings Rd_Leg_E	Loop Rd_Leg_S		Oakland	MI
MI033	Coats Rd_Leg_N	Indianwood Rd_Leg_E	S Baldwin Rd_Leg_S	N Baldwin Rd_Leg_W	Oakland	MI
MI034	Old US-27_Leg_N	Livingston Blvd_Leg_E	Old US-27_Leg_S		Otsego	MI
MI035	68th Ave N_Leg_N	W Randall St_Leg_E	68th Ave S_Leg_S	W Randall St_Leg_W	Ottawa	MI
MI036	I75 SB Ramps_Leg_N	E Washington Rd_Leg_E	I75 SB Ramps_Leg_S	E Washington Rd_Leg_W	Saginaw	MI
MI037	I75 NB Ramps_Leg_N	E Washington Rd_Leg_E	I75 NB Ramps_Leg_S	E Washington Rd_Leg_W	Saginaw	MI
MI038	Nixon Rd_Leg_N	Huron Pkwy_Leg_E	Nixon Rd_Leg_S	Huron Pkwy_Leg_W	Washtenaw	MI
MI039	Geddes Rd_Leg_N	Geddes Rd_Leg_E	Superior Rd_Leg_S		Washtenaw	MI
MI040	N Maple Rd_Leg_N	M-14 WB Ramps_Leg_E	N Maple Rd_Leg_S	M-14 WB Ramps_Leg_W	Washtenaw	MI
MI041	N Maple Rd_Leg_N	M-14 EB Ramps_Leg_E	N Maple Rd_Leg_S	M-14 EB Ramps_Leg_W	Washtenaw	MI
MI042	N Maple Rd_Leg_N	Skyline High School_Leg_E	N Maple Rd_Leg_S		Washtenaw	MI
MI043	Community Dr_Leg_N	Campus Pkwy_Leg_E	Driveway_Leg_S	Campus Pkwy_Leg_W	Washtenaw	MI
MI044	Driveway_Leg_N	Campus Pkwy_Leg_E	Suncrest Dr_Leg_S	Campus Pkwy_Leg_W	Washtenaw	MI
MI045	Sheldon Rd_Leg_N	E Tienken Rd_Leg_E	Driveway_Leg_S	E Tienken Rd_Leg_W	Oakland	MI
MI046	Washington Rd_Leg_N	Runyon Rd_Leg_E	Washington Rd_Leg_S	E Tienken Rd_Leg_W	Oakland	MI
MI047	S W Ave_Leg_N	4th St_Leg_E	4th St_Leg_S	Driveway_Leg_W	Jackson	MI
MI048	Marsh Rd_Leg_N	Hamilton Rd_Leg_E		Hamilton Rd_Leg_W	Ingham	MI
MN003	TH3_Leg_N	195thStW_Leg_E	ChippendaleAve_Leg_S	195thStW_Leg_W	Farmington	MN
MN004	RahnRd_Leg_N	DiffleyRd_Leg_E	RahnRd_Leg_S	DiffleyRd_Leg_W	Eagan	MN
MN005	PortlandAve_Leg_N	66thSt_Leg_E	PortlandAve_Leg_S	66thSt_Leg_W	Richfield	MN
MN006	BusinessParkBlvd_Leg_N	117thAve_Leg_E	BusinessParkBlvd_Leg_S	117thAve_Leg_W	Champlin	MN
MN007	BurnhavenDr_Leg_N	CrystalLakeRd_Leg_E	BurnhavenDr_Leg_S	150thSt_Leg_W	Burnsiville	MN
MN008	ElmCreekPkwy_Leg_N	109thAve_Leg_E	LancasterLn_Leg_S	109thAve_Leg_W	Maple Grove	MN
MN009	JeffersonSt_Leg_N	39thAve_Leg_E	JeffersonSt_Leg_S	39thAve_Leg_W	Columbia Heights	MN
MN010	EnglishSt_Leg_N	FrostAve_Leg_E	EnglishSt_Leg_S	FrostAve_Leg_W	Maplewood	MN
MN011	TH3_Leg_N		TH3_Leg_S	AmanaTrail_Leg_W	Inver Grove Heights	MN
MN012	LangfordAve_Leg_N	260thSt_Leg_E	LangfordAve_Leg_S	260thSt_Leg_W	Cedar Lake Township	MN
MN015	GooseLakeRd_Leg_N	109thAve_Leg_E	ZacharyLn_Leg_S		Maple Grove	MN
MN016	ClubWestParkway_Leg_N	113thAve_Leg_E	ClubWestParkway_Leg_S	113thAve_Leg_W	Blaine	MN
MN017	CSAH 10_Leg_N	TH7_Leg_E	CSAH10_Leg_S	TH7_Leg_W	Carver	MN
MN018	TH25_Leg_N	TH7_Leg_E	TH25_Leg_S	TH7_Leg_W	Carver	MN

Link ID	North Leg	East Leg	South Leg	West Leg	City	State
MN019	HarpersSt_Leg_N	LakesPkway_Leg_E	HarperCt_Leg_S	LakesPkway_Leg_W	Blaine	MN
MN021	KenrickAve_Leg_N		KenrickAve_Leg_S	175thStW_Leg_W	Lakeville	MN
MN022	TH284_Leg_N	SparrowRd_Leg_E	TH284_Leg_S	SparrowRd_Leg_W	Waconia	MN
MN023	11thAve_Leg_N	TH19_Leg_E		TH19_Leg_W	New Prague	MN
MN024	MinnehahaAve_Leg_N	GodfreyPkwy_Leg_E	MinnehahaDr_Leg_S	MinnehahaPkwy_Leg_W	Minneapolis	MN
MN025	GallivantPl_Leg_N	153rdSt_Leg_E	FoundersLn_Leg_S	153rdSt_Leg_W	Apple Valley	MN
MN027	TH15_Leg_N	HighDr_Leg_E	MainSt_Leg_S	HighDr_Leg_W	Hutchinson	MN
MN028	LynnAve_Leg_N	McCollDr_Leg_E	GlendaleRd_Leg_S	McCollDr_Leg_W	Savage	MN
MN029		CobblestoneLakePkwy_Leg_E	ElmCreekLn_Leg_S	CobblestoneLakePkwy_Leg_W	Apple Valley	MN
MN030	CSAH18_Leg_N	CSAH118_Leg_E	FenningAve_Leg_S		Monticello	MN
MN031	Road2_Leg_N	CSAH120_Leg_E		CSAH120_Leg_W	Sartell	MN
MN032	PulaskiRd_Leg_N	16thSt_Leg_E	PulaskiRd_Leg_S		Buffalo	MN
MN033	RadioDr_Leg_N	BaileyRd_Leg_E	RadioDr_Leg_S	BaileyRd_Leg_W	Woodbury	MN
MN034		CSAH1_Leg_E	CR120_Leg_S	CSAH1_Leg_W	Sartell	MN
MN035	StCroixTrail_Leg_N	4thSt_Leg_E	CSAH18_Leg_S	5thSt_Leg_W	Lakeland	MN
MN036	StCroixTrail_Leg_N	5thSt_Leg_E	CSAH18_Leg_S	5thSt_Leg_W	Lakeland	MN
MN037	StCroixTrail_Leg_N	DivisionSt_Leg_E	CSAH18_Leg_S	DivisionSt_Leg_W	Lakeland	MN
MN038	JamaicaAve_Leg_N	StillwaterBlvd_Leg_E	CSAH6_Leg_S	34thSt_Leg_W	Lake Elmo	MN
MN040	JamaicaAve_Leg_N	TH61OffRamp_Leg_E	JamaicaAve_Leg_S	TH61OnRamp_Leg_W	CottageGrove	MN
MN043	Road1_Leg_N	CSAH120_Leg_E		CSAH120_Leg_W	Sartell	MN
MN044	PanamaAve_Leg_N	CSAH3_Leg_E	IndependenceAve_Leg_S	NewPragueBlvd_Leg_W	Cedar Lake Township	MN
MN045	BusinessParkBlvd_Leg_N	EmeryPkwy_Leg_E	BusinessParkBlvd_Leg_S		Champlin	MN
NC0003	WILLIAMS_Leg_N	HWY421ONRAMP_Leg_E	CONCORD_Leg_S	WILLIAMS_Leg_W	Lewisville	NC
NC0004	EFRONTST_Leg_N		EFRONTST_Leg_S	BROADST_Leg_W	New Bern	NC
NC0008	HAYMEADO_Leg_N	YELLOWBA_Leg_E	HAYMEADO_Leg_S	YELLOWBA_Leg_W	Mulberry	NC
NC0010	ONRAMP_Leg_N	WILLIAMS_Leg_E	OFFRAMP_Leg_S	WILLIAMS_Leg_W	Lewisville	NC
NC0011	ETREMONT_Leg_N	PARKRD_Leg_E	PARKRD_Leg_S	BROOKSID_Leg_W	Charlotte	NC
NC0013	ZEMERYLN_Leg_N	ASHEVILL_Leg_E	RATCLIFF_Leg_S	ASHEVILL_Leg_W	Waynesville	NC
NC0016		POPERD_Leg_E	EPHESUSC_Leg_S	EPHESUSC_Leg_W	Chapel Hill	NC
NC0017	OLDSALEM_Leg_N	ESALEMAV_Leg_E	SMAINST_Leg_S	SALEMAVE_Leg_W	Winston-Salem	NC
NC0018	GASTONDA_Leg_N	KENDRICK_Leg_E	GASTONDA_Leg_S	KENDRICK_Leg_W	Gastonia	NC
NC0020		SKNOLLRD_Leg_E	VOITGILM_Leg_S	VOITGILM_Leg_W	Southern Pines	NC
NC0022	SGREENE_Leg_N	WMCGEEST_Leg_E	SGREENE_Leg_S	WMCGEEST_Leg_W	Greensboro	NC
NC0023	BASSCHAP_Leg_N	CHECKERB_Leg_E	NELMST_Leg_S	LAKEJEAN_Leg_W	Greensboro	NC
NC0026	COTTONVI_Leg_N	SSTANLYS_Leg_E	COTTONVI_Leg_S	SSTANLYS_Leg_W	Norwood	NC
NC0027	POTTERRD_Leg_N	WAXHAWIN_Leg_E	POTTERRD_Leg_S	WAXHAWIN_Leg_W	Waxhaw	NC

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NC0028		FRONTST_Leg_E	SMAINST_Leg_S	SOUTHAVE_Leg_W	Wake Forest	NC
NC0029	CASCADE_Leg_N	CABARRUS_Leg_E	OLDCHARL_Leg_S	CABARRUS_Leg_W	Concord	NC
NC0031	CAUSEWAY_Leg_N	WFIRSTST_Leg_E		EFIRSTST_Leg_W	Ocean Isle	NC
NC0034		WMEADOWRD_Leg_E	BOONERD_Leg_S	CHURCHST_Leg_W	Eden	NC
NC0035	JEFFERSO_Leg_N	EELIZABE_Leg_E	INDIANTO_Leg_S	EELIZABE_Leg_W	Clinton	NC
NC0036	HARBOROA_Leg_N	EMOOREST_Leg_E	FERRYRDS_Leg_S	EMOOREST_Leg_W	Southport	NC
NC0037	PERSONST_Leg_N	NCOOLSPR_Leg_E	PERSONST_Leg_S	NCOOLSPR_Leg_W	Fayetteville	NC
NC0038	MTMORIAH_Leg_N	OLDCHAPE_Leg_E		OLDCHAPE_Leg_W	Durham	NC
NC0039	BUCHANAN_Leg_N		BREWERRD_Leg_S	BREWERRD_Leg_W	Winston-Salem	NC
NC0041		EROCKYRIVER_Leg_E	DAVIDSONC_Leg_S	CONCORD_Leg_W	Davidson	NC
NC0043	SR22_Leg_N	AIRPORTR_Leg_E	SR22_Leg_S	AIRPORTR_Leg_W	Southern Pines	NC
NC0046	NLAFAYET_Leg_N	EZIONCHU_Leg_E	NLAFAYET_Leg_S	WZIONCHU_Leg_W	Shelby	NC
NC0047		WESLEYCH_Leg_E	GOLDMINE_Leg_S	WESLEYCH_Leg_W	Indian Trail	NC
NY0001	EMAINST_Leg_N	CR39_Leg_E	EMAINST_Leg_S	CR39_Leg_W	Bainbridge	NY
NY0002	HARLEMRD_Leg_N	CLEVELAN_Leg_E	HARLEMRD_Leg_S	CLEVELAN_Leg_W	Cheektowaga	NY
NY0003	BUFFALOR_Leg_N	MAINST_Leg_E		HAMBURG_Leg_W	East Aurora	NY
NY0004	BUFFALO_Leg_N	MAIN_Leg_E	BUFFALO_Leg_S	MAIN_Leg_W	Hamburg	NY
NY0005	Riversid_Leg_N	FloralDr_Leg_E	Riversid_Leg_S	FloralDr_Leg_W	Johnson City	NY
NY0006	OldNiska_Leg_N	Watervli_Leg_E	WolfRd_Leg_S	Watervli_Leg_W	Latham	NY
NY0007	NY-52_Leg_N	SH-17Ramp_Leg_E	NY-52_Leg_S	MAIN_Leg_W	Liberty	NY
NY0010	RoundLa_Leg_N	George_Leg_E		CurryRd_Leg_W	Malta	NY
NY0011	Clover_Leg_N	RushMend_Leg_E	Clover_Leg_S	RushMend_Leg_W	Mendon	NY
NY0012	Raymond_Leg_N		Raymond_Leg_S	College_Leg_W	Poughkeepsie	NY
NY0013	NY-376_Leg_N	College_Leg_E	Raymond_Leg_S	Fulton_Leg_W	Poughkeepsie	NY
NY0016	Slinger_Leg_N		Slinger_Leg_S	Blessing_Leg_W	Slingerlands	NY
NY0017	Scotland_Leg_N	Cherry_Leg_E	Scotland_Leg_S	Lagrange_Leg_W	Slingerlands	NY
NY0018	State_Leg_N	Maple_Leg_E		Maple_Leg_W	Voorheesville	NY
NY0022	NY40_Leg_N	NY29_Leg_E		NY29_Leg_W	Greenwich	NY
NY0023	SandCreekRd_Leg_N	AviationRd_Leg_E	SandCreekRd_Leg_S	ColonieCtr_Leg_W	Colonie	NY
NY0024	NY28_Leg_N	I587Ramps_Leg_E	Col Chandler Dr_Leg_S	WashingtonAve_Leg_W	Kingston	NY
NY0025	US9_Leg_N	NY2_Leg_E	US9_Leg_S	NY2_Leg_W	Latham	NY
NY0026	FerryRd_Leg_N	TyndallRd_Leg_E	ShortBeachRd_Leg_S	TyndallRd_Leg_W	North Haven	NY
ON001	Townline Rd_Leg_N		Townline Rd_Leg_S	Can-Amera Pkwy_Leg_W	Waterloo	ON
ON003	Ira Needles Blvd_Leg_N	University Ave W_Leg_E	Ira Needles Blvd_Leg_S	University Ave W_Leg_W	Waterloo	ON
ON005	Ira Needles Blvd_Leg_N	Highview Dr_Leg_E	Ira Needles Blvd_Leg_S	Trussler Rd_Leg_W	Waterloo	ON
ON007	Ira Needles Blvd_Leg_N	Victoria St S_Leg_E	Ira Needles Blvd_Leg_S		Waterloo	ON

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ON008	Ira Needles Blvd_Leg_N	Highland Rd W_Leg_E	Ira Needles Blvd_Leg_S	Highland Rd W_Leg_W	Waterloo	ON
ON009	Ira Needles Blvd_Leg_N	Erb St W_Leg_E	Ira Needles Blvd_Leg_S	Erb St W_Leg_W	Waterloo	ON
ON013		Bridge St E_Leg_E	Lancaster St W_Leg_S	Bridge St W_Leg_W	Waterloo	ON
ON015	Ira Needles Blvd_Leg_N		Ira Needles Blvd_Leg_S	The Boardwalk_Leg_W	Waterloo	ON
PA0001		DoeRun_Leg_E	Union_Leg_S	DoeRun_Leg_W	Unionville	PA
PA0002	Bethleha_Leg_N	Station_Leg_E	Bethleha_Leg_S	Station_Leg_W	Quakertown	PA
PA0003	Roths_Leg_N	Main_Leg_E	Hanover_Leg_S	Hanover_Leg_W	Spring Grove	PA
PA0004	Gravel_Leg_N	Gravel_Leg_E	CVSAccess_Leg_S	Big_Leg_W	Lower Frederick Township	PA
PA0005	Newtown_Leg_N	Davids_Leg_E	Newtown_Leg_S	Davids_Leg_W	Newtown	PA
PA0006	DELTA RD_Leg_N	BROAD ST_Leg_E	DELTA RD_Leg_S	BRIANSVI_Leg_W	Peach Bottom	PA
PA0007	Trail898_Leg_N	Monaster_Leg_E	Vincent_Leg_S	Monaster_Leg_W	Unity Twp.	PA
PA0008	Mulberry_Leg_N	ViaBella_Leg_E		ViaBella_Leg_W	Williamsport	PA
PA0009	William_Leg_N	ViaBella_Leg_E		ViaBella_Leg_W	Williamsport	PA
WA0001	539_Leg_N		539_Leg_S	River_Leg_W	Lynden	WA
WA0002	Commerc_Leg_N	20_Leg_E	Commerc_Leg_S		Anacortes	WA
WA0003	Weber_Leg_N	DeerPark_Leg_E	Weber_Leg_S	Crawford_Leg_W	Deer Park	WA
WA0004	Kellogg_Leg_N	4th_Leg_E	Kellogg_Leg_S	4th_Leg_W	Kennewick	WA
WA0005	WClearwater_Leg_N	Ridgeline_Leg_E	EBadger_Leg_S	Leslie_Leg_W	Kennewick	WA
WA0008	539_Leg_N	Pole_Leg_E	539_Leg_S	Pole_Leg_W	Lynden	WA
WA0009	539_Leg_N	TenMile_Leg_E	539_Leg_S	EmployerAccess_Leg_W	Lynden	WA
WA0010	539_Leg_N	Wiser_Leg_E	539_Leg_S	Wiser_Leg_W	Lynden	WA
WA0011	Best_Leg_N	McLean_Leg_E	Best_Leg_S	McLean_Leg_W	Mount Vernon	WA
WA0013	Freya_Leg_N	Spokane_Leg_E	Freya_Leg_S		Spokane	WA
WA0014		Mansfield_Leg_E	Montgom_Leg_S	Montgom_Leg_W	Spokane Valley	WA
WA0015	148th_Leg_N	146th_Leg_E	Redmond_Leg_S	145th_Leg_W	Woodinville	WA
WA0018	124th_Leg_N	304th_Leg_E	124th_Leg_S	304th_Leg_W	Auburn	WA
WA0019	168TH_Leg_N	276TH_Leg_E	168TH_Leg_S	COMMERCIAL ACCESS_Leg_W	Covington	WA
WA0020	168th_Leg_N	Parking_Leg_E	168th_Leg_S	Parking_Leg_W	Covington	WA
WA0023	Gora_Leg_N		Selah_Leg_S	Selah_Leg_W	Selah	WA
WA0024	Casino_Leg_N	North_Leg_E		North_Leg_W	Snoqualmie	WA
WA0025	Myra_Leg_N	Heritage_Leg_E	Myra_Leg_S	Heritage_Leg_W	Walla Walla	WA
WA0026	NHwy_Leg_N	5thAve_Leg_E	Railroad_Leg_S		Colville	WA
WA0027	164th_Leg_N	256th_Leg_E	164th_Leg_S	256th_Leg_W	Covington	WA
WA0028	FallCity_Leg_N	FallCity_Leg_E	River_Leg_S		Fall City	WA
WA0029	Vista_Leg_N	Malloy_Leg_E	Vista_Leg_S	Ferndale_Leg_W	Ferndale	WA
WA0030		Britton_Leg_E	Gateway_Leg_S	Britton_Leg_W	Lacey	WA

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WA0031	SR9_Leg_N	SR9_Leg_E		SR538_Leg_W	Mount Vernon	WA
WA0033	N45thAve_Leg_N	SR501_Leg_E	S45thAve_Leg_S	SR501_Leg_W	Ridgefield	WA
WA0034	28thAveS_Leg_N	S317thSt_Leg_E		S317thSt_Leg_W	Federal Way	WA
WA0035	Driveway_Leg_N	Driveway_Leg_E	S332St_Leg_S	S333St_Leg_W	Federal Way	WA
WA0037	Ncenter_Leg_N	Grand_Leg_E	Ncenter_Leg_S	Grand_Leg_W	Kennewick	WA
WA0038	ShoultesRd_Leg_N	108thStNE_Leg_E	51stAveNE_Leg_S	ShoultesRd_Leg_W	Marysville	WA
WA0039	SR903_Leg_N	SR903_Leg_E	BullfrogRd_Leg_S		Cle Elum	WA
WA0041	Rainier_Leg_N	Balustra_Leg_E	Rainier_Leg_S	67thAveSE_Leg_W	Lacey	WA
WA0042	14thAveSE_Leg_N	Henderson_Leg_E	HenParkDrSE_Leg_S	Henderson_Leg_W	Olympia	WA
WA0043	BruceRd_Leg_N	MtSpokan_Leg_E	BruceRd_Leg_S	SR206_Leg_W	Spokane	WA
WA0045	Duvall_Leg_N	124th_Leg_E	Duvall_Leg_S	124th_Leg_W	Duvall	WA
WA0046	Weyer_Leg_N		Weyer_Leg_S	336thSt_Leg_W	Federal Way	WA
WA0047	Vancouve_Leg_N	27thAve_Leg_E	Vancouve_Leg_S	27thAve_Leg_W	Kennewick	WA
WA0048	Union_Leg_N	19thAve_Leg_E	Union_Leg_S	19thAve_Leg_W	Kennewick	WA
WA0049	Union_Leg_N	4thAve_Leg_E	Union_Leg_S	4thAve_Leg_W	Kennewick	WA
WA0051	Union_Leg_N	Kennewic_Leg_E	Union_Leg_S	Kennewic_Leg_W	Kennewick	WA
WA0052	Quinault_Leg_N	Okanogan_Leg_E	Young_Leg_S		Kennewick	WA
WA0053	Laconner_Leg_N	Chilberg_Leg_E		Chilberg_Leg_W	La Conner	WA
WA0054	Marvin_Leg_N	Pacific_Leg_E	Marvin_Leg_S	Pacific_Leg_W	Lacey	WA
WA0055	Frontier_Leg_N		Anderson_Leg_S	Anderson_Leg_W	Silverdale	WA
WA0056	Broadway_Leg_N	Division_Leg_E		Division_Leg_W	Arlington	WA
WA0057	WestAve_Leg_N	Division_Leg_E	West_Leg_S	Division_Leg_W	Arlington	WA
WA0058	College_Leg_N	45thAve_Leg_E	College_Leg_S	45thAve_Leg_W	Lacey	WA
WA0059	Marvin_Leg_N	Hawks_Leg_E	Marvin_Leg_S	Hawks_Leg_W	Lacey	WA
WA0060	Orion_Leg_N	Meridian_Leg_E	Meridian_Leg_S		Lacey	WA
WA0061	59thAve_Leg_N	MainSt_Leg_E		MainSt_Leg_W	Lakewood	WA
WA0064	Division_Leg_N	Yonezawa_Leg_E	Division_Leg_S	Belmont_Leg_W	Moses Lake	WA
WA0065	Madison_Leg_N	HighSchool_Leg_E	Madison_Leg_S	HighSchool_Leg_W	Bainbridge Island	WA
WA0067	33RDPL_Leg_N		WEYERHAU_Leg_S	WEYERHAU_Leg_W	Federal Way	WA
WA0068	Union_Leg_N	CreekstoneDr_Leg_E	Southridge_Leg_S	CreekstoneDr_Leg_W	Kennewick	WA
WA0069	Freedom_Leg_N	Quinault_Leg_E	Galaxy_Leg_S	I-5Ramp_Leg_W	Lacey	WA
WA0070	Marvin_Leg_N	Willamette_Leg_E	Marvin_Leg_S	Britton_Leg_W	Lacey	WA
WA0072	Bethel_Leg_N	MileHill_Leg_E		Bethel_Leg_W	Port Orchard	WA
WA0073	Parking Lot_Leg_N	Market_Leg_E	Capitol_Leg_S	Corky_Leg_W	Olympia	WA
WI0001	Velp_Leg_N	Woodale_Leg_E	Velp_Leg_S	Woodale_Leg_W	Howard	WI
WI0002	EWiscon_Leg_N	Ewiscon_Leg_E		Wiscon_Leg_W	Oconomowoc	WI

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WI0003	116th_Leg_N	Grange_Leg_E	116th_Leg_S	Grange_Leg_W	Hales Corners	WI
WI0004	9th_Leg_N	Grant_Leg_E	9th_Leg_S	Grant_Leg_W	De Pere	WI
WI0005	River_Leg_N	Paradise_Leg_E	River_Leg_S	Paradise_Leg_W	West Bend	WI
WI0006	Cardinal_Leg_N	Woodale_Leg_E	Cardinal_Leg_S	Woodale_Leg_W	Howard	WI
WI0007	Thompson_Leg_N	Main_Leg_E	Thompson_Leg_S	Main_Leg_W	Sun Prairie	WI
WI0008	LIBAL ST_Leg_N	ALLOUEZ_Leg_E	LIBAL ST_Leg_S	ALLOUEZ_Leg_W	Allouez	WI
WI0009	40th_Leg_N	Superior_Leg_E	Wilgus_Leg_S	Superior_Leg_W	Sheboygan	WI
WI0010	Greenbay_Leg_N	Cecil_Leg_E	Jewelers_Leg_S	Cecil_Leg_W	Neenah	WI
WI0012	Hillcres_Leg_N	Glendale_Leg_E	Hillcres_Leg_S	Glendale_Leg_W	Howard	WI
WI0013	LakePark_Leg_N	Midway_Leg_E	LakePark_Leg_S	Midway_Leg_W	Appleton	WI
WI0014	Hart_Leg_N	CoRd_Leg_E		CoRd_Leg_W	Beloit	WI
WI0015	Hart_Leg_N	EBOn_Leg_E	Hart_Leg_S	EBOff_Leg_W	Beloit	WI
WI0016	Hart_Leg_N	WBOff_Leg_E	Hart_Leg_S	WBOn_Leg_W	Beloit	WI
WI0017	Bridge_Leg_N	Rushman_Leg_E	Bridge_Leg_S	River_Leg_W	Chippewa Falls	WI
WI0018	Seymour_Leg_N	County_Leg_E	Seymour_Leg_S	County_Leg_W	Chippewa Falls	WI
WI0019		County_Leg_E	CountyM_Leg_S	County_Leg_W	Milton	WI
WI0020	Black_Leg_N	Summit_Leg_E	Black_Leg_S	Summit_Leg_W	Wales	WI
WI021		US22_Leg_E	US22_Leg_S	RoyaltonSt_Leg_W	Waupaca	WI
WI022	17thAveS_Leg_N	2ndAveS_Leg_E	2ndAveS_Leg_S	GaynorAve_Leg_W	Wisconsin Rapids	WI
WI023	LawrenceDr_Leg_N	GrantSt_Leg_E	LawrenceDr_Leg_S	GrantSt_Leg_W	De Pere	WI
WI024	CountyRdN_Leg_N	US10_Leg_E	CountyRdN_Leg_S	US10_Leg_W	Calumet	WI
WI025	CountyRdA_Leg_N	CountyRdJJ_Leg_E	NLynndaleDr_Leg_S		Outagamie	WI
WI026	CountyRdGV_Leg_N	DickinsonRd_Leg_E	MonroeRd_Leg_S	CountyRdG_Leg_W	Brown	WI
WI027	WI55_Leg_N	US10_Leg_E	WI55_Leg_S	US10_Leg_W	Calumet	WI
WI028	CountyRdB_Leg_N	WI310_Leg_E	CountyRdB_Leg_S	WI310_Leg_W	(unincorporated)	WI
WI029	CountyRdQ_Leg_N	WI310_Leg_E	CountyRdQ_Leg_S	WI310_Leg_W	(unincorporated)	WI
WI030	WI67_Leg_N	RockvilleRd_Leg_E	WI67_Leg_S	CountyRdAA_Leg_W	Kiel	WI
WI031	WI67_Leg_N	WI57_Leg_E	WI67_Leg_S	WI57_Leg_W	Kiel	WI
WI032	CountyRdR_Leg_N	WI310_Leg_E	NRapidsRd_Leg_S	WI310_Leg_W	(unincorporated)	WI
WI033	SWalterAve_Leg_N	CountyRdCE_Leg_E	EJohnSt_Leg_S	ECollegeAve_Leg_W	Appleton	WI
WI034	WI42_Leg_N	CountyRdJJ_Leg_E	WI42_Leg_S	CountyRdJJ_Leg_W	(unincorporated)	WI
WI035	WI42_Leg_N	CountyRdY_Leg_E	WI42_Leg_S	CountyRdY_Leg_W	(unincorporated)	WI
WI036	S9thSt_Leg_N	ScheuringRd_Leg_E	MatthewDr_Leg_S	CountyRdF_Leg_W	De Pere	WI
WI037	SuburbanDr_Leg_N	ScheuringRd_Leg_E	SuburbanDr_Leg_S	CountyRdF_Leg_W	De Pere	WI
WI038	WI32_Leg_N	WI96_Leg_E	GreenleafRd_Leg_S	DaySt_Leg_W	(unincorporated)	WI
WI039	Main St_Leg_N	AllouezAve_Leg_E	US141_Leg_S	AllouezAve_Leg_W	Bellevue	WI

Link ID	North Leg	East Leg	South Leg	West Leg	City	State
WI040	WI57_Leg_N	SWisonsinSt_Leg_E	Sbroadway_Leg_S	WI32_Leg_W	De Pere	WI
WI041	WI55_Leg_N	CalumetSt_Leg_E	FriendshipDr_Leg_S	CountyRdKK_Leg_W	(unincorporated)	WI
WI042	LakeParkRd_Leg_N	PlankRd_Leg_E	LakeParkRd_Leg_S	PlankRd_Leg_W	(unincorporated)	WI
WI043	CountyRdN_Leg_N	BuchananRd_Leg_E	CountyRdN_Leg_S	EmonsRd_Leg_W	(unincorporated)	WI
WI044	GiddingsAve_Leg_N	WI28_Leg_E	WI32_Leg_S	WI28_Leg_W	(unincorporated)	WI
WI047	VanguardDr_Leg_N	WI42_Leg_E	VanguardDr_Leg_S	WI42_Leg_W	(unincorporated)	WI
WI048	TullarRd_Leg_N	BreezewoodLn_Leg_E	TullarRd_Leg_S	BreezewoodLn_Leg_W	Neenah	WI
WI050	WI53_Leg_N	ScullyDr_Leg_E	WI53_Leg_S	OldTownHallRd_Leg_W	Eau Claire	WI
WI052	US53SBofframp_Leg_N	CountyRdO_Leg_E	US53SBonramp_Leg_S	CountyRdO_Leg_W	Rice Lake	WI
WI053	US53SBonramp_Leg_N	CountyRdO_Leg_E	US53SBofframp_Leg_S	CountyRdO_Leg_W	Rice Lake	WI
WI054	WI124_Leg_N	CountyRdS_Leg_E	WI124_Leg_S	CountyRdS_Leg_W	(unincorporated)	WI
WI058	SMoorlandRd_Leg_N	RockridgeRd_Leg_E	CountyRdO_Leg_S	I43NBofframp_Leg_W	New Berlin	WI
WI059	NWalnutSt_Leg_N	WI16_Leg_E	NWalnutSt_Leg_S	EWisconsinAve_Leg_W	Oconomowoc	WI
WI061	CountyRdG_Leg_N	PrivateAccess_Leg_E	WoodRd_Leg_S	ParksideBlvd_Leg_W	(unincorporated)	WI
WI062	CountyRdP_Leg_N	CountyRdPV_Leg_E	CountyRdP_Leg_S	PleasantvalleyRd_Leg_W	(unincorporated)	WI
WI063	US41SBofframp_Leg_N	WI145_Leg_E	US41SBonramp_Leg_S	WI145_Leg_W	(unincorporated)	WI
WI064	US45NBonramp_Leg_N	WI145_Leg_E	US45NBofframp_Leg_S	WI145_Leg_W	(unincorporated)	WI
WI065	VeteransAve_Leg_N	WaterSt_Leg_E		IslandAve_Leg_W	West Bend	WI
WI066	CountyRdY_Leg_N	KelseyDr_Leg_E	RacineAve_Leg_S	KelseyDr_Leg_W	Muskego	WI
WI067	RacineAve_Leg_N	I43NBofframp_Leg_E	RacineAve_Leg_S	I43NBonramp_Leg_W	New Berlin	WI
WI069	WI164_Leg_N	CountyRdQ_Leg_E	WI164_Leg_S	CountyRdQ_Leg_W	(unincorporated)	WI
WI070	N25thSt_Leg_N	CanalSt_Leg_E		CanalSt_Leg_W	Milwaukee	WI
WI071	WI38_Leg_N	CountyRdK_Leg_E		WI38_Leg_W	Caledonia	WI
WI072	WI59_Leg_N	BluffRd_Leg_E	ElkhornRd_Leg_S	ClaySt_Leg_W	Whitewater	WI
WI073	S6thSt_Leg_N	Florida_Leg_E	SAlexanderSt_Leg_S	S6thSt_Leg_W	Milwaukee	WI
WI074	PortageRd_Leg_N	HansonRd_Leg_E		HansonRd_Leg_W	Madison	WI
WI075	BennettRd_Leg_N	US18_Leg_E	BennettRd_Leg_S	US18_Leg_W	Dodgeville	WI
WI076	N8thSt_Leg_N	SpringdaleSt_Leg_E	S8thSt_Leg_S	WI78_Leg_W	Mount Horeb	WI
WI077	ThompsonDr_Leg_N	CommercialAve_Leg_E	ThompsonDr_Leg_S	CommercialAve_Leg_W	Madison	WI
WI078	ThompsonDr_Leg_N		ThompsonDr_Leg_S	WI30EBofframp_Leg_W	Madison	WI
WI079	ParmenterSt_Leg_N	ParmenterSt_Leg_E	US14NBofframp_Leg_S	DiscoveryDr_Leg_W	Middleton	WI

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Introduction

The following content is proposed text for the *Highway Safety Manual* (HSM) to integrate the findings from this project into the appropriate locations in the HSM. This text was provided to the contractor for NCHRP Project 17-71 *Proposed AASHTO Highway Safety Manual, Second Edition* for their consideration and coordination with the NCHRP Project 17-71 Panel. The following presents the proposed text. The content is added to HSM, First Edition chapters. The content is organized as changes to:

- **Chapter 4 Network Screening** – To integrate the findings from the planning-level crash prediction models. This was done primarily by providing a paragraph noting the roundabout crash prediction models are available for use in a number of the different network screening measures and referring to the new appendix 4B. Therefore, in this appendix, we are providing the page of Chapter 4 where that new proposed text from Project 17-70 is located rather than providing all of the HSM, Chapter 4. The remainder of the body of Chapter 4 would be the same.
- **Chapter 4 Appendix 4B** – Providing a new appendix with the details regarding the planning-level roundabout crash prediction models.
- **Chapter 12 Predictive Method for Urban and Suburban Arterials** – To integrate the leg-level and intersection-level crash prediction models for roundabouts.

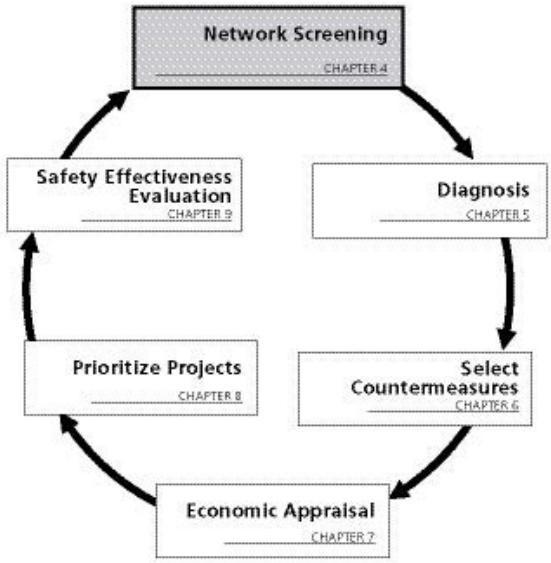
The entirety of the leg-level and intersection-level crash prediction models are presented in Chapter 12. References from the rural-based changes would be provided in Chapter 10 and Chapter 11.

Chapter 4 Network Screening

4.1 INTRODUCTION

Network screening is a process for reviewing a transportation network to identify and rank sites from most likely to least likely to realize a reduction in crash frequency with implementation of a countermeasure. Those sites identified as most likely to realize a reduction in crash frequency are studied in more detail to identify crash patterns, contributing factors, and appropriate countermeasures. Network screening can also be used to formulate and implement a policy, such as prioritizing the replacement of non-standard guardrail statewide at sites with a high number of run-off-the-road crashes.

As shown in Figure 4-1, network screening is the first activity undertaken in a cyclical Roadway Safety Management Process outlined in Part B. Any one of the steps in the Roadway Safety Management Process can be conducted in isolation; however, the overall process is shown here for context. This chapter explains the steps of the network screening process, the performance measures of network screening, and the methods for conducting the screening.



- Figure 4-1. Roadway Safety Management Process

NETWORK SCREENING PROCESS

There are five major steps in network screening as shown in Figure 4-2:

Establish Focus—Identify the purpose or intended outcome of the network screening analysis. This decision will influence data needs, the selection of performance measures and the screening methods that can be applied.

Identify Network and Establish Reference Populations—Specify the type of sites or facilities being screened (i.e., segments, intersections, at-grade rail crossings) and identify groupings of similar sites or facilities.

Select Performance Measures—There are a variety of performance measures available to evaluate the potential to reduce crash frequency at a site. In this step, the performance measure is selected as a function of the screening focus and the data and analytical tools available.

Select Screening Method—There are three principle screening methods described in this chapter (i.e., ranking, sliding window, and peak searching). The advantages and disadvantages of each are described in order to help identify the most appropriate method for a given situation.

Screen and Evaluate Results—The final step in the process is to conduct the screening analysis and evaluate results.

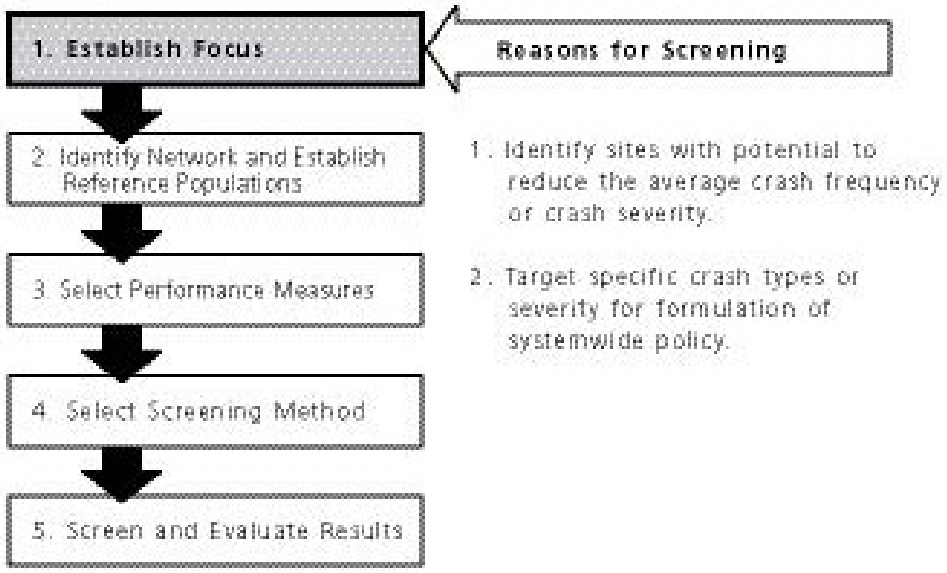
The following sections explain each of the five major steps in more detail.

STEP 1—Establish the Focus of Network Screening

The first step in network screening is to establish the focus of the analysis (Figure 4-2). Network screening can be conducted and focused on one or both of the following:

1. Identify and rank sites where improvements have potential to reduce the number of crashes.

Evaluate a network to identify sites with a particular crash type or severity in order to formulate and implement a policy (e.g., identify sites with a high number of run-off-the-road crashes to prioritize the replacement of non-standard guardrail statewide).



If network screening is being applied to identify sites where modifications could reduce the number of crashes, the performance measures are applied to all sites. Based on the results of the analysis, those sites that show potential for improvement are identified for additional analysis. This analysis is similar to a typical "black spot" analysis conducted by a jurisdiction to identify the "high crash locations."

A transportation network can also be evaluated to identify sites that have potential to benefit from a specific program (e.g., increased enforcement) or countermeasure (e.g., a guardrail implementation program). An analysis such as this might identify locations with a high proportion or average frequency of a specific crash type or severity. In this case, a subset of the sites is studied.

Determining the Network Screening Focus

Question
A State DOT has received a grant of funds for installing rumble strips on rural two-lane highways. How could State DOT staff screen their network to identify the best sites for installing the rumble strips?

Answer
State DOT staff would want to identify those sites that can possibly be improved by installing rumble strips. Therefore, assuming run-off-the-road crashes respond to rumble strips, staff would select a method that provides a ranking of sites with more run-off-the-road crashes than expected for sites with similar characteristics. The State DOT analysis would focus on only a subset of the total crash database—run-off-the-road crashes.

If, on the other hand, the State DOT had applied a screening process and ranked all of their two-lane rural highways, this would not reveal which of the sites would specifically benefit from installing rumble strips.

There are many specific activities that could define the focus of a network screening process. The following are hypothetical examples of what could be the focus of network screening:

An agency desires to identify projects for a Capital Improvement Program (CIP) or other established funding sources. In this case, all sites would be screened.

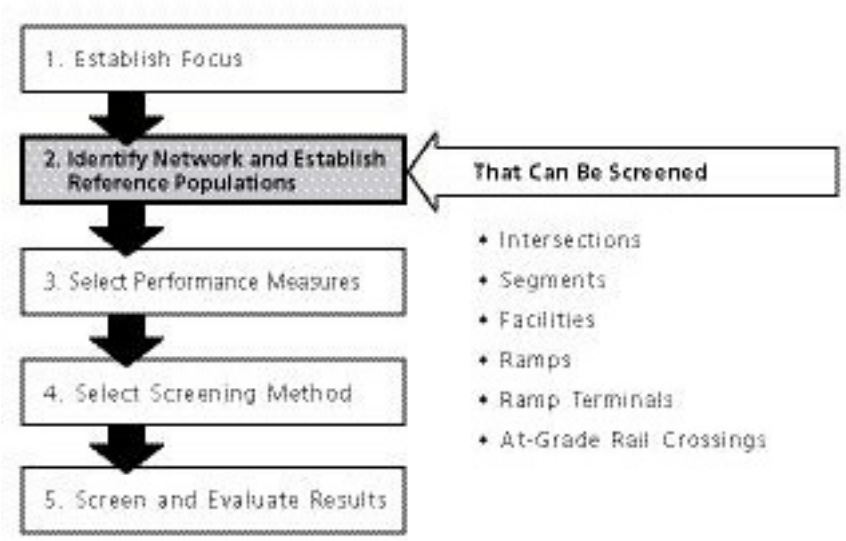
An agency has identified a specific crash type of concern and desires to implement a systemwide program to reduce that type of crash. In this case all sites would be screened to identify those with more of the specific crashes than expected.

An agency has identified sites within a sub-area or along a corridor that are candidates for further safety analysis. Only the sites on the corridor would be screened.

An agency has received funding to apply a program or countermeasure(s) systemwide to improve safety (e.g., automated enforcement). Network screening would be conducted at all signalized intersections, a subset of the whole transportation system.

STEP 2—Identify the Network and Establish Reference Populations

The focus of the network screening process established in Step 1 forms the basis for the second step in the network screening process, which includes identifying the network elements to be screened and organizing these elements into reference populations (Figure 4-3). Examples of roadway network elements that can be screened include intersections, roadway segments, facilities, ramps, ramp terminal intersections, and at-grade rail crossings.



▪ Figure 4-3. The Network Screening Process—Step 2, Identify Network and Establish Reference Populations

A reference population is a grouping of sites with similar characteristics (e.g., four-legged signalized intersections, two-lane rural highways). Ultimately, prioritization of individual sites is made within a reference population. In some cases, the performance measures allow comparisons across reference populations. The characteristics used to establish reference populations for intersections and roadway segments are identified in the following sections.

Intersection Reference Populations

Potential characteristics that can be used to establish reference populations for intersections include:

Traffic control (e.g., signalized, two-way or four-way stop control, yield control, roundabout);

Number of approaches (e.g., three-leg or four-leg intersections);

Cross-section (e.g., number of through lanes and turning lanes);

Functional classification (e.g., arterial, collector, local);

Area type (e.g., urban, suburban, rural);

Traffic volume ranges (e.g., total entering volume (TEV), peak hour volumes, average annual daily traffic (AADT)); or

Terrain (e.g., flat, rolling, mountainous).

The characteristics that define a reference population may vary depending on the amount of detail known about each intersection, the purpose of the network screening, the size of the network being screened, and the performance measure selected. Similar groupings are also applied if ramp terminal intersections or at-grade rail crossings, or both, are being screened.

Establishing Reference Populations for Intersection Screening

The following table provides an example of data for several intersections within a network that have been sorted by functional classification and traffic control. These reference populations may be appropriate for an agency that has received funding to apply red-light-running cameras or other countermeasure(s) systemwide to improve safety at signalized intersections. As such, the last grouping of sites would not be studied since they are not signalized.

Example Intersection Reference Populations Defined by Functional Classification and Traffic Control

Chapter 5 Reference Population	Chapter 6 Segment ID	Chapter 7 Street Type 1	Chapter 8 Street Type 2	Chapter 9 Traffic Control	Chapter 10 Exposure Range (TEV/Average Annual Day)
Arterial-Arterial Signalized Intersections	3	Arterial	Arterial	Signal	55,000 to 70,000
	4	Arterial	Arterial	Signal	55,000 to 70,000
	10	Arterial	Arterial	Signal	55,000 to 70,000
Arterial-Collector Signalized Intersections	33	Arterial	Collector	Signal	30,000 to 55,000
	12	Arterial	Collector	Signal	30,000 to 55,000
	23	Arterial	Collector	Signal	30,000 to 55,000
Collector-Local All-Way Stop Intersections	22	Collector	Local	All-way Stop	10,000 to 15,000
	26	Collector	Local	All-way Stop	10,000 to 15,000

Segment Reference Populations

A roadway segment is a portion of a facility that has a consistent roadway cross-section and is defined by two endpoints. These endpoints can be two intersections, on- or off-ramps, a change in roadway cross-section, mile markers or mile posts, or a change in any of the roadway characteristics listed below.

Potential characteristics that can be used to define reference populations for roadway segments include:

Number of lanes per direction;

Access density (e.g., driveway and intersection spacing);

Traffic volumes ranges (e.g., TEV, peak hour volumes, AADT);

Median type or width, or both;

Operating speed or posted speed;

Adjacent land use (e.g., urban, suburban, rural);

Terrain (e.g., flat, rolling, mountainous); and

Functional classification (e.g., arterial, collector, local).

Other more detailed example roadway segment reference populations are: four-lane cross-section with raised concrete median; five-lane cross-section with a two-way, left-turn lane; or rural two-lane highway in mountainous terrain. If ramps are being screened, groupings similar to these are also applied.

Establishing Reference Populations for Segment Screening

Example:
The following table provides data for several roadway segments within a network. The segments have been sorted by median type and cross-section. These reference populations may be appropriate for an agency that desires to implement a systemwide program to employ access management techniques in order to potentially reduce the number of left-turn crashes along roadway segments.

Chapter 11 Reference Population	Chapter 12 Segment ID	Chapter 13 Cross-Section (lanes per direction)	Chapter 14 Median Type	Chapter 15 Segment Length (miles)
4-Lane Divided Roadways	A	2	Divided	0.60
	B	2	Divided	0.40
	C	2	Divided	0.90
5-Lane Roadway with Two-Way Left-Turn Lane	D	2	TWLTL	0.35
	E	2	TWLTL	0.55
	F	2	TWLTL	0.80

STEP 3—Select Network Screening Performance Measures

The third step in the network screening process is to select one or several performance measures to be used in evaluating the potential to reduce the number of crashes or crash severity at a site (Figure 4-4). Just as intersection traffic operations analysis can be measured as a function of vehicle delay, queue length, or a volume-to-capacity ratio, intersection safety can be quantitatively measured in terms of average crash frequency, expected average crash frequency, a critical crash rate, or several other performance measures. In network screening, using multiple performance measures to evaluate each site may improve the level of confidence in the results.

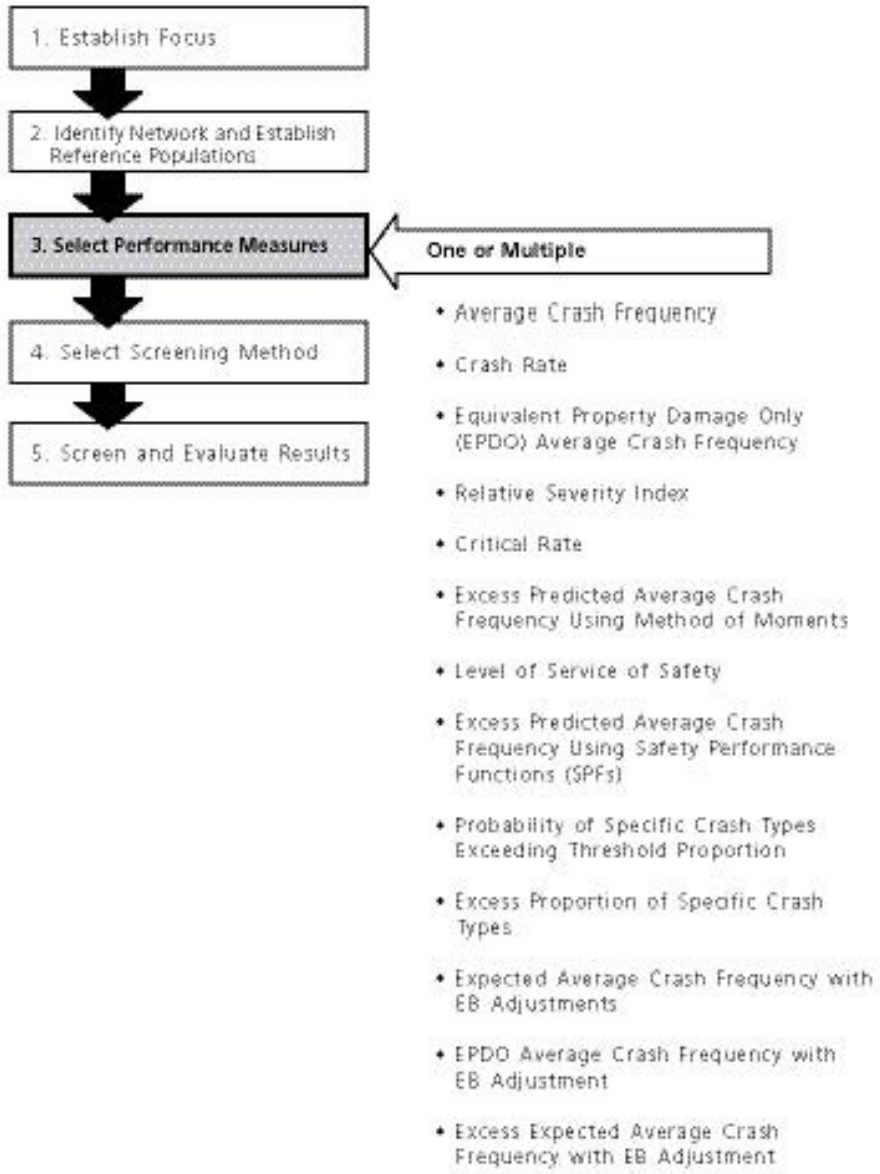


Figure 4-4. The Network Screening Process—Step 3, Select Performance Measures

Key Criteria for Selecting Performance Measures

The key considerations in selecting performance measures are: data availability, regression-to-the-mean bias, and how the performance threshold is established. This section describes each of these concepts. A more detailed description of the performance measures with supporting equations and example calculations is provided in Section 4.4.

Data and Input Availability

Typical data required for the screening analysis includes the facility information for establishing reference populations, crash data, traffic volume data, and, in some cases, safety performance functions. The amount of data and inputs that are available limits the number of performance measures that can be used. If traffic volume data is not available or cost prohibitive to collect, fewer performance measures are available for ranking sites. If traffic

volumes are collected or made available, but calibrated safety performance functions and overdispersion parameters are not, the network could be prioritized using a different set of performance measures. Table 4-1 summarizes the data and inputs needed for each performance measure.

Table 4-1. Summary of Data Needs for Performance Measures

Chapter 16	Chapter 17 Data and Inputs				
Chapter 18 Performance Measure	Chapter 19 Crash Data	Chapter 20 Roadway Information for Categorization	Chapter 21 Traffic Volume[a]	Chapter 22 Calibrated Safety Performance Function and Overdispersion Parameter	Chapter 23 Other
Average Crash Frequency	X	X			
Crash Rate	X	X	X		
Equivalent Property Damage Only (EPDO) Average Crash Frequency	X	X			EPDO Weighting Factors
Relative Severity Index	X	X			Relative Severity Indices
Critical Rate	X	X	X		
Excess Predicted Average Crash Frequency Using Method of Moments[b]	X	X	X		
Level of Service of Safety	X	X	X	X	
Excess Predicted Average Crash Frequency Using Safety Performance Functions (SPFs)	X	X	X	X	
Probability of Specific Crash Types Exceeding Threshold Proportion	X	X			
Excess Proportion of Specific Crash Types	X	X			
Expected Average Crash Frequency with EB Adjustment	X	X	X	X	
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment	X	X	X	X	EPDO Weighting Factors
Excess Expected Average Crash Frequency with EB Adjustment	X	X	X	X	

[a]Traffic volume could be AADT, ADT, or peak hour volumes.

[b]The Method of Moments consists of adjusting a site's observed crash frequency based on the variance in the crash data and average crash counts for the site's reference population. Traffic volume is needed to apply Method of Moments to establish the reference populations based on ranges of traffic volumes as well as site geometric characteristics.

Regression-to-the-Mean Bias

Crash frequencies naturally fluctuate up and down over time at any given site. As a result, a short-term average crash frequency may vary significantly from the long-term average crash frequency. The randomness of crash occurrence indicates that short-term crash frequencies alone are not a reliable estimator of long-term crash frequency. If a three-year period of crashes were to be used as the sample to estimate crash frequency, it would be

difficult to know if this three-year period represents a high, average, or low crash frequency at the site compared to previous years.

When a period with a comparatively high crash frequency is observed, it is statistically probable that a lower crash frequency will be observed in the following period (7). This tendency is known as regression-to-the-mean (RTM), and also applies to the statistical probability that a comparatively low crash frequency period will be followed by a higher crash frequency period.

Failure to account for the effects of RTM introduces the potential for "RTM bias", also known as "selection bias". RTM bias occurs when sites are selected for treatment based on short-term trends in observed crash frequency. For example, a site is selected for treatment based on a high observed crash frequency during a very short period of time (e.g., two years). However, the site's long-term crash frequency may actually be substantially lower and therefore the treatment may have been more cost-effective at an alternate site.

Performance Threshold

A performance threshold value provides a reference point for comparison of performance measure scores within a reference population. Sites can be grouped based on whether the estimated performance measure score for each site is greater than or less than the threshold value. Those sites with a performance measure score less than the threshold value can be studied in further detail to determine if reduction in crash frequency or severity is possible.

The method for determining a threshold performance value is dependent on the performance measure selected. The threshold performance value can be a subjectively assumed value, or calculated as part of the performance measure methodology. For example, threshold values are estimated based on: the average of the observed crash frequency for the reference population, an appropriate safety performance function, or Empirical Bayes methods. Table 4-2 summarizes whether or not each of the performance measures accounts for regression-to-the-mean bias or estimates a performance threshold, or both. The performance measures are presented in relative order of complexity, from least to most complex. Typically, the methods that require more data and address RTM bias produce more reliable performance threshold values.

- Table 4-2. Stability of Performance Measures

Chapter 24 Performance Measure	Chapter 25 Accounts for RTM Bias	Chapter 26 Method Estimates a Performance Threshold
Average Crash Frequency	No	No
Crash Rate	No	No
Equivalent Property Damage Only (EPDO) Average Crash Frequency	No	No
Relative Security Index	No	Yes
Critical Rate	Considers data variance but does not account for RTM bias	Yes
Excess Predicted Average Crash Frequency Using Method of Moments	Considers data variance but does not account for RTM bias	Yes
Level of Service of Safety	Considers data variance but does not account for RTM bias	Expected average crash frequency plus/minus 1.5 standard deviations
Excess Expected Average Crash Frequency Using SPFs	No	Predicted average crash frequency at the site
Probability of Specific Crash Types Exceeding Threshold Proportion	Considers data variance; not effected by RTM bias	Yes
Excess Proportions of Specific Crash Types	Considers data variance; not effected by RTM bias	Yes
Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency per year at the site

Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment	Yes	Expected average crash frequency per year at the site
Excess Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency per year at the site

Definition of Performance Measures

This section defines the performance measures in the HSM and the strengths and limitations of each measure. The following definitions, in combination with Tables 4-1 and 4-2, provide guidance on selecting performance measures. The procedures to apply each performance measures are presented in detail in Section 4.4.

Average Crash Frequency

The site with the greatest number of total crashes or the greatest number of crashes of a particular crash severity or type, in a given time period, is given the highest rank. The site with the second highest number of crashes in total or of a particular crash severity or type, in the same time period, is ranked second, and so on. The strengths and limitations of the Average Crash Frequency performance measure include the following:

Chapter 27 Strengths	Chapter 28 Limitations
Simple	Does not account for RTM bias
	Does not estimate a threshold to indicate sites experiencing more crashes than predicted for sites with similar characteristics
	Does not account for traffic volume
	Will not identify low-volume collision sites where simple cost-effective mitigating countermeasures could be easily applied

Crash Rate

The crash rate performance measure normalizes the frequency of crashes with the exposure, measured by traffic volume. When calculating a crash rate, traffic volumes are reported as million entering vehicles (MEV) per intersection for the study period. Roadway segment traffic volumes are measured as vehicle-miles traveled (VMT) for the study period. The exposure on roadway segments is often measured per million VMT.

The strengths and limitations of the Crash Rate performance measure include the following:

Chapter 29 Strengths	Chapter 30 Limitations
Simple	Does not account for RTM bias
Could be modified to account for severity if an EPDO or RSI-based crash count is used	Does not identify a threshold to indicate sites experiencing more crashes than predicted for sites with similar characteristics
	Comparisons cannot be made across sites with significantly different traffic volumes
	Will mistakenly prioritize low volume, low collision sites

Equivalent Property Damage Only (EPDO) Average Crash Frequency

The Equivalent Property Damage Only (EPDO) Average Crash Frequency performance measure assigns weighting factors to crashes by severity (fatal, injury, property damage only) to develop a combined frequency and severity score per site. The weighting factors are often calculated relative to Property Damage Only (PDO) crash costs. The crash costs by severity are summarized yielding an EPDO value. Although some agencies have developed weighting methods based on measures other than costs, crash costs are used consistently in the HSM to demonstrate use of the performance measure.

Crash costs include direct and indirect costs. Direct costs could include: ambulance service, police and fire services, property damage, or insurance. Indirect costs include the value society would place on pain and suffering or loss of life associated with the crash.

The strengths and limitations of the EPDO Average Crash Frequency performance measure include the following:

Chapter 31 Strengths	Chapter 32 Limitations
Simple	Does not account for RTM bias
Considers crash severity	Does not identify a threshold to indicate sites experiencing more crashes than predicted for sites with similar characteristics
	Does not account for traffic volume
	May overemphasize locations with a low frequency of severe crashes depending on weighting factors used

Relative Severity Index

Monetary crash costs are assigned to each crash type and the total cost of all crashes is calculated for each site. An average crash cost per site is then compared to an overall average crash cost for the site's reference population. The overall average crash cost is an average of the total costs at all sites in the reference population. The resulting Relative Severity Index (RSI) performance measure shows whether a site is experiencing higher crash costs than the average for other sites with similar characteristics.

The strengths and limitations of the RSI performance measure include the following:

Chapter 33 Strengths	Chapter 34 Limitations
Simple	Does not account for RTM bias
Considers collision type and crash severity	May overemphasize locations with a small number of severe crashes depending on weighting factors used
	Does not account for traffic volume
	Will mistakenly prioritize low-volume, low-collision sites

Critical Rate

The observed crash rate at each site is compared to a calculated critical crash rate that is unique to each site. The critical crash rate is a threshold value that allows for a relative comparison among sites with similar characteristics. Sites that exceed their respective critical rate are flagged for further review. The critical crash rate depends on the average crash rate at similar sites, traffic volume, and a statistical constant that represents a desired level of significance.

The strengths and limitations of the Critical Rate performance measure include the following:

Chapter 35 Strengths	Chapter 36 Limitations
Reduces exaggerated effect of sites with low volumes	Does not account for RTM bias
Considers variance in crash data	
Establishes a threshold for comparison	

Excess Predicted Average Crash Frequency Using Method of Moments

A site's observed average crash frequency is adjusted based on the variance in the crash data and average crash frequency for the site's reference population (4). The adjusted observed average crash frequency for the site is

compared to the average crash frequency for the reference population. This comparison yields the potential for improvement which can serve as a measure for ranking sites.

The strengths and limitations of the Excess Predicted Average Crash Frequency Using Method of Moments performance measure include the following:

Chapter 37 Strengths	Chapter 38 Limitations
Establishes a threshold of predicted performance for a site	Does not account for RTM bias
Considers variance in crash data	Does not account for traffic volume
Allows sites of all types to be ranked in one list	Some sites may be identified for further study because of unusually low frequency of non-target crash types
Method concepts are similar to Empirical Bayes methods	Ranking results are influenced by reference populations; sites near boundaries of reference populations may be over-emphasized

Level of Service of Safety (LOSS)

Sites are ranked according to a qualitative assessment in which the observed crash count is compared to a predicted average crash frequency for the reference population under consideration (1,4,5). Each site is placed into one of four LOSS classifications, depending on the degree to which the observed average crash frequency is different than predicted average crash frequency. The predicted average crash frequency for sites with similar characteristics is predicted from an SPF calibrated to local conditions.

The strengths and limitations of the LOSS performance measure include the following:

Chapter 39 Strengths	Chapter 40 Limitations
Considers variance in crash data	Effects of RTM bias may still be present in the results
Accounts for volume	
Establishes a threshold for measuring potential to reduce crash frequency	

Excess Predicted Average Crash Frequency Using Safety Performance Functions (SPFs)

The site's observed average crash frequency is compared to a predicted average crash frequency from an SPF. The difference between the observed and predicted crash frequencies is the excess predicted crash frequency using SPFs. When the excess predicted average crash frequency is greater than zero, a site experiences more crashes than predicted. When the excess predicted average crash frequency value is less than zero, a site experiences fewer crashes than predicted.

The strengths and limitations of the Excess Predicted Average Crash Frequency Using SPFs performance measure include the following:

Chapter 41 Strengths	Chapter 42 Limitations
Accounts for traffic volume	Effects of RTM bias may still be present in the results
Estimates a threshold for comparison	

Probability of Specific Crash Types Exceeding Threshold Proportion

Sites are prioritized based on the probability that the true proportion, pi, of a particular crash type or severity (e.g., long-term predicted proportion) is greater than the threshold proportion, p*i (6). A threshold proportion (p*i) is

selected for each population, typically based on the proportion of the target crash type or severity in the reference population. This method can also be applied as a diagnostic tool to identify crash patterns at an intersection or on a roadway segment (Chapter 5).

The following summarizes the strengths and limitations of the Probability of Specific Crash Types Exceeding Threshold Proportion performance measure:

Chapter 43 Strengths	Chapter 44 Limitations
Can also be used as a diagnostic tool (Chapter 5)	Does not account for traffic volume
Considers variance in data	Some sites may be identified for further study because of unusually low frequency of non-target crash types
Not affected by RTM Bias	

Excess Proportions of Specific Crash Types

This performance measure is very similar to the Probability of Specific Crash Types Exceeding Threshold Proportion performance measure except that sites are prioritized based on the excess proportion. The excess proportion is the difference between the observed proportion of a specific collision type or severity and the threshold proportion from the reference population. A threshold proportion (p*i) is selected for each population, typically based on the proportion of the target crash type or severity in the reference population. The largest excess value represents the most potential for reduction in average crash frequency. This method can also be applied as a diagnostic tool to identify crash patterns at an intersection or on a roadway segment (Chapter 5).

The strengths and limitations of the Excess Proportions of Specific Crash Types performance measure include the following:

Chapter 45 Strengths	Chapter 46 Limitations
Can also be used as a diagnostic tool	Does not account for traffic volume
Considers variance in data	Some sites may be identified for further study because of unusually low frequency of non-target crash types
Not effected by RTM Bias	

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

The observed average crash frequency and the predicted average crash frequency from an SPF are weighted together using the EB method to calculate an expected average crash frequency that accounts for RTM bias. Part C, Introduction and Applications Guidance provides a detailed presentation of the EB method. Sites are ranked from high to low based on the expected average crash frequency.

The following summarizes the strengths and limitations of the Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment performance measure:

Chapter 47 Strengths	Chapter 48 Limitations
Accounts for RTM bias	Requires SPFs calibrated to local conditions

Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

Crashes by severity are predicted using the EB procedure. Part C, Introduction and Applications Guidance provides a detailed presentation of the EB method. The expected crashes by severity are converted to EPDO crashes using the EPDO procedure. The resulting EPDO values are ranked. The EPDO Average Crash Frequency with EB Adjustments measure accounts for RTM bias and traffic volume.

The following summarizes the strengths and limitations of the EPDO Average Crash Frequency with EB Adjustment performance measure:

Chapter 49 Strengths	Chapter 50 Limitations
Accounts for RTM bias Considers crash severity	May overemphasize locations with a small number of severe crashes depending on weighting factors used

Excess Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

The observed average crash frequency and the predicted crash frequency from an SPF are weighted together using the EB method to calculate an expected average crash frequency. The resulting expected average crash frequency is compared to the predicted average crash frequency from a SPF. The difference between the EB adjusted average crash frequency and the predicted average crash frequency from an SPF is the excess expected average crash frequency.

When the excess expected crash frequency value is greater than zero, a site experiences more crashes than expected. When the excess expected crash frequency value is less than zero, a site experiences fewer crashes than expected.

The following summarizes the strengths and limitations of the Excess Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment performance measure:

Chapter 51 Strengths	Chapter 52 Limitations
Accounts for RTM bias Identifies a threshold to indicate sites experiencing more crashes than expected for sites with similar characteristics	Requires SPFs calibrated to local conditions

Network Screening Safety Performance Functions for Roundabouts

For those wishing to screen intersections with roundabouts, SPFs for roundabouts have been developed for network screening applications. These SPFs take into account land-use context (i.e., urban vs. rural area), the number of circulating lanes (i.e., single-lane vs. multilane), and major and minor street AADT volumes. These SPFs can be calibrated to local conditions and then used to screen a network of roundabouts using one of the previously described performance measures that require the use of a calibrated SPF (e.g., Excess Predicted Average Crash Frequency Using Safety Performance Functions). More information on the models, including the equations, can be found in Appendix 4B.

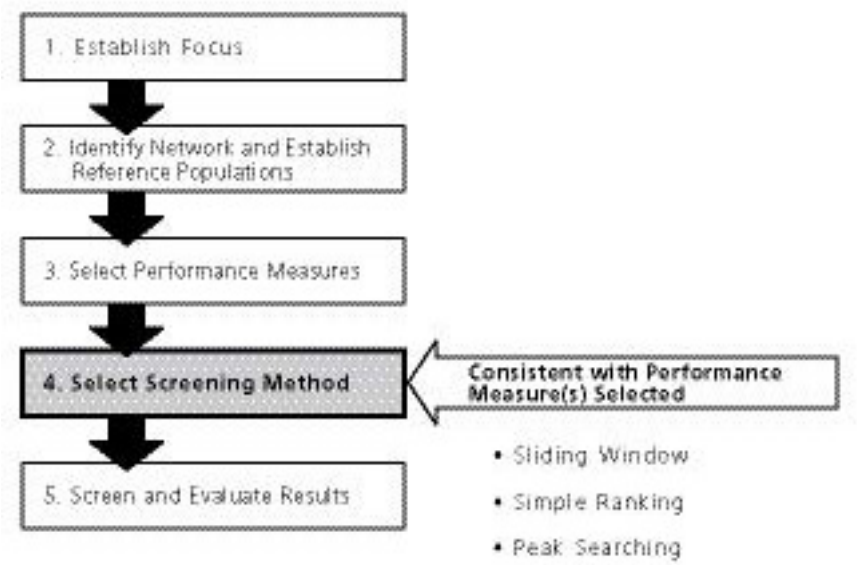
STEP 4—Select Screening Method

The fourth step in the network screening process is to select a network screening method (Figure 4-5). In a network screening process, the selected performance measure would be applied to all sites under consideration using a screening method. In the HSM, there are three types of three categories of screening methods:

Segments (e.g., roadway segment or ramp) are screened using either sliding window or peak searching methods.

Nodes (e.g., intersections or ramp terminal intersections) are screened using simple ranking method.

Facilities (combination of nodes and segments) are screened using a combination of segment and node screening methods.



▪ Figure 4-5. Network Screening Process—Step 4, Select Screening Method

Segment Screening Methods

Screening roadway segments and ramps requires identifying the location within the roadway segment or ramp that is most likely to benefit from a countermeasure intended to result in a reduction in crash frequency or severity. The location (i.e., subsegment) within a segment that shows the most potential for improvement is used to specify the critical crash frequency of the entire segment and subsequently select segments for further investigation. Having an understanding of what portion of the roadway segment controls the segment's critical crash frequency will make it easier and more efficient to identify effective countermeasures. Sliding window and peak searching methods can be used to identify the location within the segment which is likely to benefit from a countermeasure. The simple ranking method can also be applied to segments, but unlike sliding window and peak searching methods, performance measures are calculated for the entire length (typically 0.1 mi) of the segment.

Sliding Window Method

In the sliding window method a window of a specified length is conceptually moved along the road segment from beginning to end in increments of a specified size. The performance measure chosen to screen the segment is applied to each position of the window, and the results of the analysis are recorded for each window. A window pertains to a given segment if at least some portion of the window is within the boundaries of the segment. From all the windows that pertain to a given segment, the window that shows the most potential for reduction in crash frequency out of the whole segment is identified and is used to represent the potential for reduction in crash frequency of the whole segment. After all segments are ranked according to the respective highest subsegment value, those segments with the greatest potential for reduction in crash frequency or severity are studied in detail to identify potential countermeasures.

Windows will bridge two or more contiguous roadway segments in the sliding window method. Each window is moved forward incrementally until it reaches the end of a contiguous set of roadway segments. Discontinuities in contiguous roadway segments may occur as a result of discontinuities in route type, mileposts or routes, site characteristics, etc. When the window nears the end of a contiguous set of roadway segments, the window length

remains the same, while the increment length is adjusted so that the last window is positioned at the end of the roadway segment.

In some instances, the lengths of roadway segments may be less than the typical window length, and the roadway segments may not be part of a contiguous set of roadway segments. In these instances, the window length (typically 0.10-mi windows) equals the length of the roadway segment.

Sliding Window Method

Question
Segment A in the urban four-lane divided arterial reference population will be screened by the "Excess Predicted Average Crash Frequency Using SPFs" performance measure. Segment A is 0.60 mi long.

If the sliding window method is used to study this segment with a window of 0.30-mi and 0.10-mi increment, how many times will the performance measure be applied on Segment A?

The following table shows the results for each window. Which subsegment would define the potential for reduction in crash frequency or severity of the entire segment?

Example Application of Sliding Window Method

Chapter 53 Subsegment	Chapter 54 Window Position	Chapter 55 Excess Predicted Average Crash Frequency
A1	0.00 to 0.30 mi	1.20
A2	0.10 to 0.40 mi	0.80
A3	0.20 to 0.50 mi	1.10
A4	0.30 to 0.60 mi	1.90

Answer
As shown in the table, there are four 0.30 subsegments (i.e., window positions) on Segment A.

Subsegment 4 from 0.30 mi to 0.60 mi has a potential for reducing the average crash frequency by 1.90 crashes. This subsegment would be used to define the total segment crash frequency because this is the highest potential for reduction in crash frequency or severity of all four windows. Therefore, Segment A would be ranked and compared to other segments.

Peak Searching Method

In the peak searching method, each individual roadway segment is subdivided into windows of similar length, potentially growing incrementally in length until the length of the window equals the length of the entire roadway segment. The windows do not span multiple roadway segments. For each window, the chosen performance measure is calculated. Based upon the statistical precision of the performance measure, the window with the maximum value of the performance measure within a roadway segment is used to rank the potential for reduction in crashes of that site (i.e., whole roadway segment) relative to the other sites being screened.

The first step in the peak searching method is to divide a given roadway segment (or ramp) into 0.1-mi windows. The windows do not overlap, with the possible exception that the last window may overlap with the previous. If the segment is less than 0.1 mi in length, then the segment length equals the window length. The performance measure is then calculated for each window, and the results are subjected to precision testing. If the performance measure calculation for at least one subsegment satisfies the desired precision level, the segment is ranked based upon the

maximum performance measure from all of the windows that meet the desired precision level. If none of the performance measures for the initial 0.1-mi windows are found to have the desired precision, the length of each window is incrementally moved forward; growing the windows to a length of 0.2 mi. The calculations are performed again to assess the precision of the performance measures. The methodology continues in this fashion until a maximum performance measure with the desired precision is found or the window length equals the site length.

The precision of the performance measure is assessed by calculating the coefficient of variation (CV) of the performance measure.

$$\text{Coefficient of Variation (CV)} = \frac{\sqrt{Var(\text{Performance Measure})}}{\text{Performance Measure}} \qquad (4\text{-}1)$$

A large CV indicates a low level of precision in the estimate, and a small CV indicates a high level of precision in the estimate. The calculated CV is compared to a specified limiting CV. If the calculated CV is less than or equal to the CV limiting value, the performance measure meets the desired precision level, and the performance measure for a given window can potentially be considered for use in ranking the segment. If the calculated CV is greater than the CV limiting value, the window is automatically removed from further consideration in potentially ranking the segment based upon the value of the performance measure.

There is no specific CV value that is appropriate for all network screening applications. However, by adjusting the CV value the user can vary the number of sites identified by network screening as candidates for further investigation. An appropriate initial or default value for the CV is 0.5.

Peak Searching Method

Question
Segment B, in an urban four-lane divided arterial reference population, will be screened using the Excess Expected Average Crash Frequency performance measure. Segment B is 0.47 mi long. The CV limiting value is assumed to be 0.25. If the peak searching method is used to study this segment, how is the methodology applied and how is the segment potentially ranked relative to other sites considered in the screening?

Answer

Iteration #1
The following table shows the results of the first iteration. In the first iteration, the site is divided into 0.1-mi windows. For each window, the performance measure is calculated along with the CV.

The variance is given as:

$$VAR_B = \frac{(5.2-5.7)^2+(7.8-5.7)^2+(1.1-5.7)^2+(6.5-5.7)^2+(7.8-5.7)^2}{(5-1)} = 7.7$$

The Coefficient of Variation for Segment B1 is calculated using Equation 4-1 as shown below:

$$CV_{B1} = \frac{\sqrt{7.7}}{5.2} = 0.53$$

Example Application of Expected Average Crash Frequency with Empirical Bayes Adjustment (Iteration #1)

Chapter 56 Subsegment	Chapter 57 Window Position	Chapter 58 Excess Expected Average Crash Frequency	Chapter 59 Coefficient of Variation (CV)
B1	0.00 to 0.10 mi	5.2	0.53
B2	0.10 to 0.20 mi	7.8	0.36
B3	0.20 to 0.30 mi	1.1	2.53
B4	0.30 to 0.40 mi	6.5	0.43
B5	0.37 to 0.47 mi	7.8	0.36
	Average	5.7	--

Because none of the calculated CVs are less than the CV limiting value, none of the windows meet the screening criterion, so a second iteration of the calculations is required.

Iteration #2

The following shows the results of the second iteration. In the second iteration, the site is analyzed using 0.2-mi windows. For each window, the performance measure is calculated along with the CV.

Example Application of Expected Average Crash Frequency with Empirical Bayes Adjustment (Iteration #2)

Chapter 60 Subsegment	Chapter 61 Window Position	Chapter 62 Excess Expected Average Crash Frequency	Chapter 63 Coefficient of Variation (CV)
B1	0.00 to 0.20 mi	6.50	0.25
B2	0.10 to 0.30 mi	4.45	0.36
B3	0.20 to 0.40 mi	3.80	0.42
B4	0.27 to 0.47 mi	7.15	0.22
	Average	5.5	---

In this second iteration, the CVs for subsegments B1 and B4 are less than or equal to the CV limiting value of 0.25. Segment B would be ranked based upon the maximum value of the performance measures calculated for subsegments B1 and B4. In this instance, Segment B would be ranked and compared to other segments according to the 7.15 Excess Expected Crash Frequency calculated for subsegment B4.

If during Iteration 2, none of the calculated CVs were less than the CV limiting value, a third iteration would have been necessary with 0.3-mi window lengths, and so on, until the final window length considered would be equal to the segment length of 0.47 mi.

Simple Ranking Method

A simple ranking method can be applied to nodes and segments. In this method, the performance measures are calculated for all of the sites under consideration, and the results are ordered from high to low. The simplicity of this

method is the greatest strength. However, for segments, the results are not as reliable as the other segment screening methods.

Node-Based Screening

Node-based screening focuses on intersections, ramp terminal intersections, and at-grade rail crossings. A simple ranking method may be applied whereby the performance measures are calculated for each site, and the results are ordered from high to low. The outcome is a list showing each site and the value of the selected performance measure. All of the performance measures can be used with simple ranking for node-based screening.

A variation of the peak searching method can be applied to intersections. In this variation, the precision test is applied to determine which performance measure to rank upon. Only intersection-related crashes are included in the node-based screening analyses.

Facility Screening

A facility is a length of highway composed of connected roadway segments and intersections. When screening facilities, the connected roadway segments are recommended to be approximately 5 to 10 mi in length. This length provides for more stable results.

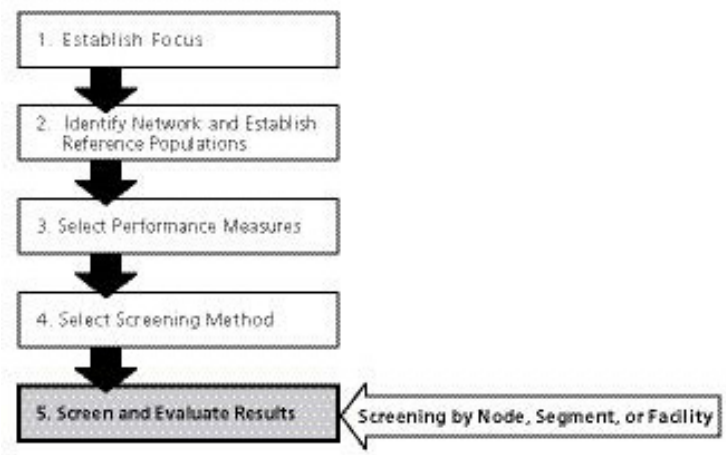
Table 4-3 summarizes the performance measures that are consistent with the screening methods.

- Table 4-3. Performance Measure Consistency with Screening Methods

Chapter 64	Chapter 65 Segments			Chapter 66 Nodes	Chapter 67 Facilities
Chapter 68 Performance Measure	Chapter 69 Simple Ranking	Chapter 70 Sliding Window	Chapter 71 Peak Searching	Chapter 72 Simple Ranking	Chapter 73 Simple Ranking
Average Crash Frequency	Yes	Yes	No	Yes	Yes
Crash Rate	Yes	Yes	No	Yes	Yes
Equivalent Property Damage Only (EPDO) Average Crash Frequency	Yes	Yes	No	Yes	Yes
Relative Severity Index	Yes	Yes	No	Yes	No
Critical Crash Rate	Yes	Yes	No	Yes	Yes
Excess Predicted Average Crash Frequency Using Method of Moments	Yes	Yes	No	Yes	No
Level of Service of Safety	Yes	Yes	No	Yes	No
Excess Predicted Average Crash Frequency Using SPFs	Yes	Yes	No	Yes	No
Probability of Specific Crash Types Exceeding Threshold Proportion	Yes	Yes	No	Yes	No
Excess Proportions of Specific Crash Types	Yes	Yes	No	Yes	No
Expected Average Crash Frequency with EB Adjustments	Yes	Yes	Yes	Yes	No
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment	Yes	Yes	Yes	Yes	No
Excess Expected Average Crash Frequency with EB Adjustments	Yes	Yes	Yes	Yes	No

STEP 5—Screen and Evaluate Results

The performance measure and the screening method are applied to one or more of the segments, nodes, or facilities according to the methods outlined in Steps 3 and 4. Conceptually, for each segment or node under consideration, the selected performance measure is calculated and recorded (see Figure 4-6). Results can be recorded in a table or on maps as appropriate or feasible.



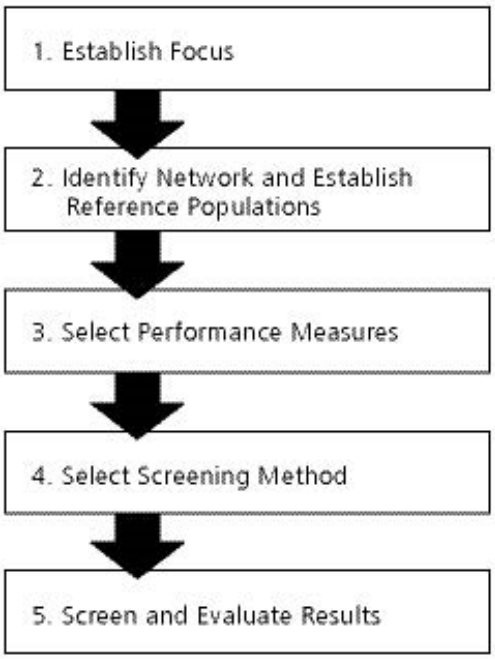
- Figure 4-6. Optional Methods for Network Screening

The results of the screening analysis will be a list of sites ordered according to the selected performance measure. Those sites higher on the list are considered most likely to benefit from countermeasures intended to reduce crash frequency. Further study of these sites will indicate what kinds of improvements are likely to be most effective (see Chapters 5, 6, and 7).

In general, it can be useful to apply multiple performance measures to the same data set. In doing so, some sites will repeatedly be at the high or low end of the resulting list. Sites that repeatedly appear at the higher end of the list could become the focus of more detailed site investigations, while those that appear at the low end of the list could be ruled out for needing further investigation. Differences in the rankings produced by the various performance measures will become most evident at sites which are ranked in the middle of the list.

SUMMARY

This chapter explains the five steps of the network screening process, illustrated in Figure 4-7, that can be applied with one of three screening methods for conducting network screening. The results of the analysis are used to determine the sites that are studied in further detail. The objective of studying these sites in more detail is to identify crash patterns and the appropriate countermeasures to reduce the number of crashes; these activities are discussed in Chapters 5, 6, and 7.



- Figure 4-7. Network Screening Process

When selecting a performance measure and screening method, there are three key considerations. The first is related to the data that is available or can be collected for the study. It is recognized that this is often the greatest constraint; therefore, methods are outlined in the chapter that do not require a significant amount of data.

The second and third considerations relate to the performance of the methodology results. The most accurate study methodologies provide for the ability to: 1) account for regression-to-the-mean bias, and 2) estimate a threshold level of performance in terms of crash frequency or crash severity. These methods can be trusted with a greater level of confidence than those methods that do not.

Section 4.4 provides a detailed overview of the procedure for calculating each of the performance measures in this chapter. The section also provides step-by-step sample applications for each method applied to intersections. These same steps can be used on ramp terminal intersections and at-grade rail crossings. Section 4.4 also provides step-by-step sample applications demonstrating use of the peak searching and sliding window methods to roadway segments. The same steps can be applied to ramps.

PERFORMANCE MEASURE METHODS AND SAMPLE APPLICATIONS

Intersection Performance Measure Sample Data

The following sections provide sample data to be used to demonstrate application of each performance measure.

Sample Situation

A roadway agency is undertaking an effort to improve safety on their highway network. They are screening twenty intersections to identify sites with potential for reducing the crash frequency.

The Facts

All of the intersections have four approaches and are in rural areas;

Thirteen are signalized intersections and 7 are unsignalized (two-way stop controlled) intersections;

Major and Minor Street AADT volumes are provided in Table 4-4;

A summary of crash data over the same three years as the traffic volumes is shown in Table 4-5; and

Three years of detailed intersection crash data is shown in Table 4-6.

Assumptions

The roadway agency has locally calibrated Safety Performance Functions (SPFs) and associated overdispersion parameters for the study intersections. Predicted average crash frequency from an SPF is provided in Table 4-6 for the sample intersections.

The roadway agency supports use of FHWA crash costs by severity and type.

Intersection Characteristics and Crash Data

Tables 4-4 and 4-5 summarize the intersection characteristics and crash data.

- Table 4-4. Intersection Traffic Volumes and Crash Data Summary

Chapter 74	Chapter 75	Chapter 76	Chapter 77	Chapter 78	Chapter 79 Crash Data		
Chapter 80 Intersections	Chapter 81 Traffic Control	Chapter 82 Number of Approaches	Chapter 83 Major AADT	Chapter 84 Minor AADT	Chapter 85 Total Year 1	Chapter 86 Total Year 2	Chapter 87 Total Year 3
1	Signal	4	30,100	4,800	9	8	5
2	TWSC	4	12,000	1,200	9	11	15
3	TWSC	4	18,000	800	9	8	6
4	Signal	4	11,200	10,900	8	2	3
5	Signal	4	30,700	18,400	3	7	5
6	Signal	4	31,500	3,600	6	1	2
7	TWSC	4	21,000	1,000	11	9	14
8	Signal	4	23,800	22,300	2	4	3
9	Signal	4	47,000	8,500	15	12	10
10	TWSC	4	15,000	1,500	7	6	4
11	Signal	4	42,000	1,950	12	15	11
12	Signal	4	46,000	18,500	10	14	8
13	Signal	4	11,400	11,400	4	1	1
14	Signal	4	24,800	21,200	5	3	2
15	TWSC	4	26,000	500	6	3	8
16	Signal	4	12,400	7,300	7	11	3
17	TWSC	4	14,400	3,200	4	4	5
18	Signal	4	17,600	4,500	2	10	7
19	TWSC	4	15,400	2,500	5	2	4
20	Signal	4	54,500	5,600	4	2	2

▪ Table 4-5. Intersection Detailed Crash Data Summary (3 Years)

Chapter 88	Chapter	Chapter 90 Crash Severity			Chapter 91 Crash Type							
Chapter 92 Intersections	Chapter otal	Chapter atal	Chapter 9 njury	Chapter DO	Chapter ear-End	Chapter 98 Si deswipe/Overtaking	Chapter 9 ight Angle	Chapte ed	Chapter ike	Chapter 1 ead-On	Chapter 1 ixed Object	Chapter 1 ther
1	22	0	6	16	11	4	4	0	0	0	1	2
2	35	2	23	10	4	2	21	0	2	5	0	1
3	23	0	13	10	11	5	2	1	0	0	4	0
4	13	0	5	8	7	2	3	0	0	0	1	0
5	15	0	4	11	9	4	2	0	0	0	0	0
6	9	0	2	7	3	2	3	0	0	0	1	0
7	34	1	17	16	19	7	5	0	0	0	3	0
8	9	0	2	7	4	3	1	0	0	0	0	1
9	37	0	22	15	14	4	17	2	0	0	0	0
10	17	0	7	10	9	4	2	0	0	0	1	1
11	38	1	19	18	6	5	23	0	0	4	0	0
12	32	0	15	17	12	2	14	1	0	2	0	1
13	6	0	2	4	3	1	2	0	0	0	0	0
14	10	0	5	5	5	1	1	1	0	0	1	1
15	17	1	4	12	9	4	1	0	0	0	1	2
16	21	0	11	10	8	4	7	0	0	0	1	1
17	13	1	5	7	6	2	2	0	0	1	0	2
18	19	0	8	11	8	7	3	0	0	0	0	1
19	11	1	5	5	5	4	0	1	0	0	0	1
20	8	0	3	5	2	3	2	0	0	0	1	0

▪ Table 4-6. Estimated Predicted Average Crash Frequency from an SPF

Chapter 105 ntersection	Chapter ear	Chapter 107 AADT Chapter 110 ajor Street	Chapter 111 inor Street	Chapter 108 Predicted Average Crash Frequency from an SPF	Chapter 109 Average 3-Year Predicted Crash Frequency from an SPF
	1	12,000	1,200	1.7	
2	2	12,200	1,200	1.7	1.7
	3	12,900	1,300	1.8	
	1	18,000	800	2.1	
3	2	18,900	800	2.2	2.2
	3	19,100	800	2.2	
	1	21,000	1,000	2.5	
7	2	21,400	1,000	2.5	2.6
	3	22,500	1,100	2.7	
	1	15,000	1,500	2.1	
10	2	15,800	1,600	2.2	2.2
	3	15,900	1,600	2.2	
	1	26,000	500	2.5	
15	2	26,500	300	2.2	2.3
	3	27,800	200	2.1	
	1	14,400	3,200	2.5	
17	2	15,100	3,400	2.6	2.6
	3	15,300	3,400	2.6	
	1	15,400	2,500	2.4	
19	2	15,700	2,500	2.5	2.5
	3	16,500	2,600	2.6	

Intersection Performance Measure Methods

The following sections provide step-by-step procedures for applying the performance measures described in Section 4.2.3, which provides guidance for selecting an appropriate performance measure.

Average Crash Frequency

Applying the Crash Frequency performance measure produces a simple ranking of sites according to total crashes or crashes by type or severity, or both. This method can be used to select an initial group of sites with high crash frequency for further analysis.

Data Needs

Crash data by location

Strengths and Limitations

The strengths and limitations of the Crash Frequency performance measure include the following:

Chapter 112	Strengths	Chapter 113	Limitations

Simple	Does not account for RTM bias Does not estimate a threshold to indicate sites experiencing more crashes than predicted for sites with similar characteristics Does not account for traffic volume Will not identify low-volume collision sites where simple cost-effective mitigating countermeasures could be easily applied

Procedure

Chapter 114	S
TEP 1—Sum Crashes for Each Location	**Average Crash Frequency**
	1 2

Count the number of crashes that occurred at each intersection.

Chapter 115	S
TEP 2—Rank Locations	**Average Crash Frequency**
	1 2

The intersections can be ranked in descending order by the number of one or more of the following: total crashes, fatal and injury crashes, or PDO crashes.

Ranking of the 20 sample intersections is shown in the table. Column A shows the ranking by total crashes, Column B is the ranking by fatal and injury crashes, and Column C is the ranking by property damage-only crashes.

As shown in the table, ranking based on crash severity may lead to one intersection achieving a different rank depending on the ranking priority. The rank of Intersection 1 demonstrates this variation.

Chapter 116	Column A	Chapter 117	Column B	Chapter 118	Column C
Chapter 119 I ntersection	Chapter 120 otal Crashes	Chapter 121 I ntersection	Chapter 122 atal and Injury	Chapter 123 I ntersection	Chapter 124 DO Crashes
11	38	2	25	11	18
9	37	9	22	12	17
2	35	11	20	1	16
7	34	7	18	7	16
12	32	12	15	9	15
3	23	3	13	15	12
1	22	16	11	5	11
16	21	18	8	18	11
18	19	10	7	2	10
10	17	1	6	3	10
15	17	17	6	10	10
5	15	19	6	16	10
4	13	4	5	4	8
17	13	14	5	6	7
19	11	15	5	8	7
14	10	5	4	17	7

6	9	20	3	14	5
8	9	6	2	19	5
20	8	8	2	20	5
13	6	13	2	13	4

Crash Rate

The crash rate performance measure normalizes the number of crashes relative to exposure (traffic volume) by dividing the total number of crashes by the traffic volume. The traffic volume includes the total number of vehicles entering the intersection, measured as million entering vehicles (MEV).

Data Needs

Crashes by location

Traffic Volume

Strengths and Limitations

The strengths and limitations of the Crash Rate performance measure include the following:

Chapter 125 Strengths	Chapter 126 Limitations
Simple	Does not account for RTM bias
Could be modified to account for severity if an EPDO or RSI-based crash count is used	Does not identify a threshold to indicate sites experiencing more crashes than predicted for sites with similar characteristics
	Comparisons cannot be made across sites with significantly different traffic volumes
	Will mistakenly prioritize low-volume, low-collision sites

Procedure

The following outlines the assumptions and procedure for ranking sites according to the crash rate method. The calculations for Intersection 7 are used throughout the remaining sample problems to highlight how to apply each method.

Chapter 127 **S**

TEP 1—Calculate MEV **Crash Rate**

	1	2	3

Calculate the million entering vehicles for all 3 years. Use Equation 4-2 to calculate the exposure in terms of million entering vehicles (MEV) at an intersection.

$$MEV = \frac{TEV}{1,000,000} \times (n) \times (365) \qquad (4\text{-}2)$$

Where:

MEV= Million entering vehicles

TEV = Total entering vehicles per day

n = Number of years of crash data

Total Entering Vehicles

This table summarizes the total entering volume (TEV) for all sample intersections. The TEV is a sum of the major and minor street AADT found in Table 4-4.

TEV is converted to MEV as shown in the following equation for Intersection 7:

$$MEV = \left(\frac{22,000}{1,000,000}\right) \times (3) \times (365) = 24.1$$

Total Entering Vehicles

Chapter 128	Intersection	Chapter 129	TEV/day	Chapter 130	MEV
1		34900		38.2	
2		13200		14.5	
3		18800		20.6	
4		22100		24.2	
5		49100		53.8	
6		35100		38.4	
7		22000		24.1	
8		46100		50.5	
9		55500		60.8	
10		16500		18.1	
11		43950		48.1	
12		64500		70.6	
13		22800		25.0	
14		46000		50.4	
15		26500		29.0	
16		19700		21.6	
17		17600		19.3	
18		22100		24.2	
19		17900		19.6	
20		60100		65.8	

Calculate the crash rate for each intersection by dividing the total number of crashes by MEV for the 3-year study period as shown in Equation 4-3.

$$R_i = \frac{N_{observed,\,i\,(total)}}{MEV_i} \qquad (4\text{-}3)$$

Where:

R_i = Observed crash rate at intersection i

$N_{observed,i(total)}$ = Total observed crashes at intersection i

MEVi = Million entering vehicles at intersection i

Below is the crash rate calculation for Intersection 7. The total number of crashes for each intersection is summarized in Table 4-5.

$$\text{Crash Rate} = \frac{34}{24.1} = 1.4$$ [crashes/MEV]

Chapter 132 Step 3—Rank Intersections Crash Rate

Rank the intersections based on their crash rates.

This table summarizes the results from applying the crash rate method.

Ranking Based on Crash Rates

Chapter 133 Intersection	Chapter 134 Crash Rate
2	2.4
7	1.4
3	1.1
16	1.0
10	0.9
11	0.8
18	0.8
17	0.7
9	0.6
15	0.6
1	0.6
19	0.6
4	0.5
12	0.5
5	0.3
13	0.2
6	0.2

14	0.2
8	0.2
20	0.1

Equivalent Property Damage Only (EPDO) Average Crash Frequency

The Equivalent Property Damage Only (EPDO) Average Crash Frequency performance measure assigns weighting factors to crashes by severity to develop a single combined frequency and severity score per location. The weighting factors are calculated relative to Property Damage Only (PDO) crashes. To screen the network, sites are ranked from the highest to the lowest score. Those sites with the highest scores are evaluated in more detail to identify issues and potential countermeasures.

This method is heavily influenced by the weighting factors for fatal and injury crashes. A large weighting factor for fatal crashes has the potential to rank sites with one fatal crash and a small number of injury or PDO crashes, or both, above sites with no fatal crashes and a relatively high number of injury or PDO crashes, or both. In some applications, fatal and injury crashes are combined into one category of Fatal/Injury (FI) crashes to avoid overemphasizing fatal crashes. Fatal crashes are tragic events; however, the fact that they are fatal is often the outcome of factors (or a combination of factors) that is out of the control of the engineer and planner.

Data Needs

Crash data by severity and location

Severity weighting factors

Crash costs by crash severity

Strengths and Limitations

The strengths and limitations of the EPDO Average Crash Frequency performance measure include the following:

Chapter 135 Strengths	Chapter 136 Limitations
Simple	Does not account for RTM bias
Considers crash severity	Does not identify a threshold to indicate sites experiencing more crashes than predicted for sites with similar characteristics
	Does not account for traffic volume
	May overemphasize locations with a low frequency of severe crashes depending on weighting factors used

Procedure for Applying the EPDO Average Crash Frequency Performance Measure

Societal crash costs are used to calculate the EPDO weights. State and local jurisdictions often have accepted societal crash costs by type or severity, or both. When available, locally developed crash cost data is preferred. If local information is not available, national crash cost data is available from the Federal Highway Administration (FHWA). In order to improve acceptance of study results that use monetary values, it is important that monetary values be reviewed and endorsed by the jurisdiction in which the study is being conducted.

The FHWA report Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, prepared in October 2005, documented mean comprehensive societal costs by severity as listed in Table 4-7 (rounded to the nearest hundred dollars) (2). As of December 2008, this was the most recent FHWA crash cost information, although these costs represent 2001 values.

Appendix 4A includes a summary of crash costs and outlines a process to update monetary values to current year values.

▪ Table 4-7. Societal Crash Cost Assumptions

Chapter 137	Severity	Chapter 138	Comprehensive Crash Cost (2001 Dollars)
Fatal (K)		$4,008,900	
Injury Crashes (A/B/C)		$82,600	
PDO (O)		$7,400	

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA-HRT-05-051, October 2005

The values in Table 4-7 were published in the FHWA study. A combined disabling (A), evident (B), and possible (C) injury crash cost was provided by FHWA to develop an average injury (A/B/C) cost. Injury crashes could also be subdivided into disabling injury, evident injury, and possible injury crashes depending on the amount of detail in the crash data and crash costs available for analysis.

Chapter 139 **S**

TEP 1—Calculate EDPO Weights **Equivalent Property Damage Only (EPDO) Average Crash Frequency**

1 2 3

Calculate the EPDO weights for fatal, injury, and PDO crashes. The fatal and injury weights are calculated using Equation 4-4. The cost of a fatal or injury crash is divided by the cost of a PDO crash, respectively. Weighting factors developed from local crash cost data typically result in the most accurate results. If local information is not available, nationwide crash cost data is available from the Federal Highway Administration (FHWA). Appendix 4A provides more information on the national data available.

The weighting factors are calculated as follows:

$$f_{y(weight)} = \frac{CC_y}{CC_{PDO}} \qquad (4\text{-}4)$$

Where:

$f_{y(weight)}$ = Weighting factor based on crash severity, y

CC_y = Crash cost for crash severity, y

CC_{PDO} = Crash cost for PDO crash severity

As shown, a sample calculation for the injury (A/B/C) EPDO weight ($f_{inj(weight)}$) is:

$$f_{y(weight)} = \frac{CC_y}{CC_{PDO}}$$

Therefore, the weighting factors for all crash severities are shown in the following table:

Sample EPDO Weights

Chapter 140	Severity	Chapter 141	Cost	Chapter 142	Weight
Fatal (K)		$4,008,900		542	

Injury (A/B/C)	$82,600	11
PDO (O)	$7,400	1

Chapter 143 **S**

TEP 2—Calculate EPDO Scores ***Equivalent Property Damage Only (EPDO) Average Crash Frequency***

1 2 3

For each intersection, multiply the EPDO weights by the corresponding number of fatal, injury, and PDO crashes as shown in Equation 4-5. The frequency of PDO, Injury, and Fatal crashes is based on the number of crashes, not the number of injuries per crash.

$$Total\ EPDO\ Score = f_{k(weight)}(N_{observed,i(F)}) + f_{inj(weight)}(N_{observed,i(I)}) + f_{PDO(weight)}(N_{observed,i(PDO)}) \quad (4\text{-}5)$$

Where:

$f_{k(weight)}$ = Fatal Crash Weight

$N_{observed,i(F)}$ = Number of Fatal Crashes per intersection, i

$f_{inj(weight)}$ = Injury Crash Weight

$N_{observed,i(I)}$ = Number of Injury Crashes per intersection, i

$f_{PDO(weight)}$ = PDO Crash Weight

$N_{observed,i(PDO)}$ = Number of PDO Crashes per intersection, i

Chapter 144 **S**

TEP 3—Rank Locations **Equivalent Property Damage Only (EPDO) Average Crash Frequency**

1 2 3

The intersections can be ranked in descending order by the EPDO score.

As shown, the calculation of EPDO Score for Intersection 7 is

Total EPDO $Score_7$ = (542 × 1) + (11 × 17) + (1 × 16) = 745

The number of fatal, injury, and PDO crashes for each intersection were shown in the example box in Section 4.4.2.1. The table below summarizes the EPDO score.

The calculation is repeated for each intersection.

The ranking for the 20 intersections is based on EPDO method. The results of calculations for Intersection 7 are highlighted.

Sample EPDO Ranking

Chapter 145	Intersection	Chapter 146 Score	EPDO
2		1347	
11		769	
7		745	
17		604	
19		602	
15		598	
9		257	
12		182	
3		153	
16		131	
18		99	
10		87	
1		82	
4		63	
14		60	
5		55	
20		38	
6		29	
8		29	
13		26	

Missing subhead?

Relative Severity Index (RSI)

Jurisdiction-specific societal crash costs are developed and assigned to crashes by crash type and location. These societal crash costs make up a relative severity index. Relative Severity Index (RSI) crash costs are assigned to each crash at each site based on the crash type. An average RSI crash cost is calculated for each site and for each population. Sites are ranked based on their average RSI cost and are also compared to the average RSI cost for their respective population.

Data Needs

Crashes by type and location

RSI Crash Costs

Strengths and Limitations

The strengths and limitations of the RSI performance measure include the following:

Chapter 147	Strengths	Chapter 148	Limitations
Simple		Does not account for RTM bias	
Considers collision type and crash severity		May overemphasize locations with a small number of severe crashes depending on weighting factors used	

Does not account for traffic volume

Will mistakenly prioritize low-volume, low-collision sites

Procedure

The RSI costs listed in Table 4-8 are used to calculate the average RSI cost for each intersection and the average RSI cost for each population. The values shown represent 2001 dollar values and are rounded to the nearest hundred dollars. Appendix 4A provides a method for updating crash costs to current year values.

Table 4-8. Crash Cost Estimates by Crash Type

Chapter 149 Crash Type	Chapter 150 Crash Cost (2001 Dollars)
Rear-End, Signalized Intersection	$26,700
Rear-End, Unsignalized Intersection	$13,200
Sideswipe/Overtaking	$34,000
Angle, Signalized Intersection	$47,300
Angle, Unsignalized Intersection	$61,100
Pedestrian/Bike at an Intersection	$158,900
Head-On, Signalized Intersection	$24,100
Head-On, Unsignalized Intersection	$47,500
Fixed Object	$94,700
Other/Undefined	$55,100

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA-HRT-05-051, October 2005

Chapter 151 **S**

TEP 1—Calculate RSI Costs per Crash Type **Relative Severity Index (RSI)**

1 2 3 4

For each intersection, multiply the observed average crash frequency for each crash type by their respective RSI crash cost.

The RSI crash cost per crash type is calculated for each location under consideration. The following example contains the detailed summary of the crashes by type at each intersection.

This table summarizes the number of crashes by crash type at Intersection 7 over the last three years and the corresponding RSI costs for each crash type.

Intersection 7 Relative Severity Index Costs

Chapter 152 Intersection 7	Chapter 153 Number of Observed Crashes	Chapter 154 Crash Costs	Chapter 155 RSI Costs
Rear-End, Unsignalized Intersection	19	$13,200	$250,800
Sideswipe Crashes, Unsignalized Intersection	7	$34,000	$238,000
Angle Crashes, Unsignalized Intersection	5	$61,100	$305,500
Fixed Object Crashes, Unsignalized Intersection	3	$94,700	$284,100
Total RSI Cost for Intersection 7			$1,078,400

Note: Crash types that were not reported to have occurred at Intersection 7 were omitted from the table; the RSI value for these crash types is zero.

Sum the RSI crash costs for all crash types and divide by the total number of crashes at the intersection to arrive at an average RSI value for each intersection.

$$\overline{RSI_{av(control)}} = \frac{\sum_{i=1}^{n} RSI_i}{\sum_{i=1}^{n} N_{observed,i}} \qquad (4\text{-}6)$$

Where:

= Average RSI cost for the intersection, i

RSI_j = RSI cost for each crash type, j

$N_{observed,i}$ = Number of observed crashes at the site i

The RSI calculation for Intersection 7 is as follows:

$$\overline{RSI_7} = \frac{\$1,078,400}{34} = \$31,700$$

1 2 3 4

Calculate the average RSI cost for the population (the control group) by summing the total RSI costs for each site and dividing by the total number of crashes within the population.

$$\overline{RSI_{av(control)}} = \frac{\sum_{i=1}^{n} RSI_i}{\sum_{i=1}^{n} N_{observed,i}} \qquad (4\text{-}7)$$

Where:

= Average RSI cost for the reference population (control group)

RSIi = Total RSI cost at site i

Nobserved,i = number of observed crashes at site i

Section 1

In this sample problem, Intersection 7 is in the unsignalized intersection population. Therefore, illustrated below is the calculation for the average RSI cost for the unsignalized intersection population.

The average RSI cost for the population () is calculated using Table 4-8. The following table summarizes the information needed to calculate the average RSI cost for the population:

Chapter 158 nsignalized Intersection	Chapter 159 ear-End	Chapter 160 ideswipe	Chapter 161 ngle	Chapter 162 ed/Bike	Chapter 163 ead-On	Chapter 164 ixed Object	Chapter 165 ther	Chapter 166 otal
Chapter 167	Number of Crashes over Three Years							
2	4	2	21	2	5	0	1	35
3	11	5	2	1	0	4	0	23
7	19	7	5	0	0	3	0	34
10	9	4	2	0	0	1	1	17
15	9	4	1	0	0	1	2	17
17	6	2	2	0	1	0	2	13
19	5	4	0	1	0	0	1	11
Total Crashes in Unsignalized Intersection Population								150
Chapter 168	RSI Crash Costs per Crash Type							
2	$52,800	$68,000	$1,283,100	$317,800	$237,500	$0	$55,100	$2,014,300
3	$145,200	$170,000	$122,200	$158,900	$0	$378,800	$0	$975,100
7	$250,800	$238,000	$305,500	$0	$0	$284,100	$0	$1,078,400
10	$118,800	$136,000	$122,200	$0	$0	$94,700	$55,100	$526,800
15	$118,800	$136,000	$61,100	$0	$0	$94,700	$110,200	$520,800
17	$79,200	$68,000	$122,200	$0	$47,500	$0	$110,200	$427,100

19	$66,000	$136,000	$0	$158,900	$0	$0	$55,100	$416,000
Sum of Total RSI Costs for Unsignalized Intersections								$5,958,500
Average RSI Cost for Unsignalized Intersections ($5,958,500/150)								$39,700

The average RSI costs are calculated by dividing the RSI crash cost for each intersection by the number of crashes for the same intersection. The average RSI cost per intersection is also compared to the average RSI cost for its respective population.

Section 2

The following table shows the intersection ranking for all 20 intersections based on their average RSI costs. The RSI costs for Intersection 7 would be compared to the average RSI cost for the unsignalized intersection population. In this instance, the average RSI cost for Intersection 7 ($31,700) is less than the average RSI cost for all unsignalized intersections ($39,700 from calculations in Step 3).

Ranking Based on Average RSI Cost per Intersection

Chapter 170 Intersection	Chapter 171 RSI Cost[a]	Average	Chapter 172 RSI_P	Exceeds
2	$57,600		X	
14	$52,400		X	
6	$42,800		X	
9	$44,100		X	
20	$43,100		X	
3	$42,400		X	
4	$42,000		X	
12	$41,000		X	
11	$39,900		X	
16	$39,500			
19	$37,800			
1	$37,400			
13	$34,800			
8	$34,600			
18	$34,100			
17	$32,900			
7	$31,700			
5	$31,400			
10	$31,000			
15	$30,600			

a Average RSI Costs per Intersection are rounded to the nearest $100.

Critical Rate

The observed crash rate at each site is compared to a calculated critical crash rate that is unique to each site. Sites that exceed their respective critical rate are flagged for further review. The critical crash rate depends on the average crash rate at similar sites, traffic volume, and a statistical constant that represents a desired confidence level.

Data Needs

Crashes by location

Traffic Volume

Strengths and Limitations

The strengths and limitations of the performance measure include the following:

Chapter 173	Strengths	Chapter 174	Limitations
Reduces exaggerated effect of sites with low volumes		Does not account for RTM bias	
Considers variance in crash data			
Establishes a threshold for comparison			

Procedure

The following outlines the assumptions and procedure for applying the critical rate method. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

Assumptions

Calculations in the following steps were conducted using a P-value of 1.645 which corresponds to a 95 percent confidence level. Other possible confidence levels, based on a Poisson distribution and one-tailed standard normal random variable, are shown in Table 4-9.

- Table 4-9. Confidence Levels and P Values for Use in Critical Rate Method

Chapter 175	Confidence Level	Chapter 176	Pc—Value
85 percent		1.036	
90 percent		1.282	
95 percent		1.645	
99 percent		2.326	
99.5 percent		2.576	

Source: *Road Safety Manual,* PIARC Technical Committee on Road Safety, 2003, p. 113

Calculate the volume in terms of million entering vehicles for all 3 years. Equation 4-8 is used to calculate the million entering vehicles (MEV) at an intersection.

(4-8)

Where:

MEV = Million entering vehicles

TEV = Total entering vehicles per day

n = Number of years of crash data

Section 4

Shown below is the calculation for the MEV of Intersection 7. The TEV is found in Table 4-4.

Chapter 178 **S**

TEP 2—Calculate the Crash Rate for Each Intersection **Critical Rate**

Calculate the crash rate for each intersection by dividing the number of crashes by MEV, as shown in Equation 4-9.

(4-9)

Where:

Ri = Observed crash rate at intersection i

Nobserved,i(total) = Total observed crashes at intersection i

MEVi = Million entering vehicles at intersection i

Section 5

Below is the crash rate calculation for Intersection 7. The total number of crashes for each intersection is summarized in Table 4-5, and the MEV is noted in Step 1.

[crashes/MEV]

Section 6

Chapter 179 **S**

TEP 3—Calculate Weighted Average Crash Rate per Population **Critical Rate**

Divide the network into reference populations based on operational or geometric differences and calculate a weighted average crash rate for each population weighted by traffic volume using Equation 4-10.

(4-10)

Where:

Ra = Weighted average crash rate for reference population

Ri = Observed crash rate at site i

TEVi = Total entering vehicles per day for intersection i

Section 7

For this sample problem, the populations are two-way, stop-controlled intersections (TWSC) and intersections controlled by traffic signals as summarized in the following table:

Chapter 180 Controlled	Two-Way Stop	Chapter 181	Crash Rate	Chapter 182 Crash Rate	Weighted Average
2		2.42		1.03	
3		1.12			
7		1.41			
10		0.94			
15		0.59			
17		0.67			
19		0.56			
Signalized		Crash Rate		Weighted Average Crash Rate	
1		0.58		0.42	
4		0.54			
5		0.28			
6		0.23			
8		0.18			
9		0.61			
11		0.79			
12		0.45			
13		0.24			
14		0.20			
16		0.97			
18		0.79			
20		0.12			

Section 8

Calculate a critical crash rate for each intersection using Equation 4-11.

(4-11)

Where:

$R_{c,i}$ = Critical crash rate for intersection i

R_a = Weighted average crash rate for reference population

P = P-value for corresponding confidence level

MEV_i = Million entering vehicles for intersection i

Section 9

For Intersection 7, the calculation of the critical crash rate is:

[crashes/MEV]

Section 10

Observed crash rates are compared with critical crash rates. Any intersection with an observed crash rate greater than the corresponding critical crash rate is flagged for further review.

Section 11

The critical crash rate for Intersection 7 is compared to the observed crash rate for Intersection 7 to determine if further review of Intersection 7 is warranted.

Critical Crash Rate for Intersection 7 = 1.40 [crashes/MEV]

Observed Crash Rate for Intersection 7 = 1.41 [crashes/MEV]

Since 1.41 > 1.40, Intersection 7 is identified for further review.

The following table summarizes the results for all 20 intersections being screened by the roadway agency.

Critical Rate Method Results

Intersection	Observed Crash Rate (crashes/MEV)	Critical Crash Rate (crashes/MEV)	Identified for Further Review
1	0.58	0.60	
2	2.42	1.51	X
3	1.12	1.43	
4	0.54	0.66	
5	0.28	0.57	
6	0.23	0.60	
7	1.41	1.40	X
8	0.18	0.58	
9	0.61	0.56	X
10	0.94	1.45	
11	0.79	0.58	X
12	0.45	0.55	
13	0.24	0.65	
14	0.20	0.58	
15	0.59	1.36	
16	0.97	0.67	X
17	0.67	1.44	
18	0.79	0.66	X
19	0.56	1.44	

20	0.12	0.56

Excess Predicted Average Crash Frequency Using Method of Moments

In the method of moments, a site's observed crash frequency is adjusted to partially account for regression to the mean. The adjusted observed average crash frequency is compared to the average crash frequency for the reference population to determine the potential for improvement (PI). The potential for improvement of all reference populations (e.g., signalized four-legged intersections, unsignalized three-legged intersections, urban, and rural, etc.) are combined into one ranking list as a basic multiple-facility network screening tool.

Data Needs

Crashes by location

Multiple reference populations

Strengths and Limitations

The strengths and limitations of the performance measure include the following:

Chapter 185 Strengths	Chapter 186 Limitations
Establishes a threshold of predicted performance for a site	Effects of RTM bias may still be present in the results
Considers variance in crash data	Does not account for traffic volume
Allows sites of all types to be ranked in one list	Some sites may be identified for further study because of unusually
low frequency of non-target crash types	
Method concepts are similar to Empirical Bayes methods	Ranking results are influenced by reference populations; sites near boundaries of reference populations may be over-emphasized

Procedure

The following outlines the procedure for ranking intersections using the Method of Moments. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

Organize historical crash data of the study period based upon factors such as facility type, location, or other defining characteristics.

The intersections from Table 4-4 have been organized into two reference populations, as shown in the first table for two-way stop controlled intersections and in the second table for signalized intersections.

TWSC Reference Population

Intersection ID	Number of Approaches	Urban/Rural	Total Crashes	Average Observed
Crash Frequency				
2	4	U	35	11.7

3	4	U	23	7.7
7	4	U	34	11.3
10	4	U	17	5.7
15	4	U	17	5.7
17	4	U	13	4.3
19	4	U	11	3.7
Sum			150	50.1

Signalized Reference Population

Chapter 188 Intersection ID	Chapter 189 Number of Approaches	Chapter 190 Urban/Rural	Chapter 191 Total Crashes	Chapter 192 Average Observed Crash Frequency
1	4	U	22	7.3
4	4	U	13	4.3
5	4	U	15	5.0
6	4	U	9	3.0
8	4	U	9	3.0
9	4	U	37	12.3
11	4	U	38	12.7
12	4	U	32	10.7
13	4	U	6	2.0
14	4	U	10	3.3
16	4	U	21	7.0
18	4	U	19	6.3
20	4	U	8	2.7
Sum			239	79.6

Sum the average annual observed crash frequency for each site in the reference population and divide by the number of sites.

(4-12)

Where:

$N_{observed\ rp}$ = Average crash frequency, per reference population

$N_{observed,I}$ = Observed crash frequency at site i

n(sites) = Number of sites per reference population

Section 13

Calculate the observed average crash frequency in the TWSC reference population:

[crashes per year]

Section 14

Chapter 194 **S**

TEP 3—Calculate Crash Frequency Variance per Reference Population

Excess Predicted Average Crash Frequency Using Method of Moments

Use Equation 4-13 to calculate variance. Alternatively, variance can be more easily calculated with common spreadsheet programs.

(4-13)

Where:

Var(N) = Variance

Nobserved,rp = Average crash frequency, per reference population

Nobserved,I = Observed crash frequency per year at site i

Nsites = Number of sites per reference population

Section 15

Calculate the crash frequency variance calculation for the TWSC reference population:

The variance for signal and TWSC reference populations is shown in the following table:

		Chapter 196	Crash Frequency		
Chapter 195	Reference Population	Chapter 197	Average	Chapter 198	Variance
Signal		6.1		13.75	
TWSC		7.1		10.5	

Section 16

Chapter 199 **S**

TEP 4—Calculate Adjusted Observed Crash Frequency per Site

Excess Predicted Average Crash Frequency Using Method of Moments

Using the variance and average crash frequency for a reference population, find the adjusted observed crash frequency for each site using Equation 4-14.

(4-14)

Where:

$N_{observed,i(adj)}$ = Adjusted observed number of crashes per year, per site

$Var(N)$ = Variance (equivalent to the square of the standard deviation, s2)

$N_{observed,I}$ = Observed average crash frequency per year at site i

$N_{observed,rp}$ = Average crash frequency, per reference population

Section 17

As shown, calculate the adjusted observed average crash frequency for Intersection 7:

[crashes per year]

Section 18

Subtract the average crash frequency per reference population from the adjusted observed average crash frequency per site.

(4-15)

Where:

PI_i = Potential for Improvement per site

$N_{observed,i(adj)}$ = Adjusted observed average crash frequency per year, per site

$N_{observed,rp}$ = Average crash frequency, per reference population

Section 19

As shown below, calculate the potential for improvement for Intersection 7:

$PI_7 = 8.5 – 7.1 = 1.4$ [crashes per year]

Section 20

Rank all sites from highest to lowest PI value. A negative PI value is not only possible but indicates a low potential for crash reduction.

Section 21

The PI rankings along with each site's adjusted observed crash frequency are as follows:

Intersections	Observed Average		
Crash Frequency	Adjusted Observed		
Crash Frequency	PI		
11	12.7	9.8	3.6
9	12.3	9.6	3.4
12	10.7	8.6	2.5
2	11.7	8.6	1.4
7	11.3	8.5	1.4
1	7.3	6.8	0.7
16	7.0	6.6	0.5
3	7.7	7.3	0.2
18	6.3	6.2	0.1
10	5.7	6.7	–0.5
15	5.7	6.7	–0.5
5	5.0	5.5	–0.6
17	4.3	6.3	–0.9
4	4.3	5.1	–1.0
19	3.7	6.0	–1.1
14	3.3	4.6	–1.5
6	3.0	4.4	–1.7
8	3.0	4.4	–1.7
20	2.7	4.2	–1.9
13	2.0	3.8	–2.3

Level of Service of Safety (LOSS)

Sites are ranked by comparing their observed average crash frequency to the predicted average crash frequency for the entire population under consideration (1,4,5). The degree of deviation from the predicted average crash frequency is divided into four LOSS classes. Each site is assigned a LOSS based on the difference between the observed average crash frequency and the predicted average crash frequency for the study group. Sites with poor LOSS are flagged for further study.

Data Needs

Crash data by location (recommended period of 3 to 5 Years)

Calibrated Safety Performance Function (SPF) and overdispersion parameter

Traffic volume

Strengths and Limitations

The strengths and limitations of the performance measure include the following:

Chapter 202 Strengths	Chapter 203 Limitations
Considers variance in crash data	Effects of RTM bias may still be present in the results
Accounts for volume	
Establishes a threshold for measuring crash frequency	

Procedure

The following sections outline the assumptions and procedure for ranking the intersections using the LOSS performance measure.

Section 22

Sample Problem Assumptions

The calculations for Intersection 7 are used throughout the sample problem to demonstrate how to apply each method.

The Sample problems provided in this section are intended to demonstrate calculation of the performance measures, not the predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using the predictive method outlined in Part C and are provided in Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the base SPF model. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are to simplify this example and are rarely valid for application of the predictive method to actual field conditions.

Section 23

Use the predictive method and SPFs outlined in Part C, or the roundabout-specific network screening SPFs described in Appendix 4B, to estimate the average crash frequency. The predicted average crash frequency is summarized in Table 4-10:

Table 4-10. Estimated Predicted Average Crash Frequency from an SPF

Chapter 205 I ntersection	Chapter 206 ear	Chapter 207 AADT Chapter 210 ajor Street	Chapter 207 AADT Chapter 211 inor Street	Chapter 208 P redicted Average Crash Frequency from an SPF	Chapter 209 A verage 3-Year Predicted Crash Frequency from an SPF
2	1	12,000	1,200	1.7	1.7
	2	12,200	1,200	1.7	
	3	12,900	1,300	1.8	
3	1	18,000	800	2.1	2.2
	2	18,900	800	2.2	
	3	19,100	800	2.2	
7	1	21,000	1,000	2.5	2.6
	2	21,400	1,000	2.5	
	3	22,500	1,100	2.7	
10	1	15,000	1,500	2.1	2.2
	2	15,800	1,600	2.2	
	3	15,900	1,600	2.2	
15	1	26,000	500	2.5	2.3
	2	26,500	300	2.2	
	3	27,800	200	2.1	
17	1	14,400	3,200	2.5	2.6
	2	15,100	3,400	2.6	
	3	15,300	3,400	2.6	
19	1	15,400	2,500	2.4	2.5
	2	15,700	2,500	2.5	
	3	16,500	2,600	2.6	

Calculate the standard deviation of the predicted crashes. Equation 4-16 is used to calculate the standard deviation. This estimate of standard deviation is valid since the SPF assumes a negative binomial distribution of crash counts.

(4-16)

Where:

σ = Standard deviation

K = Overdispersion parameter of the SPF

Npredicted = Predicted average crash frequency from the SPF

Section 24

As shown, the standard deviation calculations for Intersection 7 are

The standard deviation calculation is performed for each intersection. The standard deviation for the TWSC intersections is summarized in the following table:

Chapter 213 Intersection	Chapter 214 Average Observed Crash Frequency	Chapter 215 Predicted Average Crash Frequency from an SPF	Chapter 216 Standard Deviation
2	11.7	1.7	1.1
3	7.7	2.2	1.4
7	11.3	2.6	1.6
10	5.7	2.2	1.4
15	5.7	2.3	1.5
17	4.3	2.6	1.6
19	3.7	2.5	1.6

Section 25

Chapter 217 **S**

TEP 3—Calculate Limits for LOSS Categories **Level of Service of Safety (LOSS)**

Calculate the limits for the four LOSS categories for each intersection using the equations summarized in Table 4-11.

- Table 4-11. LOSS Categories

Chapter 218 LOSS	Chapter 219 Condition	Chapter 220 Description
I	$\sigma < N_{observed} < (N_{predicted} - 1.5 \times (\square))$	Indicates a low potential for crash reduction
II	$(N_{predicted} - 1.5 \times (\sigma)) \leq N_{observed} < N_{predicted}$	Indicates low to moderate potential for crash reduction
III	$N_{predicted} \leq N_{observed} < (N_{predicted} + 1.5 \times (\sigma))$	Indicates moderate to high potential for crash reduction
IV	$N_{observed} \geq (N_{predicted} + 1.5 \times (\sigma))$	Indicates a high potential for crash reduction

Section 26

Section 27

This sample calculation for Intersection 7 demonstrates the upper limit calculation for LOSS III.

$N_{predicted} + 1.5 \times (\sigma) = 2.6 + 1.5 \times (1.6) = 5.0$

A similar pattern is followed for the other LOSS limits.

The values for this calculation are provided in the following table:

LOSS Limits for Intersection 7

Chapter 221 Intersection	Chapter 222 LOSS I Limits	Chapter 223 LOSS II Limits	Chapter 224 LOSS III Upper Limit	Chapter 225 LOSS IV Limits

7	0 to 0.2	0.2 to 2.6	2.6 to 5.0	≥ 5.0

Section 28

Compare the total observed crash frequency at each intersection, NO, to the limits of the four LOSS categories. Assign a LOSS to each intersection based on the category in which the total observed crash frequency falls.

Section 29

Given that an average of 11.3 crashes were observed per year at Intersection 7 and the LOSS IV limits are 5.0 crashes per year, Intersection 7 is categorized as Level IV.

Section 30

List the intersections based on their LOSS for total crashes.

Section 31

The following table summarizes the TWSC reference population intersection ranking based on LOSS:

Intersection LOSS Ranking

Chapter 228	Intersection	Chapter 229	LOSS
2		IV	
3		IV	
7		IV	
10		IV	
15		IV	
17		III	
19		III	

Section 32

Excess Predicted Average Crash Frequency Using SPFs

Locations are ranked in descending order based on the excess crash frequency or the excess predicted crash frequency of a particular collision type or crash severity.

Data Needs

Crash data by location

Strengths and Limitations

The strengths and limitations of the performance measure include the following:

Chapter 230 Strengths	Chapter 231 Limitations
Accounts for traffic volume	Effects of RTM bias may still be present in the results
Estimates a threshold for comparison	

Procedure

The following sections outline the assumptions and procedure for ranking intersections using the Excess Predicted Crash Frequency using SPFs performance measure.

Section 33

Sample Problem Assumptions

The Sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in Part C and are provided in Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the SPF. These assumptions are for theoretical application and are rarely valid for application of Part C predictive method to actual field conditions.

Section 34

Tabulate the number of crashes by type and severity at each site for each reference population being screened.

Section 35

The reference population for TWSC intersections is shown as an example in the following table:

TWSC Reference Population

Chapter 233 Intersection	Chapter 234 Year	Chapter 235 AADT		Chapter 236 Observed Number of Crashes	Chapter 237 Average Observed Crash Frequency
		Major Street	Minor Street		
2	1	12,000	1,200	9	
	2	12,200	1,200	11	11.7
	3	12,900	1,300	15	
3	1	18,000	800	9	
	2	18,900	800	8	7.7
	3	19,100	800	6	
7	1	21,000	1,000	11	
	2	21,400	1,000	9	11.3
	3	22,500	1,100	14	

10	1	15,000	1,500	7	
	2	15,800	1,600	6	5.7
	3	15,900	1,600	4	
15	1	26,000	500	6	
	2	26,500	300	3	5.7
	3	27,800	200	8	
17	1	14,400	3,200	4	
	2	15,100	3,400	4	4.3
	3	15,300	3,400	5	
19	1	15,400	2,500	5	
	2	15,700	2,500	2	3.7
	3	16,500	2,600	4	

Section 36

Using the predictive method in Part C (and, if appropriate, the roundabout-specific network screening SPFs described in Appendix 4B), calculate the predicted average crash frequency, Npredicted,n, for each year, n, where n = 1,2,…,Y. Refer to Part C—Introduction and Applications Guidance for a detailed overview of the method to calculate the predicted average crash frequency. The example provided here is simplified to emphasize calculation of the performance measure, not the predictive method.

Section 37

The predicted average crash frequency from SPFs are summarized for the TWSC intersections for a three-year period in the following table:

SPF Predicted Average Crash Frequency

Chapter 239 I ntersection	Chapter 240 ear	Chapter 241 P redicted Average Crash Frequency from SPF (Total)	Chapter 242 P redicted Average Crash Frequency from an SPF (FI)	Chapter 243 P redicted Average Crash Frequency from an SPF (PDO)	Chapter 244 A verage 3-Year Predicted Crash Frequency from SPF
2	1	1.7	0.6	1.1	1.7
	2	1.7	0.6	1.1	
	3	1.8	0.7	1.1	
3	1	2.1	0.8	1.3	2.2
	2	2.2	0.8	1.4	
	3	2.2	0.9	1.4	
7	1	2.5	1.0	1.6	2.6
	2	2.5	1.0	1.6	
	3	2.7	1.1	1.7	
10	1	2.1	0.8	1.3	2.2
	2	2.2	0.9	1.4	

	3	2.2	0.9	1.4	
15	1	2.5	1.0	1.6	2.3
	2	2.2	0.9	1.4	
	3	2.1	0.8	1.3	
17	1	2.5	1.0	1.5	2.6
	2	2.6	1.0	1.6	
	3	2.6	1.0	1.6	
19	1	2.4	1.0	1.5	2.5
	2	2.5	1.0	1.5	
	3	2.6	1.0	1.6	

For each intersection the excess predicted average crash frequency is based upon the average of all years of data. The excess is calculated as the difference in the observed average crash frequency and the predicted average crash frequency from an SPF.

(4-17)

Where:

= Observed average crash frequency for site i

= Predicted average crash frequency from SPF for site.

- Shown below is the predicted excess crash frequency calculation for Intersection 7:

Excess(TWSC) = 11.3 – 2.6 = 8.7 [crashes per year]

Section 38

The following table shows the excess expected average crash frequency for the TWSC reference population:

Excess Predicted Average Crash Frequency for TWSC Population

Chapter 246 Intersection	Chapter 247 Observed Average Crash Frequency	Chapter 248 Predicted Average Crash Frequency from an SPF	Chapter 249 Excess Predicted Average Crash Frequency
2	11.7	1.7	10.0
3	7.7	2.2	5.5
7	11.3	2.6	8.7
10	5.7	2.2	3.5
15	5.7	2.3	3.4

17	4.3	2.6	1.7
19	3.7	2.5	1.2

Section 39

Rank all sites in each reference population according to the excess predicted average crash frequency.

Section 40

The following table ranks the TWSC intersections according to the excess predicted average crash frequency:

Ranking of TWSC Population Based on Excess Predicted Average Crash Frequency from an SPF

Chapter 250	Intersection	Chapter 251	Excess Predicted Average Crash Frequency
2		10.0	
7		8.7	
3		5.5	
10		3.5	
15		3.4	
17		1.7	
19		1.2	

Section 41

Probability of Specific Crash Types Exceeding Threshold Proportion

Sites are prioritized based on the probability that the true proportion, pi, of a particular crash type or severity

(e.g., long-term predicted proportion) is greater than the threshold proportion, p*i (6). A threshold proportion (p*i) is identified for each crash type.

Data Needs

Crash data by type and location

Strengths and Limitations

The strengths and limitations of the Probability of Specific Crash Types Exceeding Threshold Proportion performance measure include the following:

Chapter 252	Strengths	Chapter 253	Limitations
Can also be used as a diagnostic tool (Chapter 5)		Does not account for traffic volume	
Considers variance in data		Some sites may be identified for further study because of unusually	
low frequency of non-target crash types			
Not effected by RTM Bias			

Procedure

Organize sites into reference populations and screen to identify those that have a high proportion of a specified collision type or crash severity.

The sample intersections are to be screened for a high proportion of angle crashes. Prior to beginning the method, the 20 intersections are organized into two subcategories (i.e., reference populations): (1) TWSC intersections and (2) signalized intersections.

Chapter 254 **S**

TEP 1—Calculate Observed Proportions **Probability of Specific Crash Types Exceeding Threshold Proportion**

A. Determine which collision type or crash severity to target and calculate observed proportion of target collision type or crash severity for each site.
B. Identify the frequency of the collision type or crash severity of interest and the total observed crashes of all types and severity during the study period at each site.
C. Calculate the observed proportion of the collision type or crash severity of interest for each site that has experienced two or more crashes of the target collision type or crash severity using Equation 4-18.

(4-18)

Where:

Pi = Observed proportion at site i

$N_{observed,I}$ = Number of observed target crashes at site i

$N_{observed,i(total)}$ = Total number of crashes at site i

Section 42

Shown below is the calculation for angle crashes for Intersection 7. The values used in the calculation are found in Table 4-5.

Section 43

Chapter 255 **S**

TEP 2—Estimate a Threshold Proportion **Probability of Specific Crash Types Exceeding Threshold Proportion**

Select the threshold proportion of crashes, p*i, for a specific collision type. A useful default starting point is the proportion of target crashes in the reference population under consideration. For example, if considering rear-end crashes, it would be the observed average rear-end crash frequency experienced at all sites in the reference population divided by the total observed average crash frequency at all sites in the reference population. The proportion of a specific crash type in the entire population is calculated using Equation 4-19.

(4-19)

Where:

p^*I = Threshold proportion

= Sum of observed target crash frequency within the population

= Sum of total observed crash frequency within the population

Section 44

Below is the calculation for threshold proportion of angle collisions for TWSC intersections.

Section 45

The following table summarizes the threshold proportions for the reference populations:

Estimated Threshold Proportion of Angle Collisions

Chapter 256 Reference Population	Chapter 257 Angle Crashes	Chapter 258 Total Crashes	Chapter 259 Observed Threshold Proportion (p*i)
TWSC	33	150	0.22
Traffic Signals	82	239	0.34

Section 46

Chapter 260 **S**

TEP 3—Calculate Sample Variance **Probability of Specific Crash Types Exceeding Threshold Proportion**

Calculate the sample variance (s2) for each subcategory. The sample variance is different than population variance. Population variance is commonly used in statistics and many software tools and spreadsheets use the population variance formula as the default variance formula.

For this method, be sure to calculate the sample variance using Equation 4-20:

(4-20)

- for Nobserved,i(total) □□2

Where:

nsites = Total number of sites being analyzed

Nobserved,I = Observed target crashes for a site i

Nobserved,i(total) = Total number of crashes for a site i

Section 47

The following table summarizes the calculations for the two-way stop-controlled subcategory. TWSC sites 15 and 19 were removed from the variance calculation because fewer than two angle crashes were reported over the study period.

Sample Variance Calculation

Chapter 261 WSC	Chapter 262 ngle Crashes (Nobserved,i)	Chapter 263 Nobserved,i)2	Chapter 264 (otal Crashes (Nobserved,i(total))	Chapter 265 Nobserved,i(total))2	Chapter 266 (	Chapter 267 WSC Variance
2	21	441	35	1225	5	0.037

7	5	25	34	1156
3	2	4	23	529
10	2	4	17	289
17	2	4	13	169

Section 48

Calculate the sample mean proportion of target crashes by type or severity for all sites under consideration using Equation 4-21.

(4-21)

Where:

Nsites = Total number of sites being analyzed

= Mean proportion of target crash types

Pi = Observed proportion

Calculate Alpha (□) and Beta (ß) for each subcategory using Equations 4-22 and 4-23.

(4-22)

(4-23)

Where:

Var(N) = Variance (equivalent to the square of the standard deviation, s2)

= Mean proportion of target crash types

Section 49

The calculation for the two-way stop-controlled subcategory is:

The following table shows the numerical values used in the equations and summarizes the alpha and beta calculations for the TWSC intersections:

Alpha and Beta Calculations

Chapter 269	Subcategories	Chapter 270	s2	Chapter 271	Chapter 272	◎	Chapter 273	ß
TWSC		0.037		0.22	0.80		2.84	

Section 50

Using a "betadist" spreadsheet function, calculate the probability for each intersection as shown in Equation 4-24.

(4-24)

Where:

p*I = Threshold proportion

pi = Observed proportion

Nobserved,I = Observed target crashes for a site i

Nobserved,i(total) = Total number of crashes for a site i

Section 51

The probability calculation for Intersection 7 is:

The following table summarizes the probability calculation for Intersection 7:

Probability Calculations

Chapter 275 WSC	Chapter 276 ngle Crashes (Nobserved,i)	Chapter 277 otal Crashes (Nobserved,i(total))	Chapter 278 i	Chapter 279 *i	Chapter 280	Chapter 281	Chapter 282 robability
7	5	34	0.15	0.22	0.80	2.84	0.13

For Intersection 7, the resulting probability is interpreted as "There is a 13 percent chance that the long-term expected proportion of angle crashes at Intersection 7 is actually greater than the long-term expected proportion for TWSC intersections." Therefore, in this case, with such a small probability, there is limited need of additional study of Intersection 7 with regards to angle crashes.

Section 52

Rank the intersections based on the probability of angle crashes occurring at the intersection.

The TWSC intersection population is ranked based on the Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure as shown in the following table:

Ranking Based on Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure

Chapter 284	Intersections	Chapter 285	Probability
2		1.00	
11		0.98	
9		0.83	
12		0.75	
16		0.48	
6		0.48	
13		0.48	
20		0.41	
4		0.35	
17		0.25	
5		0.21	
1		0.19	
18		0.19	
7		0.13	
10		0.13	
3		0.04	

Excess Proportion of Specific Crash Types

Sites are evaluated to quantify the extent to which a specific crash type is overrepresented compared to other crash types at a location. The sites are ranked based on excess proportion, which is the difference between the true proportion, pi, and the threshold proportion, p*i. The excess is calculated for a site if the probability that a site's long-term observed proportion is higher than the threshold proportion, p*i, exceeds a certain limiting probability (e.g., 90 percent).

Data Needs

Crash data by type and location

Strengths and Limitations

The strengths and limitations of the Excess Proportions of Specific Crash Types Proportion performance measure include the following:

Chapter 286	Strengths	Chapter 287	Limitations
Can also be used as a diagnostic tool		Does not account for traffic volume.	
Considers variance in data		Some sites may be identified for further study because of unusually low frequency of non-target crash types	
Not effected by RTM Bias			

Procedure

Calculation of the excess proportion follows the same procedure outlined in Steps 1 through 5 of the Probability of Specific Crash Types Exceeding Threshold Proportions method. Therefore, the procedure outlined in this section builds on the previous method and applies results of sample calculations shown above in the example table of Step 6.

For the sample situation, the limiting probability is selected to be 60 percent. The selection of a limiting probability can vary depending on the probabilities of each specific crash types exceeding a threshold proportion. For example, if many sites have high probability, the limiting probability can be correspondingly higher in order to limit the number of sites to a reasonable study size. In this example, a 60 percent limiting probability results in four sites that will be evaluated based on the Excess Proportions performance measure.

Chapter 288 **S**

TEP 6—Calculate the Excess Proportion **Excess Proportion of Specific Crash Types**

Calculate the difference between the true observed proportion and the threshold proportion for each site using Equation 4-25:

(4-25)

Where:

p*I = Threshold proportion

pi = Observed proportion

Chapter 289 **S**

TEP 7—Rank Locations **Excess Proportion of Specific Crash Types**

Rank locations in descending order by the value of Pdiff. The greater the difference between the observed and threshold proportion, the greater the likelihood that the site will benefit from a countermeasure targeted at the collision type under consideration.

Section 54

The four intersections that met the limiting probability of 60 percent are ranked in the following table:

Ranking Based on Excess Proportion

Chapter 290 Inters ections	Chapter 291 Prob ability	Chapter 292 Ob served Proportion	Chapter 293 Thr eshold Proportion	Chapter 294 E xcess Proportion
2	1.00	0.60	0.22	0.38
11	0.99	0.61	0.34	0.27
9	0.81	0.46	0.34	0.12
12	0.71	0.44	0.34	0.10

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

The Empirical Bayes (EB) method is applied in the estimation of expected average crash frequency. The EB method, as implemented in this chapter, is implemented in a slightly more sophisticated manner than in Part C, Appendix A. The version of the EB method implemented here uses yearly correction factors for consistency with network screening applications in the SafetyAnalyst software tools.

Data Needs

Crash data by severity and location

Traffic volume

Basic site characteristics (i.e., roadway cross-section, intersection control, etc.)

Calibrated Safety Performance Functions (SPFs) and overdispersion parameters

Strengths and Limitations

The strengths and limitations of the Expected Average Crash Frequency with EB Adjustment performance measure include the following:

Strengths	Limitations
Accounts for RTM bias	Requires SPFs calibrated to local conditions

Procedure

The following sample problem outlines the assumptions and procedure for ranking intersections based on the expected average crash frequency with Empirical Bayes adjustments. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

Section 55

Sample Problem Assumptions

The sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in Part C and are provided in Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of the Part C predictive method to actual field conditions.

Section 56

Chapter 295 **S**

TEP 1—Calculate the Predicted Average Crash Frequency from an SPF

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

Using the predictive method in Part C (and, if appropriate, the roundabout-specific network screening SPFs described in Appendix 4B) calculate the predicted average crash frequency, Npredicted,n, for each year, n, where n = 1,2,…,Y. Refer to Part C—Introduction and Applications Guidance for a detailed overview of the method to

calculate the predicted average crash frequency. The example provided here is simplified to emphasize calculation of the performance measure, not predictive method.

In the following steps this prediction will be adjusted using an annual correction factor and an Empirical Bayes weight. These adjustments will account for annual fluctuations in crash occurrence due to variability in roadway conditions and other similar factors; they will also incorporate the historical crash data specific to the site.

Calculate the annual correction factor (Cn) at each intersection for each year and each severity (i.e., total and FI).

The annual correction factor is predicted average crash frequency from an SPF for year n divided by the predicted average crash frequency from an SPF for year 1. This factor is intended to capture the effect that annual variations in traffic, weather, and vehicle mix have on crash occurrences. (3)

(4-26)

Where:

Cn(total) = Annual correction factor for total crashes

Cn(FI) = Annual correction factor for fatal or injury crashes, or both

Npredicted, n(total) = Predicted number of total crashes for year n

Npredicted,n(FI) = Predicted number of fatal or injury crashes, or both, for year n

Section 57

Shown below is the calculation for Intersection 7 based on the annual correction factor for year 3. The predicted crashes shown in the equation are the result of Step 1 and are summarized in the table that follows.

This calculation is repeated for each year and each intersection. The following table summarizes the annual correction factor calculations for the TWSC intersections:

Annual Correction Factors for all TWSC Intersections

Chapter 297 I ntersection	Chapter 298 ear	Chapter 299 P redicted Average Crash Frequency from SPF (total)	Chapter 300 P redicted Average Crash Frequency from SPF (FI)	Chapter 301 C orrection Factor (total)	Chapter 302 C orrection Factor (FI)
2	1	1.7	0.6	1.0	1.0
	2	1.7	0.6	1.0	1.0
	3	1.8	0.7	1.1	1.2
3	1	2.1	0.8	1.0	1.0

	2	2.2	0.8	1.0	1.0
	3	2.2	0.9	1.0	1.1
7	1	2.5	1.0	1.0	1.0
	2	2.5	1.0	1.0	1.0
	3	2.7	1.1	1.1	1.1
10	1	2.1	0.8	1.0	1.0
	2	2.2	0.9	1.0	1.1
	3	2.2	0.9	1.0	1.1
15	1	2.5	1.0	1.0	1.0
	2	2.2	0.9	0.9	0.9
	3	2.1	0.8	0.8	0.8
17	1	2.5	1.0	1.0	1.0
	2	2.6	1.0	1.0	1.0
	3	2.6	1.0	1.0	1.0
19	1	2.4	1.0	1.0	1.0
	2	2.5	1.0	1.0	1.0
	3	2.6	1.0	1.1	1.0

Calculate the weighted adjustment, w, for each intersection and each severity (i.e., total and FI). The weighted adjustment accounts for the reliability of the safety performance function that is applied. Crash estimates produced using Safety Performance Functions with overdispersion parameters that are low (which indicates higher reliability) have a larger weighted adjustment. Larger weighting factors place a heavier reliance on the SPF estimate.

(4-27)

Where:

w = Empirical Bayes weight

k = Overdispersion parameter of the SPF

Npredicted, n(total) = Predicted average total crash frequency from an SPF in year n

Npredicted, n(FI) = Predicted average fatal and injury crash frequency from an SPF in year n

Section 58

Shown below is the weighted adjustment calculation for total and fatal/injury crashes for Intersection 7.

The sum of the predicted crashes (7.7 and 3.1) is the result of summing the annual predicted crashes summarized in Step 2 for Intersection 7.

The calculated weights for the TWSC intersections are summarized in the following table:

Weighted Adjustments for TWSC Intersections

Chapter 304	Intersection	Chapter 305	wtotal	Chapter 306	wFI
2		0.3		0.4	
3		0.2		0.4	
7		0.2		0.3	
10		0.2		0.3	
15		0.2		0.3	
17		0.2		0.3	
19		0.2		0.3	

Section 59

Chapter 307 **S**

TEP 4—Calculate First Year EB-adjusted Expected Average Crash Frequency

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

Calculate the base EB-adjusted expected average crash frequency for year 1, Nexpected,1 using Equations 4-28 and 4-29.

This stage of the method integrates the observed crash frequency with the predicted average crash frequency from an SPF. The larger the weighting factor, the greater the reliance on the SPF to estimate the long-term predicted average crash frequency per year at the site. The observed crash frequency on the roadway segments is represented in the equations below as $N_{observed,n}$.

(4-28)

and

(4-29)

Where:

Nexpected,1 = EB-adjusted estimated average crash frequency for year 1

w = Weight

Npredicted,i(total) = Estimated average crash frequency for year 1 for the intersection

Nobserved,n = Observed crash frequency at the intersection

Cn = Annual correction factor for the intersection

n = year

Section 60

Shown below is the total and fatal/injury calculation for Intersection 7.

These calculations are based on information presented in Steps 2 and 3.

Section 61

Calculate the EB-adjusted expected number of fatal and injury crashes and total crashes for the final year (in this example, the final year is year 3).

$$N_{expected,n(total)} = N_{expected,1(total)} \times C_{n(total)} \quad (4\text{-}30)$$

$$N_{expected,n(FI)} = N_{expected,1(FI)} \times C_{n(FI)} \quad (4\text{-}31)$$

Where:

$N_{expected,n}$ = EB-adjusted expected average crash frequency for final year

$N_{expected,1}$ = EB-adjusted expected average crash frequency for year 1

C_n = Annual correction factor for year, n

Section 62

Shown below are the calculations for Intersection 7.

$N_{expected,3(total)} = 9.3 \times (1.1) = 10.2$

$N_{expected,3(FI)} = 4.4 \times (1.1) = 4.8$

$N_{expected,3(PDO)} = N_{expected,3(total)} - N_{expected,3(FI)}$

The following table summarizes the calculations for Intersection 7:

Year 3—EB-Adjusted Expected Average Crash Frequency[a]

Chapter 309 ersection	Int	Chapter 310 atal and/or Injury Crashes	Chapter 311 otal Crashes	Chapter 312 DO Crashes	Chapter 31	Chapter	Chapter 31	Chapter 31
		NE,1(FI)	C3(FI)	NE,3(FI)	NE,1(total)	C3(total)	NE,3(total)	NE,3(PDO)
7		4.4	1.1	4.8	9.3	1.1	10.2	5.4

a E = "expected" in the variables presented in this table

Section 63

When using the peak searching method (or an equivalent method for intersections), calculate the variance of the

EB-adjusted expected number of crashes for year n. Equation 4-32 is applicable to roadway segments and ramps, and Equation 4-33 is applicable to intersections.

(4-32)

(4-33)

Section 64

Shown below are the variation calculations for Year 3 at Intersection 7.

The following table summarizes the calculations for Year 3 at Intersection 7:

Year 3—Variance of EB-Adjusted Expected Average Crash Frequency

Chapter 318	Intersection	Chapter 319	Variance
2		2.1	
3		1.4	
7		2.9	
10		1.1	
15		1.0	
17		1.0	
19		1.0	

Section 65

Rank the intersections based on the EB-adjusted expected average crash frequency for the final year in the analysis, as calculated in Step 5.

Section 66

This table summarizes the ranking based on EB-Adjusted Crash Frequency for the TWSC Intersections.

EB-Adjusted Expected Average Crash Frequency Ranking

Chapter 321 Intersection	Chapter 322 EB-Adjusted Average Crash Frequency
7	10.2
2	9.6
3	6.1
10	4.5
15	4.3
17	3.9
19	3.7

Section 67

Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

Equivalent Property Damage Only (EPDO) Method assigns weighting factors to crashes by severity to develop a single combined frequency and severity score per location. The weighting factors are calculated relative to Property Damage Only (PDO) crashes. To screen the network, sites are ranked from the highest to the lowest score. Those sites with the highest scores are evaluated in more detail to identify issues and potential countermeasures.

The frequency of PDO, Injury, and Fatal crashes is based on the number of crashes, not the number of injuries per crash.

Data Needs

Crashes by severity and location

Severity weighting factors

Traffic volume on major and minor street approaches

Basic site characteristics (i.e., roadway cross-section, intersection control, etc.)

Calibrated safety performance functions (SPFs) and overdispersion parameters

Strengths and Limitations

The strengths and limitations of the performance measure include the following:

Chapter 323 Strengths	Chapter 324 Limitations
Accounts for RTM bias	May overemphasize locations with a small number of severe crashes depending on weighting factors used
Considers crash severity	

Assumptions

The societal crash costs listed in Table 4-12 are used to calculate the EPDO weights.

- Table 4-12. Societal Crash Cost Assumptions

Chapter 325	Severity	Chapter 326	Cost
Fatal (K)		$4,008,900	
Injury Crashes (A/B/C)		$82,600	
PDO (O)		$7,400	

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA-HRT-05-051, October 2005

Section 68

Sample Problem Assumptions

The Sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in Part C and are provided in Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the base SPF model. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of predictive method to actual field conditions.

Section 69

Chapter 327 **S**

TEP 1—Calculate Weighting Factors for Crash Severity

Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

Calculate the EPDO weights for fatal, injury, and PDO crashes. The fatal and injury weights are calculated using Equation 4-34. The cost of a fatal or injury crash is divided by the cost of a PDO crash, respectively. Weighting factors developed from local crash cost data typically result in the most accurate results. If local information is not available, nationwide crash cost data is available from the Federal Highway Administration (FHWA). Appendix 4A provides information on the national data available and a method for updating crash costs to current dollar values.

The weighting factors are calculated as follows:

(4-34)

Where:

fy(weight) = EPDO weighting factor based on crash severity, y;

CCy = Crash cost for crash severity, y; and,

CCPDO = Crash cost for PDO crash severity.

Section 70

Incapacitating (A), evident (B), and possible (C) injury crash costs developed by FHWA were combined to develop an average injury (A/B/C) cost. Below is a sample calculation for the injury (A/B/C) EPDO weight (WI):

Therefore, the EPDO weighting factors for all crash severities are shown in the following table:

Example EPDO Weights

Chapter 328	Severity	Chapter 329	Cost	Chapter 330	Weight
Fatal (K)		$4,008,900		542	
Injury (A/B/C)		$82,600		11	
PDO (O)		$7,400		1	

Section 71

Chapter 331 **S**

TEP 2—Calculate Predicted Average Crash Frequency from an SPF

Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

Using the predictive method in Part C (and, if appropriate, the roundabout-specific network screening SPFs described in Appendix 4B), calculate the predicted average crash frequency, Npredicted,n, for each year, n, where n = 1, 2,…, N. Refer to Part C—Introduction and Applications Guidance for a detailed overview of the method to calculate the predicted average crash frequency. The example provided here is simplified to emphasize calculation of the performance measure, not the predictive method. The predicted average crash frequency from SPFs is summarized for the TWSC intersections for a three-year period in Table 4-13.

Calculations will have to be made for both total and Fatal/Injury crashes, or for Fatal/Injury and Property Damage Only crashes. This example calculates total and Fatal/Injury crashes, from which Property Damage Only crashes are derived.

Table 4-13. Estimated Predicted Average Crash Frequency from an SPF

Chapter 332 ntersection	I	Chapter 333 ear	Chapter 334 AADT Chapter 337 ajor Street	Chapter 338 inor Street	Chapter 335 P redicted Average Crash Frequency from an SPF	Chapter 336 A verage 3-Year Predicted Crash Frequency from an SPF
2		1	12,000	1,200	1.7	1.7
		2	12,200	1,200	1.7	
		3	12,900	1,300	1.8	
3		1	18,000	800	2.1	2.2
		2	18,900	800	2.2	
		3	19,100	800	2.2	
7		1	21,000	1,000	2.5	2.6
		2	21,400	1,000	2.5	
		3	22,500	1,100	2.7	
10		1	15,000	1,500	2.1	2.2
		2	15,800	1,600	2.2	
		3	15,900	1,600	2.2	
15		1	26,000	500	2.5	2.3
		2	26,500	300	2.2	
		3	27,800	200	2.1	
17		1	14,400	3,200	2.5	2.6
		2	15,100	3,400	2.6	
		3	15,300	3,400	2.6	
19		1	15,400	2,500	2.4	2.5
		2	15,700	2,500	2.5	
		3	16,500	2,600	2.6	

Chapter 339 S
TEP 3—Calculate Annual Correction Factors
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

Calculate the annual correction factors (Cn) at each intersection for each year and each severity using Equation 4-35.

The annual correction factor is predicted average crash frequency from an SPF for year y divided by the predicted average crash frequency from an SPF for year 1. This factor is intended to capture the effect that annual variations in traffic, weather, and vehicle mix have on crash occurrences (3).

(4-35)

Where:

Cn(total) = Annual correction factor for total crashes

Cn(FI) = Annual correction factor for fatal and/or injury crashes

Npredicted,n(total) = Predicted number of total crashes for year, n

Npredicted,1(total) = Predicted number of total crashes for year 1

Npredicted,n(FI) = Predicted number of fatal and/or injury crashes for year, n

Npredicted,1(FI) = Predicted number of fatal and/or injury crashes for year 1

Section 72

Shown below is the calculation for Intersection 7 based on the yearly correction factor for year 3. The predicted crashes shown in the equation are the result of Step 2.

The annual correction factors for all TWSC intersections are summarized in the following table:

Annual Correction Factors for all TWSC Intersections

Intersection	Year	Predicted Average Crash Frequency from an SPF (total)	Predicted Average Crash Frequency from an SPF (FI)	Correction Factor (total)	Correction Factor (FI)
2	1	1.7	0.6	1.0	1.0
	2	1.7	0.6	1.0	1.0
	3	1.8	0.7	1.1	1.2
3	1	2.1	0.8	1.0	1.0
	2	2.2	0.8	1.0	1.0
	3	2.2	0.9	1.0	1.1
7	1	2.5	1.0	1.0	1.0
	2	2.5	1.0	1.0	1.0
	3	2.7	1.1	1.1	1.1
10	1	2.1	0.8	1.0	1.0
	2	2.2	0.9	1.0	1.1
	3	2.2	0.9	1.0	1.1
15	1	2.5	1.0	1.0	1.0
	2	2.2	0.9	0.9	0.9
	3	2.1	0.8	0.8	0.8
17	1	2.5	1.0	1.0	1.0
	2	2.6	1.0	1.0	1.0
	3	2.6	1.0	1.0	1.0
19	1	2.4	1.0	1.0	1.0
	2	2.5	1.0	1.0	1.0
	3	2.6	1.0	1.1	1.0

Calculate the weighted adjustment, w, for each intersection and each severity. The weighted adjustment accounts for the reliability of the safety performance function that is applied. Crash estimates produced using safety performance functions with overdispersion parameters that are low (which indicates higher reliability) have a larger weighted adjustment. Larger weighting factors place a heavier reliance on the SPF to predict the long-term predicted average crash frequency per year at a site. The weighted adjustments are calculated using Equation 4-36.

(4-36)

Where:

w = Empirical Bayes weight

n = years

k = Overdispersion parameter of the SPF

Npredicted,n = Predicted average crash frequency from an SPF in year n

Section 73

Shown below is the weighted adjustment calculation for fatal/injury and total crashes for Intersection 7.

The overdispersion parameters shown below are found in Part C along with the SPFs. The sum of the predicted crashes (7.7 and 3.1) is the result of summing the annual predicted crashes for Intersection 7 summarized in Step 3.

The total and FI weights are summarized for the TWSC intersections in Step 5.

Section 74

Chapter 341 **S**

TEP 5—Calculate First Year EB-adjusted Expected Average Crash Frequency

Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

Calculate the base EB-adjusted expected average crash frequency for year 1, NE,1.

This stage of the method integrates the observed crash frequency with the predicted average crash frequency from an SPF. The larger the weighting factor, the greater the reliance on the SPF to estimate the long-term expected average crash frequency per year at the site. The observed crash frequency, Nobserved,y, on the roadway segments is represented in Equations 4-37 and 4-38 below.

(4-37)

and

(4-38)

Where:

Nexpected 1 = EB-adjusted expected average crash frequency for year 1

w = Weight

Npredicted,1 = Predicted average crash frequency for year 1

Nobserved,n = Observed average crash frequency at the intersection

Cn = Annual correction factor for the intersection

n = years

Section 75

Shown below is the total crash calculation for Intersection 7.

The following table summarizes the calculations for total crashes at Intersection 7.

Year 1—EB-Adjusted Number of Total Crashes

Chapter 342 ntersection	Chapter 343 N predicted,1(total)	Chapter 344 (total)	Chapter 345 N observed,n(total) (All Years)	Chapter 346 um of Total Correction Factors (C1 + C2 + C3)	Chapter 347 N expected,1(total)
7	2.5	0.2	34	3.1	9.3

The EB-adjusted expected average crash frequency calculations for all TWSC intersections are summarized in Step 6.

Section 76

Chapter 348 **S**

TEP 6—Calculate Final Year EB-adjusted Average Crash Frequency

Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

Calculate the EB-adjusted expected number of fatal and injury crashes and total crashes for the final year. Total and fatal and injury EB-adjusted expected average crash frequency for the final year is calculated using Equations 4-39 and 4-40, respectively.

$$N_{expected,n(total)} = N_{expected,1(total)} \times C_{n(total)} \quad (4\text{-}39)$$

$$N_{expected,n(FI)} = N_{expected,1(FI)} \times C_{n(FI)} \quad (4\text{-}40)$$

Where:

Nexpected,,n = EB-adjusted expected average crash frequency for final year, n (the final year of analysis in this sample problem is n = 3).

Nexpected,1 = EB-adjusted expected average crash frequency for first year, n = 1

Cn = Annual correction factor for year, n

Section 77

Shown below are the calculations for Intersection 7. The annual correction factors shown below are summarized in Step 3 and the EB-adjusted crashes for Year 1 are values from Step 4.

Nexpected,3(total) = 9.3 × (1.1) = 10.2

Nexpected,3(FI) = 4.4 × (1.1) = 4.8

Nexpected,3(PDO) = 10.2 – 4.8 = 5.4

The calculation of Nexpected,3(PDO) is based on the difference between the Total and FI expected average crash frequency. The following table summarizes the results of Steps 4 through 6, including the EB-adjusted expected average crash frequency for all TWSC intersections:

EB-Adjusted Expected Average Crash Frequency for TWSC Intersections

Chapter 349 ntersection	Chapter 350 ear	Chapter 351 bserved Number of Crashes (total)	Chapter 352 redicted Average Crash Frequency from an SPF (total)	Chapter 353 eight (total)	Chapter 354 eight (FI)	Chapter 355 B-Adjusted Expected Average Crash Frequency (total)	Chapter 356 B-Adjusted Expected Average Crash Frequency (FI)	Chapter 357 B-Adjusted Expected Average Crash Frequency (PDO)
2	1	9.0	1.7	0.3	0.4	8.7	4.9	3.8
	2	11.0	1.7			8.7	4.9	3.8
	3	15.0	1.8			9.6	5.8	3.8
3	1	9.0	2.1	0.2	0.4	6.1	3.0	3.1
	2	8.0	2.2			6.1	3.0	3.1
	3	6.0	2.2			6.1	3.3	2.8
7	1	11.0	2.5	0.2	0.3	9.3	4.3	5.0
	2	9.0	2.5			9.3	4.3	5.0
	3	14.0	2.7			10.2	4.8	5.4
10	1	7.0	2.1	0.2	0.3	4.5	1.7	2.8
	2	6.0	2.2			4.7	1.9	2.8
	3	4.0	2.2			4.5	1.9	2.6
15	1	6.0	2.5	0.2	0.3	5.4	1.6	3.8
	2	3.0	2.2			4.8	1.4	3.4
	3	8.0	2.1			4.3	1.3	3.0
17	1	4.0	2.5	0.2	0.3	3.9	1.7	2.2
	2	4.0	2.6			4.1	1.7	2.4
	3	5.0	2.6			3.9	1.7	2.2
19	1	5.0	2.4	0.2	0.3	3.4	1.7	1.7
	2	2.0	2.5			3.5	1.7	1.8
	3	4.0	2.6			3.7	1.7	2.0

Section 78

Chapter 358 **S**

TEP 7—Calculate the Proportion of Fatal and Injury Crashes

Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

Equations 4-41 and 4-42 are used to identify the proportion of fatal crashes with respect to all non-PDO crashes in the reference population and injury crashes with respect to all non-PDO crashes in the reference population.

(4-41)

(4-42)

Where:

Nobserved,(F) = Observed number of fatal crashes from the reference population;

Nobserved,(I) = Observed number of injury crashes from the reference population;

Nobserved,(FI) = Observed number of fatal-and-injury crashes from the reference population;

PF = Proportion of observed number of fatal crashes out of FI crashes from the reference population;

PI = Proportion of observed number of injury crashes out of FI crashes from the reference population.

Section 79

Shown below are the calculations for the TWSC intersection reference population.

Section 80

Compared to PDO crashes the relative EPDO weight of fatal and injury crashes is calculated using Equation 4-43.

$$wEPDO,FI = PF \times fK(weight) + PI \times finj(weight) \quad (4\text{-}43)$$

Where:

finj(weight) = EPDO injury weighting factor;

fK(weight) = EPDO fatal weighting factor;

PF = Proportion of observed number of fatal crashes out of FI crashes from the reference population.

Section 81

Shown below is the calculation for Intersection 7. The EPDO weights, fK(weight) and WI are summarized in Step 1.

wEPDO,FI = (0.075 × 542) + (0.925 × 11) = 50.8

Section 82

Equation 4-43 can be used to calculate the EPDO expected average crash frequency for the final year for which data exist for the site.

$N_{expected,3(EPDO)} = N_{expected,n(PDO)} + w_{EPDO,FI} \times N_{expected,n(FI)}$

Section 83

Shown below is the calculation for Intersection 7.

$N_{expected,3(EPDO)} = 5.4 + 50.8 \times 4.8 = 249.2$

Section 84

Order the database from highest to lowest by EB-adjusted EPDO score. The highest EPDO score represents the greatest opportunity to reduce the number of crashes.

Section 85

The following table summarizes the EB-Adjusted EPDO Ranking for the TWSC Intersections.

EB-Adjusted EPDO Ranking

Chapter 362 Intersection	Chapter 363 EB-Adjusted EPDO
2	298.4
7	249.2
3	170.4
10	99.1
17	88.6
19	88.4
15	69.0

Excess Expected Average Crash Frequency with EB Adjustments

The Empirical Bayes Method is applied to estimate expected crash frequency. Part C Introduction and Applications Guidance, explains how to apply the EB Method. Intersections are ranked based on the difference between the predicted estimates and EB-adjusted estimates for each intersection, the excess expected average crash frequency per year.

Data Needs

Crash data by severity and location

Traffic volume

Basic site characteristics (i.e., roadway cross-section, intersection control)

Calibrated Safety Performance Functions (SPFs) and overdispersion parameters

Strengths and Limitations

The strengths and limitations of the Excess Expected Average Crash Frequency with EB Adjustments performance measure include the following:

Chapter 364	Strengths	Chapter 365	Limitations
Accounts for RTM bias		None	
Identifies a threshold to indicate sites experiencing more crashes than expected for sites with similar characteristics			

Procedure

The following sample problem outlines the assumptions and procedure for ranking seven TWSC intersections based on the expected crash frequency with Empirical Bayes adjustments. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

- Table 4-14. Societal Crash Cost Assumptions

Chapter 366	Crash Severity	Chapter 367	Crash Cost
Combined Cost for Crashes with a Fatality or Injury, or Both (K/A/B/C)		$158,200	
PDO (O)		$7,400	

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA-HRT-05-051, October 2005

As shown in Table 4-14, the crash cost that can be used to weigh the expected number of FI crashes is $158,200. The crash cost that can be used to weigh the expected number of PDO crashes is $7,400. More information on crash costs, including updating crash cost values to current year of study values, is provided in Appendix 4A.

Section 86

Sample Problem Assumptions

The sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in Part C and are provided in Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of the Part C predictive method to actual field conditions.

Section 87

Calculation of this performance measure follows Steps 1–5 outlined for the Expected Average Crash Frequency with EB Adjustments performance measure.

The results of Steps 1, 4, and 5 that are used in calculations of the excess expected average crash frequency are summarized in the following table:

Summary of Performance Measure Calculations for Steps 1, 4, and 5

Chapter 368 ntersection	Chapter 369 ear	Chapter 370 bserved Average Crash Frequency (FI)	Chapter 371 bserved Average Crash Frequency (PDO)	Chapter 372 PF Predicted Average Crash Frequency (FI)	Chapter 373 PF Predicted Average Crash Frequency (PDO)	Chapter 374 B-Adjusted Expected Average Crash Frequency (FI)	Chapter 375 B-Adjusted Expected Average Crash Frequency (PDO)
2	1	8	1	0.6	1.1	4.9	3.8
	2	8	3	0.6	1.1	4.9	3.8
	3	9	6	0.7	1.1	5.8	3.8
3	1	8	1	0.8	1.3	3.0	3.1
	2	3	5	0.8	1.4	3.0	3.1
	3	2	4	0.9	1.4	3.3	2.8
7	1	5	6	1.0	1.6	4.3	5.0
	2	5	4	1.0	1.6	4.3	5.0
	3	8	6	1.1	1.7	4.8	5.4
10	1	4	3	0.8	1.3	1.7	2.8
	2	2	4	0.9	1.4	1.9	2.8
	3	1	3	0.9	1.4	1.9	2.6
15	1	1	5	1.0	1.6	1.6	3.8
	2	1	2	0.9	1.4	1.4	3.4
	3	3	5	0.8	1.3	1.3	3.0
17	1	2	2	1.0	1.5	1.7	2.2
	2	2	2	1.0	1.6	1.7	2.4
	3	2	3	1.0	1.6	1.7	2.2
19	1	3	2	1.0	1.5	1.7	1.7
	2	1	1	1.0	1.5	1.7	1.8
	3	2	2	1.0	1.6	1.7	2.0

The difference between the predicted estimates and EB-adjusted estimates for each intersection is the excess as calculated by Equation 4-45.

$$Excessy = (Nexpected,n(PDO) - Npredicted,n(PDO)) + (Nexpected,n(FI) - Npredicted,n(FI)) \qquad (4\text{-}45)$$

Where:

Excessy = Excess expected crashes for year, n

Nexpected,n = EB-adjusted expected average crash frequency for year, n

Npredicted,n = SPF predicted average crash frequency for year, n

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Shown below is the calculation for Intersection 7.

Excess3 = 5.4 – 1.7 + 4.8 – 1.1 = 7.4 [crashes per year]

The calculations for all TWSC intersections are summarized in Step 8.

Section 91

Calculate the severity weighted EB-adjusted excess expected crash value in dollars.

$$Excess(sw) = (Nexpected,n(PDO) - Npredicted,n(PDO)) \times CC(PDO) + (Nexpected,n(FI) - Npredicted,n(FI)) \times CC(FI) \quad (4\text{-}46)$$

Where:

Excess(sw) = Severity weighted EB-adjusted expected excess crash value

CC(Y) = Crash cost for crash severity, Y

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Shown below is the calculation for Intersection 7.

Excess(sw) = (5.4 –1.7) × $7.400 + (4.8 – 1.1) × $158,200 = $612,720

The calculations for all TWSC intersections are summarized in Step 8.

Section 93

Rank the intersections based on either EB-adjusted expected excess crashes calculated in Step 6 or based on EB-adjusted severity weighted excess crashes calculated in Step 7. The first table shows the ranking of TWSC intersections based on the EB-adjusted expected excess crashes calculated in Step 6. The intersection ranking shown in the second table is based on the EB-adjusted severity weighted excess crashes calculated in Step 7.

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Rankings according to calculations are as follows:

EB-Adjusted Excess Expected Crash Ranking

Chapter 379	Intersection	Chapter 380	Excess
2		7.8	
7		7.4	
3		3.8	
10		2.2	
15		2.2	
17		1.3	
19		1.1	

EB-Adjusted Severity Weighted Excess Crash Ranking

Chapter 381	Intersection	Chapter 382	Excess(sw)[a]
2		$826,800	
7		$612,700	
3		$390,000	
10		$167,100	
17		$115,200	
19		$113,700	
15		$91,700	

[a] All Excess(SW) values rounded to the nearest hundred dollars.

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4.4.3. Roadway Segments Performance Measure Sample Data

The Situation

A roadway agency is undertaking an effort to improve safety on their highway network. There are ten roadway segments from which the roadway agency wants to identify sites that will be studied in more detail because they show a potential for reducing the average crash frequency.

After reviewing the guidance in Section 4.2, the agency chooses to apply the sliding window method using the RSI performance measure to analyze each roadway segment. If desired, the agency could apply other performance measures or the peak searching method to compare results and confirm ranking.

The Facts

The roadway segments are comprised of:

1.2 mi of rural undivided two-lane roadway

2.1 mi are undivided urban/suburban arterial with four lanes

0.6 mi of divided urban/suburban two-lane roadway

Segment characteristics and a three-year summary of crash data is in Table 4-15.

Three years of detailed roadway segment crash data is shown in Table 4-16.

Assumptions

The roadway agency has accepted the FHWA crash costs by severity and type as shown in Table 4-17.

Roadway Segment Characteristics and Crash Data

Tables 4-15 and 4-16 summarize the roadway segment characteristics and crash data.

- Table 4-15. Roadway Segment Characteristics

Chapter 383 egments	S					Chapter 388 rash Data		
	Chapter 384 ross-Section (Number of Lanes)	Chapter 385 egment Length (miles)	Chapter 386 ADT	Chapter 387 vided/Divided	Undi	Tota l Year 1	Tota l Yea r 2	Tot al Yea r 3
1	2	0.80	9,000	U		16	15	14
2	2	0.40	15,000	U		12	14	10
3	4	0.50	20,000	D		6	9	5
4	4	0.50	19,200	D		7	5	1
5	4	0.35	22,000	D		18	16	15
6	4	0.30	25,000	D		14	12	10
7	4	0.45	26,000	D		12	11	13
8	2	0.20	10,000	U		2	1	3
9	2	0.25	14,000	U		3	2	1
10	2	0.15	15,000	U		1	2	1

- Table 4-16. Roadway Segment Detail Crash Data Summary (3 Years)

		Chapter 391 Severity		Crash	Chapter 392		Crash Type					
Chapter 38 egment	Chapter 3 otal	Chapter 3 atal	Chapter 3 njury	Chapter 3 DO	Chapter 3 ear-End	Chapter 3 ngle	Chapter 3 ead-On	Chapter 39 ideswipe	Chapter 40 edestrian	Chapter 4 ixed Object	Chapter 40 ollover	Chapter 4 ther
1	45	3	17	25	0	0	6	5	0	15	19	0
2	36	0	5	31	0	1	3	3	3	14	10	2
3	20	0	9	11	1	0	5	5	0	5	3	1
4	13	0	5	8	3	0	1	2	0	4	0	3

5	49	0	9	40	1	1	21	12	2	5	5	2
6	36	0	5	31	4	0	11	10	0	5	4	2
7	36	0	6	30	2	0	13	11	0	4	3	3
8	6	0	1	5	2	0	0	1	0	1	0	2
9	6	0	1	5	1	0	0	1	0	2	0	2
10	4	0	0	4	2	0	0	0	0	1	0	1

- Table 4-17. Relative Severity Index Crash Costs

Chapter 404 Crash Type	Chapter 405 Crash Costs	RSI
Rear-End, Non-Intersection	$30,100	
Sideswipe/Overtaking	$34,000	
Angle, Non-Intersection	$56,100	
Pedestrian/Bike, Non-Intersection	$287,900	
Head-On, Non-Intersection	$375,100	
Rollover	$239,700	
Fixed Object	$94,700	
Other/Undefined	$55,100	

13 Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA-HRT-05-051, October 2005

Sliding Window Procedure

The sliding window approach is one analysis method that can be applied when screening roadway segments. It consists of conceptually sliding a window of a specified length along the road segment in increments of a specified size. The method chosen to screen the segment is applied to each position of the window and the results of the analysis are recorded for each window. The window that shows the greatest potential for improvement is used to represent the total performance of the segment. After all segments are ranked according to the respective highest window value, those segments with the greatest potential for reduction in crash frequency or severity are studied in detail to identify potential countermeasures.

The following assumptions are used to apply the sliding window analysis technique in the roadway segment sample problems:

Segment 1 extends from mile point 1.2 to 2.0

The length of window in the sliding window analysis is 0.3 mi.

The window slides in increments of 0.1 mi.

The name of the window subsegments and the limits of each subsegment are summarized in Table 4-18.

- Table 4-18. Segment 1 Sliding Window Parameters

Chapter 406 Subsegments	Window	Chapter 407 Limit (Mile Point)	Beginning	Chapter 408 Limit (Mile Point)	Ending
1a		1.2		1.5	
1b		1.3		1.6	

1c	1.4	1.7
1d	1.5	1.8
1e	1.6	1.9
1f	1.7	2.0

The windows shown in Table 4-18 are the windows used to evaluate Segment 1 throughout the roadway segment sample problems. Therefore, whenever window subsegment 1a is referenced, it is the portion of Segment 1 that extends from mile point 1.2 to 1.5 and so forth.

Table 4-19 summarizes the crash data for each window subsegment within Segment 1. This data will be used throughout the roadway segment sample problems to illustrate how to apply each screening method.

- Table 4-19. Segment 1 Crash Data per Sliding Window Subsegments

Chapter 409		Chapter 411	Crash Severity		Chapter 412	Crash Type		
indow Subsegments	Chapter 410 otal	Chapter 413 atal	Chapter 414 njury	Chapter 415 DO	Chapter 416 ead-On	Chapter 417 ideswipe	Chapter 418 ixed Object	Chapter 419 ollover
1a	8	0	3	5	0	0	3	5
1b	8	0	4	4	1	1	3	3
1c	7	0	3	4	3	1	0	3
1d	11	2	3	6	1	2	5	3
1e	4	0	0	4	0	0	1	3
1f	7	1	4	2	1	1	3	2

When the sliding window approach is applied to a method, each segment is ranked based on the highest value found on that segment.

For each window subsegment, multiply the average crash frequency for each crash type by their respective RSI crash type.

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The following table summarizes the observed average crash frequency by crash type for each window subsegment over the last three years and the corresponding RSI crash costs for each crash type.

Crash Type Summary for Segment 1 Window Subsegments

Chapter 421	W Chapter 422	Chapter 423	S Chapter 424	Chapter 425	R Chapter 426
indow Subsegments	ead-On	ideswipe	ixed Object	ollover	otal[a]
Chapter 427	Observed Average Crash Frequency				
1a	0	0	3	5	8
1b	1	1	3	3	8

1c	3	1	0	3	7
1d	1	2	5	3	11
1e	0	0	1	3	4
1f	1	1	3	2	7
Chapter 428	RSI Crash Costs per Crash Typeb				
1a	$0	$0	$284,100	$1,198,500	$1,482,600
1b	$375,100	$34,000	$284,100	$719,100	$1,412,300
1c	$1,125,300	$34,000	$0	$719,100	$1,878,400
1d	$375,100	$68,000	$473,500	$719,100	$1,635,700
1e	$0	$0	$94,700	$719,100	$813,800
1f	$375,100	$34,000	$284,100	$479,400	$1,172,600

a Crash types that were not reported to have occurred on Roadway Segment 1 were omitted from the table. The RSI costs for these crash types are zero.

b The values in this table are the result of multiplying the average crash frequency for each crash type by the corresponding RSI cost.

The calculation for Window Subsegment 1d is shown below.

Total RSI Cost = (1 × $375,100) + (2 × $34,000) + (5 × $94,700) + (3 × $239,700) = $1,635,700

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Sum the RSI costs for all crash types and divide by the total average crash frequency for the specific window subsegment as shown in Equation 4-47. The result is an Average RSI cost for each window subsegment.

(4-47)

Where:

Nobserved,i(total) = Total observed crashes at site, i

Section 98

The calculation for Window Subsegment 1d is:

The following table summarizes the Average RSI Crash Cost calculation for each window subsegment within Segment 1.

Average RSI Crash Cost per Window Subsegment

Chapter 430 Subsegment	Window	Chapter 431 Number of Crashes	Total	Chapter 432 Value	Total RSI	Chapter 433 RSI Value	Average
1a		8		$1,482,600		$185,300	
1b		8		$1,412,300		$176,500	

1c	7	$1,878,400	$268,300
1d	11	$1,635,700	$148,700
1e	4	$813,800	$203,500
1f	7	$1,172,600	$167,500

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Calculate the average RSI cost for the entire population by summing the total RSI costs for each site and dividing by the total average crash frequency within the population. In this sample problem, the population consists of Segment 1 and Segment 2. Preferably, there are more than two Segments within a population; however, for the purpose of illustrating the concept and maintaining brevity, this set of example problems only has two segments within the population.

The average RSI cost for the population () is calculated using Equation 4-48.

(4-48)

Where:

= Average RSI cost for the population

RSIi = RSI cost per site in the population

Nobserved,i = Number of observed crashes in the population

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The following example summarizes the information needed to calculate the average RSI cost for the population.

Average RSI Cost for Two-Lane Undivided Rural Highway Population

Chapter 435 oadway Segments	Chapter 436 ngle	Chapter 437 ead-On	Chapter 438 ide-swipe	Chapter 439 edestrian	Chapter 440 ixed Object	Chapter 441 ollover	Chapter 442 ther	Chapter 443 otal
Chapter 444 Average Crash Frequency Over Three Years								
1	0	6	5	0	15	19	0	45
2	1	3	3	3	14	10	2	36
Chapter 445 RSI Crash Costs per Crash Type								
1	$0	$2,250,600	$170,000	$0	$1,420,500	$4,554,300	$0	$8,395,400
2	$56,100	$1,125,300	$102,000	$863,700	$1,325,800	$2,397,000	$110,000	$5,979,900

Below is the average RSI cost calculation for the Rural Two-Lane Highway population. This can be used as a threshold for comparison of RSI cost of individual subsegments within a segment.

Steps 1 and 2 are repeated for each roadway segment and Step 3 is repeated for each population. The roadway segments are ranked using the highest average RSI cost calculated for each roadway segment. For example, Segment 1 would be ranked using the highest average RSI cost shown in Step 2 from Window Subsegment 1c ($268,300). The highest average RSI cost for each roadway segment is also compared to the average RSI cost for the entire population. This comparison indicates whether or not the roadway segment's average RSI cost is above or below the average value for similar locations.

4.5. REFERENCES

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New Appendix to Chapter 4 — Crash Models for Network Screening Applications for Roundabouts

Appendix 4B provides SPFs for roundabouts that can be used for Network Screening. Chapter 4 discusses Network Screening approaches, including those that apply SPFs and Section 4.4 provides detailed sample applications and illustrates how SPFs are used. The SPFs in appendix 4B include basic, readily available variables and can be applied when the sites under consideration are roundabouts. They need to be used for calculation of SPF Predicted Average Crash Frequency in the following methods:

- Excess Predicted Average Crash Frequency Using Method of Moments (Sec. 4.4.2.6)
- Level of Service of Safety (LOSS) (Sec. 4.4.2.7)
- Excess Predicted Average Crash Frequency Using SPFs (Sec. 4.4.2.8)
- Excess Expected Average Crash Frequency with EB Adjustments (4.4.2.13)

For the last method, the Part C Predictive Methodology and SPFs in Chapter 12 may also be used if the required variables are available for all sites subject to the screening.

The SPFs were developed for NCHRP project 17-70 using generalized linear modeling with a negative binomial error structure. Each SPF predicts the average crash frequency of one roundabout, inclusive of crashes within the circulating roadway and those crashes on the roundabout legs that are considered related to the roundabout (i.e., the leg geometry or operation was a likely contributing factor in the crash). For this reason, the SPFs are referred to as "intersection-level" models. Bicycle and pedestrian involved crashes are not included in the estimates.

The SPFs provided fall into three categories:

1. Rural single- or two-lane roundabouts,
2. Urban single-lane roundabouts, or,
3. Urban multi-lane roundabouts.

Prior to applying the SPFs it is desirable to calibrate the SPFs to the jurisdiction in which they will be applied. Calibration provides a method to account for differences between jurisdictions in factors such as climate, driver populations, animal populations, crash reporting thresholds, and crash reporting system procedures. The calibration procedure is documented in [Production Contract to Insert Appropriate Chapter or Appendix in HSM].

The form of the SPF for rural roundabouts is provided in equation 4B-1 and the parameter estimates in table 4B-1.

$$N = exp^{a} MAJAADT^{b} MINAADT^{c} exp^{(d\ x\ NUMBERLEGS + e\ x\ CIRCLANES)}$$ Equation 4B-1

where,

N = predicted average crash frequency, crashes/yr
MAJAADT = Total entering AADT on major road
MINAADT = Total entering AADT on minor road
NUMBERLEGS = 1 if a 3-leg roundabout; 0 if 4-legs
CIRCLANES = 1 if a single lane roundabout; 0 if more than 1 circulating lanes
k = overdispersion parameter

Table 4B-1. Network Screening SPFs for Rural Roundabouts

Chapter 447 rash Severity	Chapter 448	Chapter 449	Chapter 450	Chapter 451	Chapter 452	Chapter 453
Total (KABCO)	-5.3299	0.3356	0.5142	-0.6854	-0.9375	0.6292
Fatal and Injury (KABC)	-10.4848	0.7756	0.4239	-1.0080	-0.5506	0.4424
PDO (O)	-5.4115	0.2980	0.5463	-0.7104	-1.0192	0.7284

The form of the SPF for urban single-lane and mulit-lane roundabouts is provided in equation 4B-2 and the parameter estimates in tables 4B-2 and 4B-3.

$$N = exp^{a} MAJAADT^{b} MINAADT^{c} exp^{(d\ x\ NUMBERLEGS)}$$ Equation 4B-2

where,

N = predicted average crash frequency, crashes/yr
MAJAADT = Total entering AADT on major road
MINAADT = Total entering AADT on minor road
NUMBERLEGS = 1 if a 3-leg roundabout; 0 if 4-legs
k = overdispersion parameter

Table 4B-2. Network Screening SPFs for Urban Single-Lane Roundabouts

Chapter 454 C rash Severity	Chapter 455	Chapter 456	Chapter 457	Chapter 458	Chapter 459
Total (KABCO)	-5.6049	0.3274	0.3960	-0.8681	0.5030
Fatal and Injury (KABC)	-8.6597	0.5271	0.3505	-0.7317	0.3290
PDO (O)	-5.5319	0.2653	0.4294	-0.9260	0.6064

Table 4B-3. Network Screening SPFs for Urban Multi-Lane Roundabouts

Chapter 460 C rash Severity	Chapter 461	Chapter 462	Chapter 463	Chapter 464	Chapter 465
Total (KABCO)	-5.6642	0.5210	0.2905	-0.4610	0.9263
Fatal and Injury (KABC)	-10.3369	0.9134	0.1937	-0.5131	0.5611
PDO (O)	-5.7669	0.4954	0.3098	-0.4618	1.0642

Chapter 12 Predictive Method for Urban and Suburban Arterials

INTRODUCTION

This chapter presents the predictive method for urban and suburban arterial facilities. A general introduction to the *Highway Safety Manual* (HSM) predictive method is provided in the Part C—Introduction and Applications Guidance.

The predictive method for urban or suburban arterial facilities provides a structured methodology to estimate the expected average crash frequency, crash severity, and collision types for facilities with known characteristics. All types of crashes involving vehicles of all types, bicycles, and pedestrians are included, with the exception of crashes between bicycles and pedestrians. The predictive method can be applied to existing sites, design alternatives to existing sites, new sites, or for alternative traffic volume projections. An estimate can be made for crash frequency in a period of time that occurred in the past (i.e., what did or would have occurred) or in the future (i.e., what is expected to occur). The development of the SPFs in Chapter 12 is documented by Harwood et al. (8, 9). The CMFs used in this chapter have been reviewed and updated by Harkey et al. (6) and in related work by Srinivasan et al. (13). The SPF coefficients, default collision type distributions, and default nighttime crash proportions have been adjusted to a consistent basis by Srinivasan et al. (14).

This chapter presents the following information about the predictive method for urban and suburban arterial facilities:

- A concise overview of the predictive method.
- The definitions of the facility types included in Chapter 12, and site types for which predictive models have been developed for Chapter 12.
- The steps of the predictive method in graphical and descriptive forms.
- Details for dividing an urban or suburban arterial facility into individual sites, consisting of intersections and roadway segments.
- Safety performance functions (SPFs) for urban and suburban arterials.
- Crash modification factors (CMFs) applicable to the SPFs in Chapter 12.
- Guidance for applying the Chapter 12 predictive method, and limitations of the predictive method specific to Chapter 12.
- Sample problems illustrating the application of the Chapter 12 predictive method for urban and suburban arterials.

OVERVIEW OF THE PREDICTIVE METHOD

The predictive method provides an 18-step procedure to estimate the "expected average crash frequency," $N_{expected}$ (by total crashes, crash severity, or collision type) of a roadway network, facility, or site. In the predictive method, the roadway is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments referred to as "sites." Different facility types are determined by surrounding land use, roadway cross-section, and degree of access. For each facility type, a number of different site types may exist, such as divided and undivided roadway segments and signalized and unsignalized intersections. A roadway network consists of a number of contiguous facilities.

The method is used to estimate the expected average crash frequency of an individual site, with the cumulative sum of all sites used as the estimate for an entire facility or network. The estimate is for a given time period of interest (in years) during which the geometric design and traffic control features are unchanged and traffic volumes are known or forecasted. The estimate relies on estimates made using predictive models which are combined with observed crash data using the Empirical Bayes (EB) Method.

The predictive models used within the Chapter 12 predictive method are described in detail in Section 12.3.

The predictive models used in Chapter 12 to predict average crash frequency, $N_{\text{predicted}}$, are of the general form shown in Equation 12-1.

$$N_{\text{predicted}} = (N_{spf\ x} \times (CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}) + N_{pedx} + N_{bikex}) \times C_x \qquad (12\text{-}1)$$

Where:

$N_{\text{predicted}}$ = predicted average crash frequency for a specific year on site type x;

$N_{spf\ x}$ = predicted average crash frequency determined for base conditions of the SPF developed for site type x;

N_{pedx} = predicted average number of vehicle-pedestrian collisions per year for site type x;

N_{bikex} = predicted average number of vehicle-bicycle collisions per year for site type x;

CMF_{yx} = crash modification factors specific to site type x and specific geometric design and traffic control features y; and

C_x = calibration factor to adjust SPF for local conditions for site type x.

The predictive models in Chapter 12 provide estimates of the crash severity and collision type distributions for roadway segments and intersections. The SPFs in Chapter 12 address two general crash severity levels: fatal-and-injury and property-damage-only crashes. Fatal-and-injury crashes include crashes involving all levels of injury severity including fatalities, incapacitating injuries, nonincapacitating injuries, and possible injuries. The relative proportions of crashes for the two severity levels are determined from separate SPFs for each severity level. The default estimates of the crash severity and crash type distributions are provided with the SPFs for roadway segments and intersections in Section 12.6.

URBAN AND SUBURBAN ARTERIALS—DEFINITIONS AND PREDICTIVE MODELS IN CHAPTER 12

This section provides the definitions of the facility and site types and the predictive models for each of the site types included in Chapter 12. These predictive models are applied following the steps of the predictive method presented in Section 12.4.

Definition of Chapter 12 Facility Types

The predictive method in Chapter 12 addresses the following urban and suburban arterial facilities: two- and four-lane undivided facilities, four-lane divided facilities, and three- and five-lane facilities with center two-way left-turn lanes. Divided arterials are nonfreeway facilities (i.e., facilities without full control of access) that have lanes in the two directions of travel separated by a raised or depressed median. Such facilities may have occasional grade-separated interchanges, but these are not the primary form of access. The predictive models do not apply to any

section of an arterial within the limits of an interchange which has free-flow ramp terminals on the arterial of interest. Arterials with a flush separator (i.e., a painted median) between the lanes in the two directions of travel are considered undivided facilities, not divided facilities. Separate prediction models are provided for arterials with a flush separator that serves as a center two-way left-turn lane. Chapter 12 does not address arterial facilities with six or more lanes.

The terms "highway" and "road" are used interchangeably in this chapter and apply to all urban and suburban arterials independent of official state or local highway designation.

Classifying an area as urban, suburban, or rural is subject to the roadway characteristics, surrounding population and land uses and is at the user's discretion. In the HSM, the definition of "urban" and "rural" areas is based on Federal Highway Administration (FHWA) guidelines which classify "urban" areas as places inside urban boundaries where the population is greater than 5,000 persons. "Rural" areas are defined as places outside urban areas where the population is less than 5,000 persons. The HSM uses the term "suburban" to refer to outlying portions of an urban area; the predictive method does not distinguish between urban and suburban portions of a developed area. The term "arterial" refers to facilities the meet the FHWA definition of "roads serving major traffic movements (high-speed, high volume) for travel between major points" (5).

Table 12-1 identifies the specific site types on urban and suburban arterial highways that have predictive models. In Chapter 12, separate SPFs are used for each individual site to predict multiple-vehicle nondriveway collisions, single-vehicle collisions, driveway-related collisions, vehicle-pedestrian collisions, and vehicle-bicycle collisions for both roadway segments and intersections. These are combined to predict the total average crash frequency at an individual site.

Table 12-1. Urban and Suburban Arterial Site Type SPFs included in Chapter 12

Site Type	Site Types with SPFs in Chapter 12
Roadway Segments	Two-lane undivided arterials (2U)
	Three-lane arterials including a center two-way left-turn lane (TWLTL) (3T)
	Four-lane undivided arterials (4U)
	Four-lane divided arterials (i.e., including a raised or depressed median) (4D)
	Five-lane arterials including a center TWLTL (5T)
Intersections	Unsignalized three-leg intersection (stop control on minor-road approaches) (3ST)
	Signalized three-leg intersections (3SG)
	Unsignalized four-leg intersection (stop control on minor-road approaches) (4ST)
	Signalized four-leg intersection (4SG)
	Three-leg single-lane roundabouts (31R)
	Four-leg single-lane roundabouts (41R)
	Three-leg two-lane roundabouts (32R)
	Four-leg two-lane roundabouts (42R)

These specific site types are defined as follows:

- *Two-lane undivided arterial (2U)*—a roadway consisting of two lanes with a continuous cross-section providing two directions of travel in which the lanes are not physically separated by either distance or a barrier.

- *Three-lane arterials (3T)*—a roadway consisting of three lanes with a continuous cross-section providing two directions of travel in which center lane is a two-way left-turn lane (TWLTL).
- *Four-lane undivided arterials (4U)*—a roadway consisting of four lanes with a continuous cross-section providing two directions of travel in which the lanes are not physically separated by either distance or a barrier.
- *Four-lane divided arterials (i.e., including a raised or depressed median) (4D)*—a roadway consisting of two lanes with a continuous cross-section providing two directions of travel in which the lanes are physically separated by either distance or a barrier.
- *Five-lane arterials including a center TWLTL (5T)*—a roadway consisting of five lanes with a continuous cross-section providing two directions of travel in which the center lane is a two-way left-turn lane (TWLTL).
- *Three-leg intersection with stop control (3ST)*—an intersection of a urban or suburban arterial and a minor road. A stop sign is provided on the minor road approach to the intersection only.
- *Three-leg signalized intersection (3SG)*—an intersection of a urban or suburban arterial and one minor road. Signalized control is provided at the intersection by traffic lights.
- *Four-leg intersection with stop control (4ST)*—an intersection of a urban or suburban arterial and two minor roads. A stop sign is provided on both the minor road approaches to the intersection.
- *Four-leg signalized intersection (4SG)*—an intersection of a urban or suburban arterial and two minor roads. Signalized control is provided at the intersection by traffic lights.
- *Three-leg single-lane roundabout (31R)*—a roundabout with 3 legs and a single circulating lane conflicting with each leg.
- *Four-leg single-lane roundabout (41R)*—a roundabout with 4 legs and a single circulating lane conflicting with each leg.
- *Three-leg two-lane roundabout (32R)*—a roundabout with 3 legs and two circulating lanes conflicting with one or more legs.
- *Four-leg two-lane roundabout (42R)*—a roundabout with 4 legs and two circulating lanes conflicting with one or more legs.

Predictive Models for Urban and Suburban Arterial Roadway Segments

The predictive models can be used to estimate total average crashes (i.e., all crash severities and collision types) or can be used to predict average frequency of specific crash severity types or specific collision types. The predictive model for an individual roadway segment or intersection combines the SPF, CMFs, and a calibration factor. Chapter 12 contains separate predictive models for roadway segments and for intersections.

The predictive models for roadway segments estimate the predicted average crash frequency of non-intersection-related crashes. Non-intersection-related crashes may include crashes that occur within the limits of an intersection but are not related to the intersection. The roadway segment predictive models estimate crashes that would occur regardless of the presence of the intersection.

The predictive models for roadway segments are presented in Equations 12-2 and 12-3 below.

$$N_{\text{predicted } rs} = C_r \times (N_{br} + N_{pedr} + N_{biker}) \quad (12\text{-}2)$$

$$N_{br} = N_{spfrs} \times (CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{nr}) \quad (12\text{-}3)$$

Where:

$N_{\text{predicted } rs}$ = predicted average crash frequency of an individual roadway segment for the selected year;

N_{br} = predicted average crash frequency of an individual roadway segment (excluding vehicle-pedestrian and vehicle-bicycle collisions);

$N_{spf\ rs}$ = predicted total average crash frequency of an individual roadway segment for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions);

N_{pedr} = predicted average crash frequency of vehicle-pedestrian collisions for an individual roadway segment;

N_{biker} = predicted average crash frequency of vehicle-bicycle collisions for an individual roadway segment;

$CMF_{1r} \ldots CMF_{nr}$ = crash modification factors for roadway segments; and

C_r = calibration factor for roadway segments of a specific type developed for use for a particular geographical area.

Equation 12-2 shows that roadway segment crash frequency is estimated as the sum of three components: N_{br}, N_{pedr}, and N_{biker}. The following equation shows that the SPF portion of N_{br}, designated as $N_{spf\ rs}$, is further separated into three components by collision type shown in Equation 12-4:

$$N_{spf\ rs} = N_{brmv} + N_{brsv} + N_{brdwy} \tag{12-4}$$

Where:

N_{brmv} = predicted average crash frequency of multiple-vehicle nondriveway collisions for base conditions;

N_{brsv} = predicted average crash frequency of single-vehicle crashes for base conditions; and

N_{brdwy} = predicted average crash frequency of multiple-vehicle driveway-related collisions.

Thus, the SPFs and adjustment factors are applied to determine five components: $N_{brmv,}$ N_{brsv}, N_{brdwy}, N_{pedr}, and N_{biker}, which together provide a prediction of total average crash frequency for a roadway segment.

Equations 12-2 through 12-4 are applied to estimate roadway segment crash frequencies for all crash severity levels combined (i.e., total crashes) or for fatal-and-injury or property-damage-only crashes.

Predictive Models for Urban and Suburban Arterial Intersections

The predictive models for intersections estimate the predicted total average crash frequency including those crashes that occur within the limits of an intersection and are a result of the presence of the intersection. The predictive model for an urban or suburban arterial intersection is given by:

$$N_{\text{predicted } int} = C_i \times (N_{bi} + N_{pedi} + N_{bikei}) \tag{12-5}$$

$$N_{bi} = N_{spf\ int} \times (CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{6i}) \tag{12-6}$$

Where:

N_{int} = predicted average crash frequency of an intersection for the selected year;

N_{bi} = predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle collisions);

$N_{spf\ int}$ = predicted total average crash frequency of intersection-related crashes for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions);

N_{pedi} = predicted average crash frequency of vehicle-pedestrian collisions;

N_{bikei} = predicted average crash frequency of vehicle-bicycle collisions;

$CMF_{1i} \ldots CMF_{6i}$ = crash modification factors for intersections; and

C_i = calibration factor for intersections developed for use for a particular geographical area.

The CMFs shown in Equation 12-6 do not apply to vehicle-pedestrian and vehicle-bicycle collisions. A separate set of CMFs that apply to vehicle-pedestrian collisions at signalized intersections is presented in Section 12.7.

Equation 12-5 shows that the intersection crash frequency is estimated as the sum of three components: N_{bi}, N_{pedi}, and N_{bikei}. For signalized and stop-controlled intersections, the following equation shows that the SPF portion of N_{bi}, designated as $N_{spf\ int}$, is further separated into two components by collision type:

$$N_{spf\ int} = N_{bimv} + N_{bisv} \tag{12-7}$$

Where:

N_{bimv} = predicted average number of multiple-vehicle collisions for base conditions; and

N_{bisv} = predicted average number of single-vehicle collisions for base conditions.

Thus, the SPFs and adjustment factors are applied to determine four components of total intersection average crash frequency: N_{bimv}, N_{bisv}, N_{pedi}, and N_{bikei}.

For intersection-level predictions at roundabouts, the SPF portion of N_{bi} is predicted using separate SPFs for fatal+injury and PDO crash severities.

For leg-level predictions at roundabouts, SPFs are available for several crash types as well as for total crashes excluding those involving a pedestrian or bicycle.

The SPFs for urban and suburban arterial highways are presented in Section 12.6. The associated CMFs for each of the SPFs are presented in Section 12.7 and summarized in Table 12-18. Only the specific CMFs associated with each SPF are applicable to that SPF (as these CMFs have base conditions which are identical to the base conditions of the SPF). The calibration factors, C_r and C_i, are determined in Part C, Appendix A.1.1. Due to continual change in the crash frequency and severity distributions with time, the value of the calibration factors may change for the selected year of the study period.

PREDICTIVE METHOD STEPS FOR URBAN AND SUBURBAN ARTERIALS

The predictive method for urban and suburban arterials is shown in Figure 12-1. Applying the predictive method yields an estimate of the expected average crash frequency (and/or crash severity and collision types) for an urban or

suburban arterial facility. The components of the predictive models in Chapter 12 are determined and applied in Steps 9, 10, and 11 of the predictive method. The information to apply each step is provided in the following sections and in Part C, Appendix A. In some situations, certain steps will not require any action. For example, a new facility will not have observed crash data and therefore steps relating to the EB Method require no action.

There are 18 steps in the predictive method. In some situations certain steps will not be needed because data is not available or the step is not applicable to the situation at hand. In other situations, steps may be repeated if an estimate is desired for several sites or for a period of several years. In addition, the predictive method can be repeated as necessary to undertake crash estimation for each alternative design, traffic volume scenario, or proposed treatment option (within the same period to allow for comparison).

The following explains the details of each step of the method as applied to urban and suburban arterials.

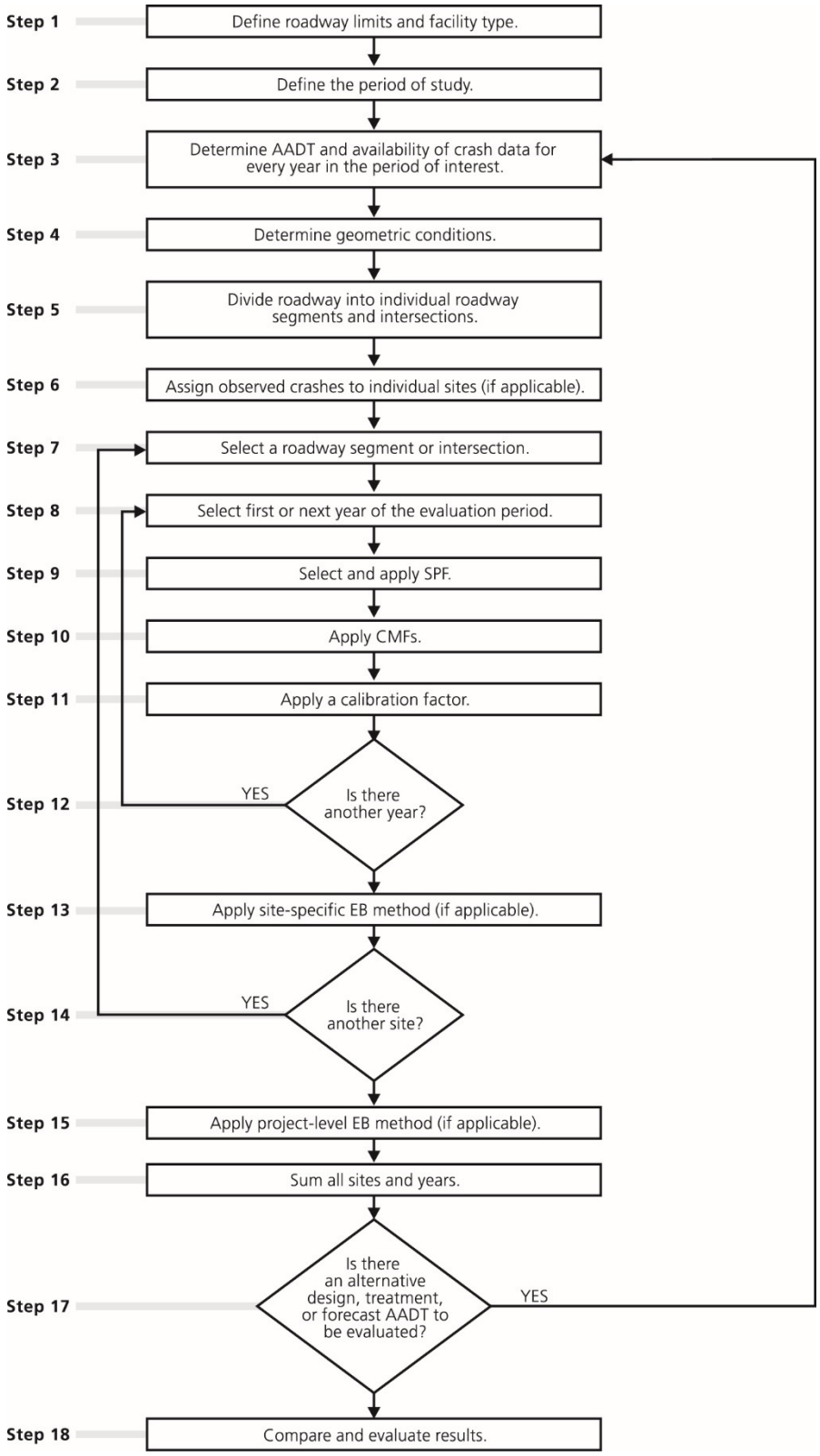


Figure 12-1. The HSM Predictive Method

Step 1—Define the limits of the roadway and facility types in the study network, facility, or site for which the expected average crash frequency, severity, and collision types are to be estimated.
The predictive method can be undertaken for a roadway network, a facility, or an individual site. A site is either an intersection or a homogeneous roadway segment. Sites may consist of a number of types, such as signalized and unsignalized intersections. The definitions of urban and suburban arterials, intersections, and roadway segments and the specific site types included in Chapter 12 are provided in Section 12.3.

The predictive method can be undertaken for an existing roadway, a design alternative for an existing roadway, or a new roadway (which may be either unconstructed or yet to experience enough traffic to have observed crash data).

The limits of the roadway of interest will depend on the nature of the study. The study may be limited to only one specific site or a group of contiguous sites. Alternatively, the predictive method can be applied to a very long corridor for the purposes of network screening which is discussed in Chapter 4.

Step 2—Define the period of interest.
The predictive method can be undertaken for either a past period or a future period. All periods are measured in years. Years of interest will be determined by the availability of observed or forecast average annual daily traffic (AADT) volumes, observed crash data, and geometric design data. Whether the predictive method is used for a past or future period depends upon the purpose of the study. The period of study may be:

- A past period (based on observed AADTs) for:
  - An existing roadway network, facility, or site. If observed crash data are available, the period of study is the period of time for which the observed crash data are available and for which (during that period) the site geometric design features, traffic control features and traffic volumes are known.
  - An existing roadway network, facility, or site for which alternative geometric design features or traffic control features are proposed (for near term conditions).
- A future period (based on forecast AADTs) for:
  - An existing roadway network, facility, or site for a future period where forecast traffic volumes are available.
  - An existing roadway network, facility, or site for which alternative geometric design or traffic control features are proposed for implementation in the future.
  - A new roadway network, facility, or site that does not currently exist but is proposed for construction during some future period.

Step 3—For the study period, determine the availability of annual average daily traffic volumes, pedestrian crossing volumes, and, for an existing roadway network, the availability of observed crash data (to determine whether the EB Method is applicable).

Determining Traffic Volumes
The SPFs used in Step 9 (and some CMFs in Step 10) include AADT volumes (vehicles per day) as a variable. For a past period the AADT may be determined by an automated recording or estimated by a sample survey. For a future period, the AADT may be a forecast estimate based on appropriate land use planning and traffic volume forecasting models or based on the assumption that current traffic volumes will remain relatively constant.

For each roadway segment, the AADT is the average daily two-way 24-hour traffic volume on that roadway segment in each year of the period to be evaluated selected in Step 8.

For each intersection, two values are required in each predictive model. These are: the two-way AADT of the major street ($AADT_{maj}$) and the two-way AADT of the minor street ($AADT_{min}$).

$AADT_{maj}$ and $AADT_{min}$ are determined as follows: if the AADTs on the two major-road legs of an intersection differ, the larger of the two AADT values is used for the intersection. If the AADTs on the two minor road legs of a four-leg intersection differ, the larger of the AADTs for the two minor road legs is used. For a three-leg intersection, the AADT of the single minor road leg is used. If AADTs are available for every roadway segment along a facility, the major-road AADTs for intersection legs can be determined without additional data.

In many cases, it is expected that AADT data will not be available for all years of the evaluation period. In that case, an estimate of AADT for each year of the evaluation period is interpolated or extrapolated, as appropriate. If there is not an established procedure for doing this, the following may be applied within the predictive method to estimate the AADTs for years for which data are not available.

- If AADT data are available for only a single year, that same value is assumed to apply to all years of the before period.
- If two or more years of AADT data are available, the AADTs for intervening years are computed by interpolation.
- The AADTs for years before the first year for which data are available are assumed to be equal to the AADT for that first year.
- The AADTs for years after the last year for which data are available are assumed to be equal to the last year.

If the EB Method is used (discussed below), AADT data are needed for each year of the period for which observed crash frequency data are available. If the EB Method will not be used, AADT data for the appropriate time period—past, present, or future—determined in Step 2 are used.

For signalized intersections, the pedestrian volumes crossing each intersection leg are determined for each year of the period to be evaluated. The pedestrian crossing volumes for each leg of the intersection are then summed to determine the total pedestrian crossing volume for the intersection. Where pedestrian volume counts are not available, they may be estimated using the guidance presented in Table 12-15. Where pedestrian volume counts are not available for each year, they may be interpolated or extrapolated in the same manner as explained above for AADT data.

Determining Availability of Observed Crash Data

Where an existing site or alternative conditions for an existing site are being considered, the EB Method is used. The EB Method is only applicable when reliable observed crash data are available for the specific study roadway network, facility, or site. Observed data may be obtained directly from the jurisdiction's crash report system. At least two years of observed crash frequency data are desirable to apply the EB Method. The EB Method and criteria to determine whether the EB Method is applicable are presented in Part C, Appendix A.2.1.

The EB Method can be applied at the site-specific level (i.e., observed crashes are assigned to specific intersections or roadway segments in Step 6) or at the project level (i.e., observed crashes are assigned to a facility as a whole). The site-specific EB Method is applied in Step 13. Alternatively, if observed crash data are available but cannot be assigned to individual roadway segments and intersections, the project level EB Method is applied (in Step 15).

If observed crash frequency data are not available, then Steps 6, 13, and 15 of the predictive method are not conducted. In this case the estimate of expected average crash frequency is limited to using a predictive model (i.e., the predictive average crash frequency).

Step 4—Determine geometric design features, traffic control features, and site characteristics for all sites in the study network.

In order to determine the relevant data needs and avoid unnecessary collection of data, it is necessary to understand the base conditions and CMFs in Step 9 and Step 10. The base conditions are defined in Section 12.6.1 for roadway segments and in Section 12.6.2 for intersections.

The following geometric design and traffic control features are used to determine whether the site specific conditions vary from the base conditions and, therefore, whether a CMF is applicable:

- Length of roadway segment (miles)
- AADT (vehicles per day)
- Number of through lanes
- Presence/type of median (undivided, divided by raised or depressed median, center TWLTL)
- Presence/type of on-street parking (parallel vs. angle; one side vs. both sides of street)
- Number of driveways for each driveway type (major commercial, minor commercial; major industrial/institutional; minor industrial/institutional; major residential; minor residential; other)
- Roadside fixed object density (fixed objects/mile, only obstacles 4-in or more in diameter that do not have a breakaway design are counted)
- Average offset to roadside fixed objects from edge of traveled way (feet)
- Presence/absence of roadway lighting
- Speed category (based on actual traffic speed or posted speed limit)
- Presence of automated speed enforcement

For signalized or stop-controlled intersections within the study area, the following geometric and traffic control features are identified:

- Number of intersection legs (3 or 4)
- Type of traffic control (minor-road stop or signal)
- Number of approaches with intersection left-turn lane (all approaches, 0, 1, 2, 3, or 4 for signalized intersection; only major approaches, 0, 1, or 2, for stop-controlled intersections)
- Number of approaches with left-turn signal phasing (0, 1, 2, 3, or 4) (signalized intersections only) and type of left-turn signal phasing (permissive, protected/permissive, permissive/protected, or protected)
- Number of approaches with intersection right turn lane (all approaches, 0, 1, 2, 3, or 4 for signalized intersection; only major approaches, 0, 1, or 2, for stop-controlled intersections)
- Number of approaches with right-turn-on-red operation prohibited (0, 1, 2, 3, or 4) (signalized intersections only)
- Presence/absence of intersection lighting
- Maximum number of traffic lanes to be crossed by a pedestrian in any crossing maneuver at the intersection considering the presence of refuge islands (for signalized intersections only)
- Proportions of nighttime crashes for unlighted intersections (by total, fatal, injury, and property damage only)

For signalized intersections, land use and demographic data used in the estimation of vehicle-pedestrian collisions include:

- Number of bus stops within 1,000 feet of the intersection
- Presence of schools within 1,000 feet of the intersection
- Number of alcohol sales establishments within 1,000 feet of the intersection
- Presence of red light camera

- Number of approaches on which right-turn-on-red is allowed
- Pedestrian volumes

For all roundabouts within the study area, the following geometric features are identified for one or more of the SPFs and CMFs. Which features are required will depend on the SPFs and CMFs being applied:

- Inscribed circle diameter in ft.
- Number of approaches
- Presence of outbound only leg associated with an interchange ramp terminal
- Presence of right-turn bypass lanes
- Number of driveways or unsignalized access points within 250 ft. of entry point on each leg
- Entry width in ft. on each leg
- Circulating width in ft. in front of each leg entry
- Posted speed on approaches in m.p.h.
- Number of entering lanes
- Number of circulating lanes
- Number of exiting lanes
- Inscribed circle diameter in ft.
- Number of luminaires within 250 ft. of entry point on each leg

Step 5—Divide the roadway network or facility into individual homogenous roadway segments and intersections which are referred to as sites.
Using the information from Step 1 and Step 4, the roadway is divided into individual sites, consisting of individual homogenous roadway segments and intersections. The definitions and methodology for dividing the roadway into individual intersections and homogenous roadway segments for use with the Chapter 12 predictive models are provided in Section 12.5. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to a minimum of 0.10 miles will decrease data collection and management efforts.

Step 6—Assign observed crashes to the individual sites (if applicable).
Step 6 only applies if it was determined in Step 3 that the site-specific EB Method was applicable. If the site-specific EB Method is not applicable, proceed to Step 7. In Step 3, the availability of observed data and whether the data could be assigned to specific locations was determined. The specific criteria for assigning crashes to individual roadway segments or intersections are presented in Part C, Appendix A.2.3.

Crashes that occur at an intersection or on an intersection leg, and are related to the presence of an intersection, are assigned to the intersection and used in the EB Method together with the predicted average crash frequency for the intersection. Crashes that occur between intersections, and are not related to the presence of an intersection, are assigned to the roadway segment on which they occur. Such crashes are used in the EB Method together with the predicted average crash frequency for the roadway segment.

Step 7—Select the first or next individual site in the study network. If there are no more sites to be evaluated, proceed to Step 15.
In Step 5 the roadway network within the study limits has been divided into a number of individual homogenous sites (intersections and roadway segments).

The outcome of the HSM predictive method is the expected average crash frequency of the entire study network, which is the sum of the all of the individual sites, for each year in the study. Note that this value will be the total number of crashes expected to occur over all sites during the period of interest. If a crash frequency is desired, the total can be divided by the number of years in the period of interest.

The estimation for each site (roadway segments or intersection) is conducted one at a time. Steps 8 through 14, described below, are repeated for each site.

Step 8—For the selected site, select the first or next year in the period of interest. If there are no more years to be evaluated for that site, proceed to Step 14
Steps 8 through 14 are repeated for each site in the study and for each year in the study period.

The individual years of the evaluation period may have to be analyzed one year at a time for any particular roadway segment or intersection because SPFs and some CMFs (e.g., lane and shoulder widths) are dependent on AADT, which may change from year to year.

Step 9—For the selected site, determine and apply the appropriate safety performance function (SPF) for the site's facility type and traffic control features.
Steps 9 through 13, described below, are repeated for each year of the evaluation period as part of the evaluation of any particular roadway segment or intersection. The predictive models in Chapter 12 follow the general form shown in Equation 12-1. Each predictive model consists of a SPF, which is adjusted to site specific conditions using CMFs (in Step 10) and adjusted to local jurisdiction conditions (in Step 11) using a calibration factor (C). The SPFs, CMFs, and calibration factor obtained in Steps 9, 10, and 11 are applied to calculate the predicted average crash frequency for the selected year of the selected site. The SPFs available for urban and suburban arterials are presented in Section 12.6.

The SPF (which is a regression model based on observed crash data for a set of similar sites) determines the predicted average crash frequency for a site with the same base conditions (i.e., a specific set of geometric design and traffic control features). The base conditions for each SPF are specified in Section 12.6. A detailed explanation and overview of the SPFs are provided in Section C.6.3.

The SPFs developed for Chapter 12 are summarized in Table 12-2. For the selected site, determine the appropriate SPF for the site type (intersection or roadway segment) and the geometric and traffic control features (undivided roadway, divided roadway, stop-controlled intersection, signalized intersection, roundabout). The SPF for the selected site is calculated using the AADT determined in Step 3 ($AADT_{maj}$ and $AADT_{min}$ for intersections) for the selected year.

Each SPF determined in Step 9 is provided with default distributions of crash severity and collision type (presented in Section 12.6). These default distributions can benefit from being updated based on local data as part of the calibration process presented in Part C, Appendix A.1.1.

Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site specific geometric design and traffic control features.
In order to account for differences between the base conditions (Section 12.6) and the specific conditions of the site, CMFs are used to adjust the SPF estimate. An overview of CMFs and guidance for their use is provided in Section C.6.4, including the limitations of current knowledge related to the effects of simultaneous application of multiple CMFs. In using multiple CMFs, engineering judgment is required to assess the interrelationships and/or independence of individual elements or treatments being considered for implementation within the same project.

All CMFs used in Chapter 12 have the same base conditions as the SPFs used in Chapter 12 (i.e., when the specific site has the same condition as the SPF base condition, the CMF value for that condition is 1.00). Only the CMFs

presented in Section 12.7 may be used as part of the Chapter 12 predictive method. Table 12-18 indicates which CMFs are applicable to the SPFs in Section 12.6.

The CMFs for roadway segments are those described in Section 12.7.1. These CMFs are applied as shown in Equation 12-3.

The CMFs for intersections are those described in Section 12.7.2, which apply to both signalized and stop-controlled intersections, in Section 12.7.3, which applies to signalized intersections only, and in Section 12.7.4, which applies to roundabouts. These CMFs are applied as shown in Equations 12-6 and 12-28.

In Chapter 12, the multiple- and single-vehicle base crashes determined in Step 9 and the CMFs values calculated in Step 10 are then used to estimate the vehicle-pedestrian and vehicle-bicycle base crashes for roadway segments and signalized or stop-controlled intersections (present in Sections 12.6.1 and 12.6.2 respectively).

Step 11—Multiply the result obtained in Step 10 by the appropriate calibration factor.
The SPFs used in the predictive method have each been developed with data from specific jurisdictions and time periods. Calibration to local conditions will account for these differences. A calibration factor (C_r for roadway segments or C_i for intersections) is applied to each SPF in the predictive method. An overview of the use of calibration factors is provided in Section C.6.5. Detailed guidance for the development of calibration factors is included in Part C, Appendix A.1.1.

Steps 9, 10, and 11 together implement the predictive models in Equations 12-2 through 12-7 to determine predicted average crash frequency.

Step 12—If there is another year to be evaluated in the study period for the selected site, return to Step 8. Otherwise, proceed to Step 13.
This step creates a loop through Steps 8 to 12 that is repeated for each year of the evaluation period for the selected site.

Step 13—Apply site-specific EB Method (if applicable).
Whether the site-specific EB Method is applicable is determined in Step 3. The site-specific EB Method combines the Chapter 12 predictive model estimate of predicted average crash frequency, $N_{predicted}$ with the observed crash frequency of the specific site, $N_{observed}$. This provides a more statistically reliable estimate of the expected average crash frequency of the selected site.

In order to apply the site-specific EB Method, overdispersion parameter, k, for the SPF is also used. This is in addition to the material in Part C, Appendix A.2.4. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the site-specific EB Method to provide a weighting to $N_{predicted}$ and $N_{observed}$. Overdispersion parameters are provided for each SPF in Section 12.6.

Apply the site-specific EB Method to a future time period, if appropriate.

The estimated expected average crash frequency obtained above applies to the time period in the past for which the observed crash data were obtained. Part C, Appendix A.2.6 provides a method to convert the estimate of expected average crash frequency for a past time period to a future time period. In doing this, consideration is given to significant changes in geometric or roadway characteristics cause by the treatments considered for future time period.

Step 14—If there is another site to be evaluated, return to 7, otherwise, proceed to Step 15.
This step creates a loop through Steps 7 to 13 that is repeated for each roadway segment or intersection within the facility.

Step 15—Apply the project level EB Method (if the site-specific EB Method is not applicable).
This step is only applicable to existing conditions when observed crash data are available, but cannot be accurately assigned to specific sites (e.g., the crash report may identify crashes as occurring between two intersections, but is not accurate to determine a precise location on the segment). Detailed description of the project level EB Method is provided in Part C, Appendix A.2.5.

Step 16—Sum all sites and years in the study to estimate total crash frequency.
The total estimated number of crashes within the network or facility limits during a study period of *n* years is calculated using Equation 12-8:

$$N_{\text{total}} = \sum_{\substack{\text{all}\\ \text{roadway}\\ \text{segments}}} N_{rs} + \sum_{\substack{\text{all}\\ \text{intersections}}} N_{int} \tag{12-8}$$

Where:

N_{total} = total expected number of crashes within the limits of an urban or suburban arterial for the period of interest. Or, the sum of the expected average crash frequency for each year for each site within the defined roadway limits within the study period;

N_{rs} = expected average crash frequency for a roadway segment using the predictive method for one specific year; and

N_{int} = expected average crash frequency for an intersection using the predictive method for one specific year.

Equation 12-8 represents the total expected number of crashes estimated to occur during the study period. Equation 12-9 is used to estimate the total expected average crash frequency within the network or facility limits during the study period.

$$N_{\text{total average}} = \frac{N_{\text{total}}}{n} \tag{12-9}$$

Where:

$N_{\text{total average}}$ = total expected average crash frequency estimated to occur within the defined network or facility limits during the study period; and

n = number of years in the study period.

Step 17—Determine if there is an alternative design, treatment, or forecast AADT to be evaluated.
Steps 3 through 16 of the predictive method are repeated as appropriate for the same roadway limits but for alternative conditions, treatments, periods of interest, or forecast AADTs.

Step 18—Evaluate and compare results.
The predictive method is used to provide a statistically reliable estimate of the expected average crash frequency within defined network or facility limits over a given period of time, for given geometric design and traffic control features, and known or estimated AADT. In addition to estimating total crashes, the estimate can be made for different crash severity types and different collision types. Default distributions of crash severity and collision type are provided with each SPF in Section 12.6. These default distributions can benefit from being updated based on local data as part of the calibration process presented in Part C, Appendix A.1.1.

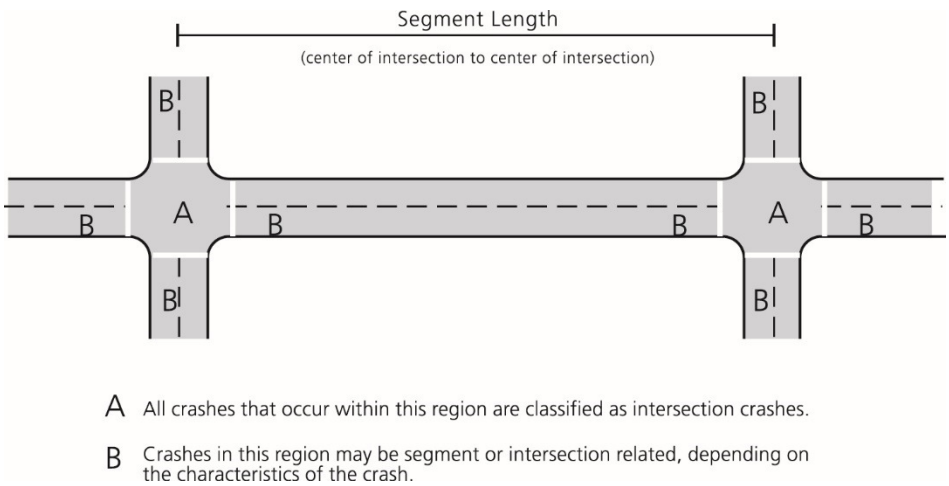
ROADWAY SEGMENTS AND INTERSECTIONS

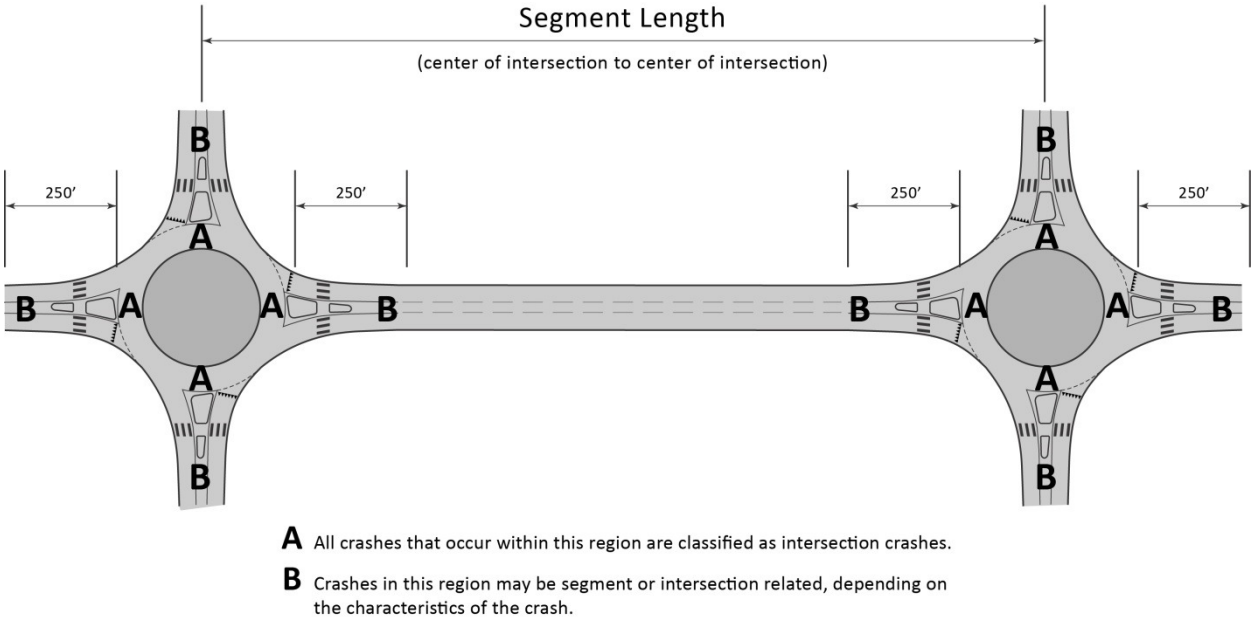
Section 12.4 provides an explanation of the predictive method. Sections 12.5 through 12.8 provide the specific detail necessary to apply the predictive method steps. Detail regarding the procedure for determining a calibration factor to apply in Step 11 is provided in Part C, Appendix A.1. Detail regarding the EB Method, which is applied in Steps 6, 13, and 15, is provided in Part C, Appendix A.2.

In Step 5 of the predictive method, the roadway within the defined limits is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as "sites." A roadway network consists of a number of contiguous facilities. Predictive models have been developed to estimate crash frequencies separately for roadway segments and intersections. The definitions of roadway segments and intersections presented below are the same as those used in the FHWA *Interactive Highway Safety Design Model* (IHSDM) (4).

Roadway segments begin at the center of an intersection and end at either the center of the next intersection or where there is a change from one homogeneous roadway segment to another homogenous segment. The roadway segment model estimates the frequency of roadway-segment-related crashes which occur in Region B in Figure 12-2. When a roadway segment begins or ends at an intersection, the length of the roadway segment is measured from the center of the intersection.

Chapter 12 provides predictive models for stop-controlled (three- and four-leg) and signalized (three- and four-leg) intersections. The intersection models estimate the predicted average frequency of crashes that occur within the limits of an intersection (Region A of Figure 12-2) and intersection-related crashes that occur on the intersection legs (Region B in Figure 12-2).





- **Figure 12-2.** Definition of Roadway Segments and Intersections

The segmentation process produces a set of roadway segments of varying length, each of which is homogeneous with respect to characteristics such as traffic volumes and key roadway design characteristics and traffic control features. Figure 12-2 shows the segment length, *L*, for a single homogenous roadway segment occurring between two intersections. However, several homogenous roadway segments can occur between two intersections. A new (unique) homogeneous segment begins at the center of each intersection and where there is a change in at least one of the following characteristics of the roadway:

- Annual average daily traffic volume (AADT) (vehicles/day)
- Number of through lanes
- Presence/type of median
- Presence of TWLTL

The following rounded widths for medians without barriers are recommended before determining "homogeneous" segments:

Measured Median Width	Rounded Median Width
1 ft to 14 ft	10 ft
15 ft to 24 ft	20 ft
25 ft to 34 ft	30 ft
35 ft to 44 ft	40 ft
45 ft to 54 ft	50 ft

55 ft to 64 ft	60 ft
65 ft to 74 ft	70 ft
75 ft to 84 ft	80 ft
85 ft to 94 ft	90 ft
95 ft or more	100 ft

- Presence/type of on-street parking
- Roadside fixed object density
- Presence of lighting
- Speed category (based on actual traffic speed or posted speed limit)
- Automated enforcement

In addition, each individual intersection is treated as a separate site for which the intersection-related crashes are estimated using the predictive method.

There is no minimum roadway segment length, *L*, for application of the predictive models for roadway segments. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to a minimum of 0.10 miles will minimize calculation efforts and not affect results.

In order to apply the site-specific EB Method, observed crashes are assigned to the individual roadway segments and intersections. Observed crashes that occur between intersections are classified as either intersection-related or roadway-segment related. The methodology for assigning crashes to roadway segments and intersections for use in the site-specific EB Method is presented in Part C, Appendix A.2.3. In applying the EB Method for urban and suburban arterials, whenever the predicted average crash frequency for a specific roadway segment during the multiyear study period is less than 1/k (the inverse of the overdispersion parameter for the relevant SPF), consideration should be given to combining adjacent roadway segments and applying the project-level EB Method. This guideline for the minimum crash frequency for a roadway segment applies only to Chapter 12 which uses fixed-value overdispersion parameters. It is not needed in Chapters 10 or 11, which use length-dependent overdispersion parameters.

SAFETY PERFORMANCE FUNCTIONS

In Step 9 of the predictive method, the appropriate safety performance functions (SPFs) are used to predict crash frequencies for specific base conditions. SPFs are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections. Each SPF in the predictive method was developed with observed crash data for a set of similar sites. The SPFs, like all regression models, estimate the value of a dependent variable as a function of a set of independent variables. In the SPFs developed for the HSM, the dependent variable estimated is the predicted average crash frequency for a roadway segment or intersection under base conditions, and the independent variables are the AADTs of the roadway segment or intersection legs (and, for roadway segments, the length of the roadway segment).

The predicted crash frequencies for base conditions obtained with the SPFs are used in the predictive models in Equations 12-2 through 12-7. A detailed discussion of SPFs and their use in the HSM is presented in Sections 3.5.2 and C.6.3.

Each SPF also has an associated overdispersion parameter, k. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically

reliable the SPF. This parameter is used in the EB Method discussed in Part C, Appendix A. The SPFs in Chapter 12 are summarized in Table 12-2.

Table 12-2. Safety Performance Functions included in Chapter 12

Chapter 12 SPFs for Urban and Suburban Arterials	SPF Components by Collision Type	SPF Equations, Tables, and Figures
Roadway segments	multiple-vehicle nondriveway collisions	Equations 12-10, 12-11, 12-12, Figure 12-3, Tables 12-3, 12-4
	single-vehicle crashes	Equations 12-13, 12-14, 12-15, Figure 12-4, Tables 12-5, 12-6
	multiple-vehicle driveway-related collisions	Equations 12-16, 12-17, 12-18, Figures 12-5, 12-6, 12-7, 12-8, 12-9, Table 12-7
	vehicle-pedestrian collisions	Equation 12-19, Table 12-8
	vehicle-bicycle collisions	Equation 12-20, Table 12-9
Signalized and Stop-Controlled Intersections	multiple-vehicle collisions	Equations 12-21, 12-22, 12-23, Figures 12-10, 12-11, 12-12, 12-13, Tables 12-10, 12-11
	single-vehicle crashes	Equations 12-24, 12-25, 12-26, 12-27, Figures 12-14, 12-15, 12-16, 12-17, Tables 12-12, 12-13
	vehicle-pedestrian collisions	Equations 12-28, 12-29, 12-30, Tables 12-14, 12-15, 12-16
	vehicle-bicycle collisions	Equation 12-31, Table 12-17
Roundabouts (Intersection Level)	all collisions (excluding vehicle-pedestrian and vehicle-bicycle)	????
Roundabouts (Leg kevel)	Entering-Circulating (all collisions and KABC collisions; Exiting-circulating; Single vehicle approach; single vehicle approach+circulating; rear-end approach; all collisions (excluding vehicle-pedestrian and vehicle-bicycle)	????

Some highway agencies may have performed statistically-sound studies to develop their own jurisdiction-specific SPFs derived from local conditions and crash experience. These models may be substituted for models presented in this chapter. Criteria for the development of SPFs for use in the predictive method are addressed in the calibration procedure presented in Part C, Appendix A.

Safety Performance Functions for Urban and Suburban Arterial Roadway Segments

The predictive model for predicting average crash frequency on a particular urban or suburban arterial roadway segment was presented in Equation 12-2. The effect of traffic volume (AADT) on crash frequency is incorporated through the SPF, while the effects of geometric design and traffic control features are incorporated through the CMFs. The SPF for urban and suburban arterial roadway segments is presented in this section. Urban and suburban arterial roadway segments are defined in Section 12.3.

SPFs and adjustment factors are provided for five types of roadway segments on urban and suburban arterials:

- Two-lane undivided arterials (2U)

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

Guidance on the estimation of traffic volumes for roadway segments for use in the SPFs is presented in Step 3 of the predictive method described in Section 12.4. The SPFs for roadway segments on urban and suburban arterials are applicable to the following AADT ranges:

- 2U: 0 to 32,600 vehicles per day
- 3T: 0 to 32,900 vehicles per day
- 4U: 0 to 40,100 vehicles per day
- 4D: 0 to 66,000 vehicles per day
- 5T: 0 to 53,800 vehicles per day

Application to sites with AADTs substantially outside these ranges may not provide reliable results.

Other types of roadway segments may be found on urban and suburban arterials but are not addressed by the predictive model in Chapter 12.

The procedure addresses five types of collisions. The corresponding equations, tables, and figures are indicated in Table 12-2 above:

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

The predictive model for estimating average crash frequency on roadway segments is shown in Equations 12-2 through 12-4. The effect of traffic volume on predicted crash frequency is incorporated through the SPFs, while the effects of geometric design and traffic control features are incorporated through the CMFs. SPFs are provided for multiple-vehicle nondriveway collisions and single-vehicle crashes. Adjustment factors are provided for multi-vehicle driveway-related, vehicle-pedestrian, and vehicle-bicycle collisions.

Multiple-Vehicle Nondriveway Collisions

The SPF for multiple-vehicle nondriveway collisions is applied as follows:

$$N_{brmv} = exp\left(a + b \times \ln\left(AADT\right) + \ln\left(L\right)\right) \quad (12\text{-}10)$$

Where:

$AADT$ = average annual daily traffic volume (vehicles/day) on roadway segment;

L = length of roadway segment (mi); and

a, b = regression coefficients.

Table 12-3 presents the values of the coefficients a and b used in applying Equation 12-10. The overdispersion parameter, k, is also presented in Table 12-3.

Table 12-3. SPF Coefficients for Multiple-Vehicle Nondriveway Collisions on Roadway Segments

	Coefficients Used in Equation 12-10		
Road Type	**Intercept (a)**	**AADT (b)**	**Overdispersion Parameter (k)**
Total crashes			
2U	−15.22	1.68	0.84
3T	−12.40	1.41	0.66
4U	−11.63	1.33	1.01
4D	−12.34	1.36	1.32
5T	−9.70	1.17	0.81
Fatal-and-injury crashes			
2U	−16.22	1.66	0.65
3T	−16.45	1.69	0.59
4U	−12.08	1.25	0.99
4D	−12.76	1.28	1.31
5T	−10.47	1.12	0.62
Property-damage-only crashes			
2U	−15.62	1.69	0.87
3T	−11.95	1.33	0.59
4U	−12.53	1.38	1.08
4D	−12.81	1.38	1.34
5T	−9.97	1.17	0.88

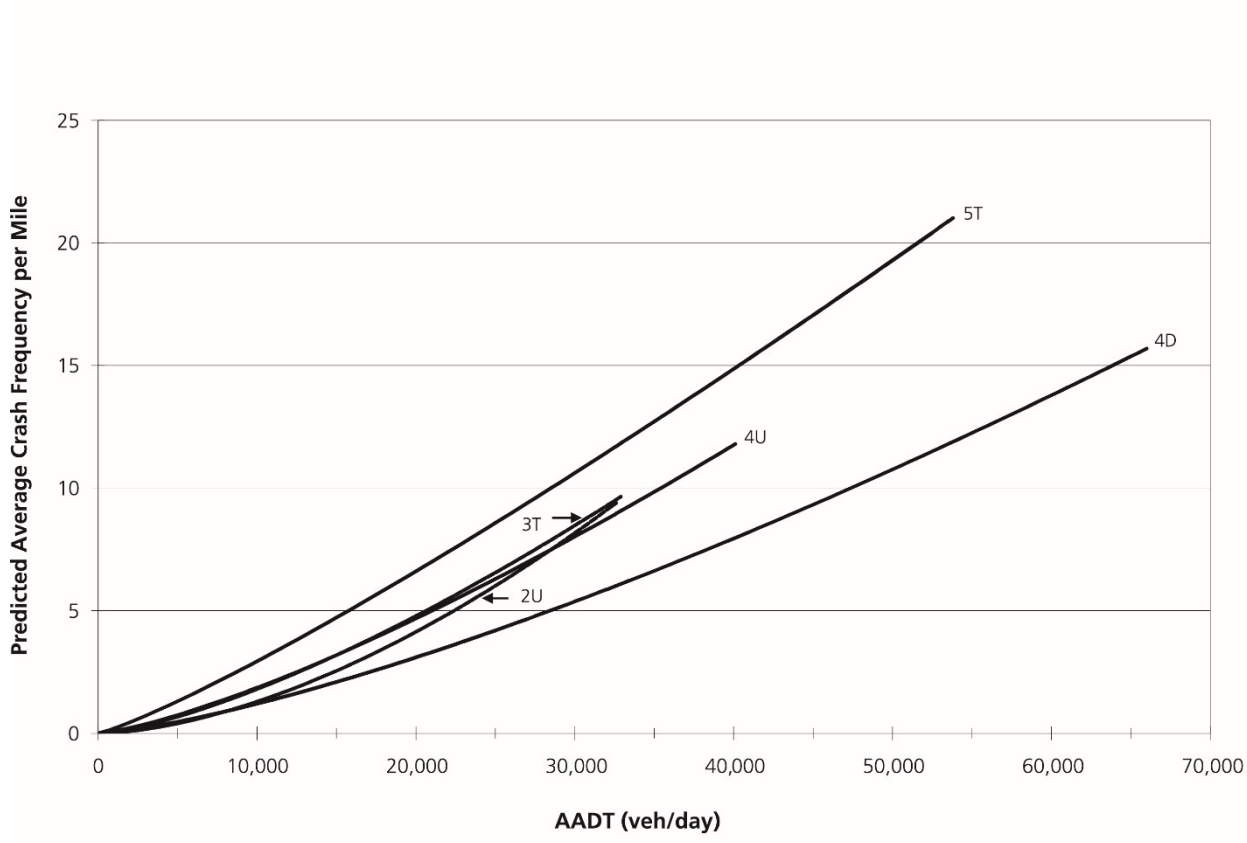


Figure 12-3. Graphical Form of the SPF for Multiple Vehicle Nondriveway collisions (from Equation 12-10 and Table 12-3)

Equation 12-10 is first applied to determine N_{brmv} using the coefficients for total crashes in Table 12-3. N_{brmv} is then divided into components by severity level, $N_{brmv(FI)}$ for fatal-and-injury crashes and $N_{brmv(PDO)}$ for property-damage-only crashes. These preliminary values of $N_{brmv(FI)}$ and $N_{brmv(PDO)}$, designated as $N'_{brmv(FI)}$ and $N'_{brmv(PDO)}$ in Equation 12-11, are determined with Equation 12-10 using the coefficients for fatal-and-injury and property-damage-only crashes, respectively, in Table 12-3. The following adjustments are then made to assure that $N_{brmv(FI)}$ and $N_{brmv(PDO)}$ sum to N_{brmv}:

$$N_{brmw(FI)} = N_{brmw(\text{total})} \left(\frac{N'_{brmv(FI)}}{N'_{brmv(FI)} + N'_{brmv(PDO)}} \right) \tag{12-11}$$

$$N_{brmv(PDO)} = N_{brmw(\text{total})} - N_{brmw(FI)} \tag{12-12}$$

The proportions in Table 12-4 are used to separate $N_{brmv(FI)}$ and $N_{brmv(PDO)}$ into components by collision type.

Table 12-4. Distribution of Multiple-Vehicle Nondriveway Collisions for Roadway Segments by Manner of Collision Type

	Proportion of Crashes by Severity Level for Specific Road Types									
	2U		3T		4U		4D		5T	
Collision Type	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**
Rear-end collision	0.730	0.778	0.845	0.842	0.511	0.506	0.832	0.662	0.846	0.651
Head-on collision	0.068	0.004	0.034	0.020	0.077	0.004	0.020	0.007	0.021	0.004
Angle collision	0.085	0.079	0.069	0.020	0.181	0.130	0.040	0.036	0.050	0.059
Sideswipe, same direction	0.015	0.031	0.001	0.078	0.093	0.249	0.050	0.223	0.061	0.248
Sideswipe, opposite direction	0.073	0.055	0.017	0.020	0.082	0.031	0.010	0.001	0.004	0.009
Other multiple-vehicle collisions	0.029	0.053	0.034	0.020	0.056	0.080	0.048	0.071	0.018	0.029

Source: HSIS data for Washington (2002–2006)

Single-Vehicle Crashes

SPFs for single-vehicle crashes for roadway segments are applied as follows:

$$N_{brsv} = exp\left(a + b \times \ln\left(AADT\right) + \ln\left(L\right)\right) \tag{12-13}$$

Table 12-5 presents the values of the coefficients and factors used in Equation 12-13 for each roadway type. Equation 12-13 is first applied to determine N_{brsv} using the coefficients for total crashes in Table 12-5. N_{brsv} is then divided into components by severity level; $N_{brsv(FI)}$ for fatal-and-injury crashes and $N_{brsv(PDO)}$ for property-damage-only crashes. Preliminary values of $N_{brsv(FI)}$ and $N_{brsv(PDO)}$, designated as $N'_{brsv(FI)}$ and $N'_{brsv(PDO)}$ in Equation 12-14, are determined with Equation 12-13 using the coefficients for fatal-and-injury and property-damage-only crashes, respectively, in Table 12-5. The following adjustments are then made to assure that $N_{brsv(FI)}$ and $N_{brsv(PDO)}$ sum to N_{brsv}:

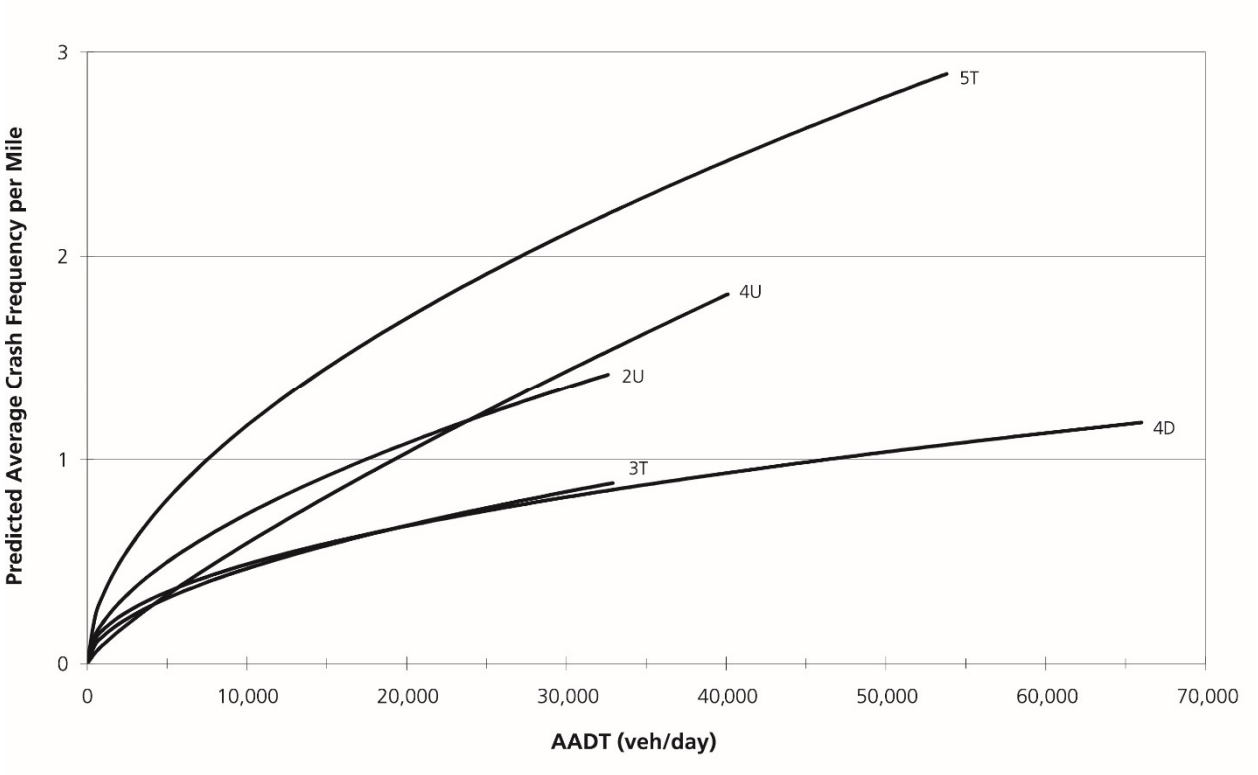
$$N_{brsv(FI)} = N_{brsv(\text{total})}\left(\frac{N'_{brsv(FI)}}{N'_{brsv(FI)} + N'_{brsv(PDO)}}\right) \tag{12-14}$$

$$N_{brsv(PDO)} = N_{brsv(\text{total})} - N_{brsv(FI)} \tag{12-15}$$

The proportions in Table 12-6 are used to separate $N_{brsv(FI)}$ and $N_{brsv(PDO)}$ into components by crash type.

Table 12-5. SPF Coefficients for Single-Vehicle Crashes on Roadway Segments

	Coefficients Used in Equation 12-11		
Road Type	**Intercept (a)**	**AADT (b)**	**Overdispersion Parameter (*k*)**
Total crashes			
2U	−5.47	0.56	0.81
3T	−5.74	0.54	1.37
4U	−7.99	0.81	0.91
4D	−5.05	0.47	0.86
5T	−4.82	0.54	0.52
Fatal-and-injury crashes			
2U	−3.96	0.23	0.50
3T	−6.37	0.47	1.06
4U	−7.37	0.61	0.54
4D	−8.71	0.66	0.28
5T	−4.43	0.35	0.36
Property-damage-only crashes			
2U	−6.51	0.64	0.87
3T	−6.29	0.56	1.93
4U	−8.50	0.84	0.97
4D	−5.04	0.45	1.06
5T	−5.83	0.61	0.55



- **Figure 12-4.** Graphical Form of the SPF for Single-Vehicle Crashes (from Equation 12-13 and Table 12-5)

Table 12-6. Distribution of Single-Vehicle Crashes for Roadway Segments by Collision Type

	Proportion of Crashes by Severity Level for Specific Road Types									
	2U		3T		4U		4D		5T	
Collision Type	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**
Collision with animal	0.026	0.066	0.001	0.001	0.001	0.001	0.001	0.063	0.016	0.049
Collision with fixed object	0.723	0.759	0.688	0.963	0.612	0.809	0.500	0.813	0.398	0.768
Collision with other object	0.010	0.013	0.001	0.001	0.020	0.029	0.028	0.016	0.005	0.061
Other single-vehicle collision	0.241	0.162	0.310	0.035	0.367	0.161	0.471	0.108	0.581	0.122

Source: HSIS data for Washington (2002–2006)

Multiple-Vehicle Driveway-Related Collisions

The model presented above for multiple-vehicle collisions addressed only collisions that are not related to driveways. Driveway-related collisions also generally involve multiple vehicles, but are addressed separately because the frequency of driveway-related collisions on a roadway segment depends on the number and type of driveways. Only unsignalized driveways are considered; signalized driveways are analyzed as signalized intersections.

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

$$N_{brdwy} = \sum_{\substack{\text{all} \\ \text{driveway} \\ \text{types}}} n_j \times N_j \times \left(\frac{AADT}{15,500} \right)^{(t)} \tag{12-16}$$

Where:

N_j = Number of driveway-related collisions per driveway per year for driveway type *j* from Table 12-7;

n_j = number of driveways within roadway segment of driveway type j including all driveways on both sides of the road; and

t = coefficient for traffic volume adjustment from Table 12-7.

The number of driveways of a specific type, n_j, is the sum of the number of driveways of that type for both sides of the road combined. The number of driveways is determined separately for each side of the road and then added together.

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

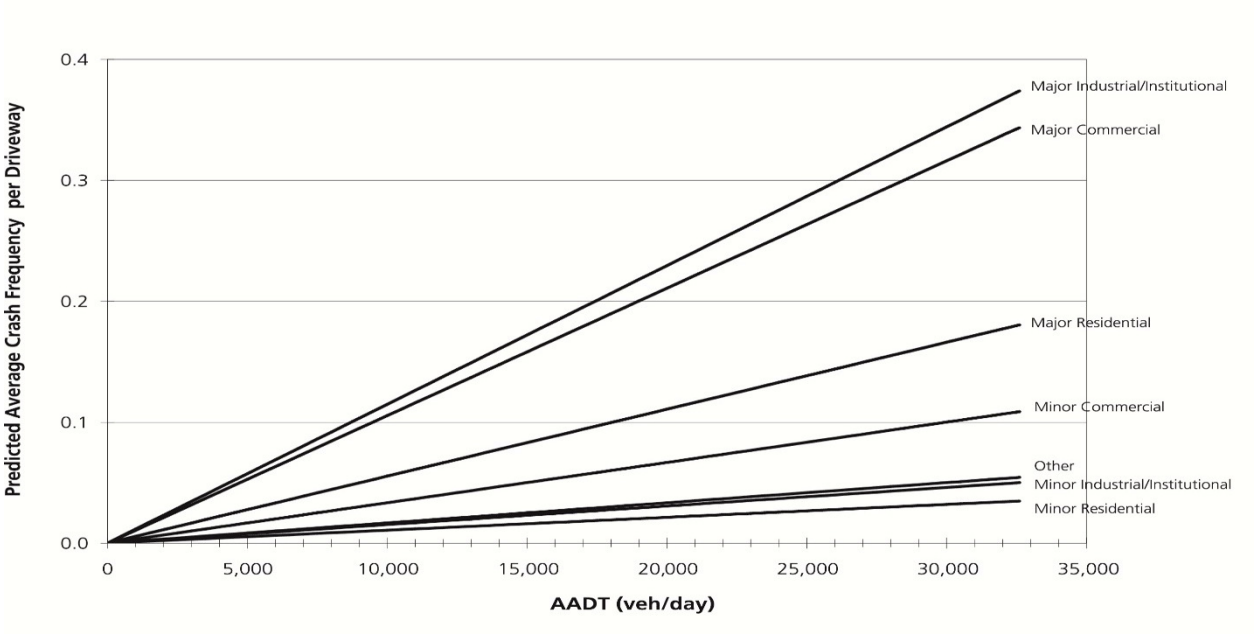
- Major commercial driveways
- Minor commercial driveways
- Major industrial/institutional driveways
- Minor industrial/institutional driveways
- Major residential driveways
- Minor residential driveways
- Other driveways

Major driveways are those that serve sites with 50 or more parking spaces. Minor driveways are those that serve sites with less than 50 parking spaces. It is not intended that an exact count of the number of parking spaces be made for each site. Driveways can be readily classified as major or minor from a quick review of aerial photographs that show parking areas or through user judgment based on the character of the establishment served by the driveway. Commercial driveways provide access to establishments that serve retail customers. Residential driveways serve single- and multiple-family dwellings. Industrial/institutional driveways serve factories, warehouses, schools, hospitals, churches, offices, public facilities, and other places of employment. Commercial sites with no restriction on access along an entire property frontage are generally counted as two driveways.

Table 12-7. SPF Coefficients for Multiple-Vehicle Driveway Related Collisions

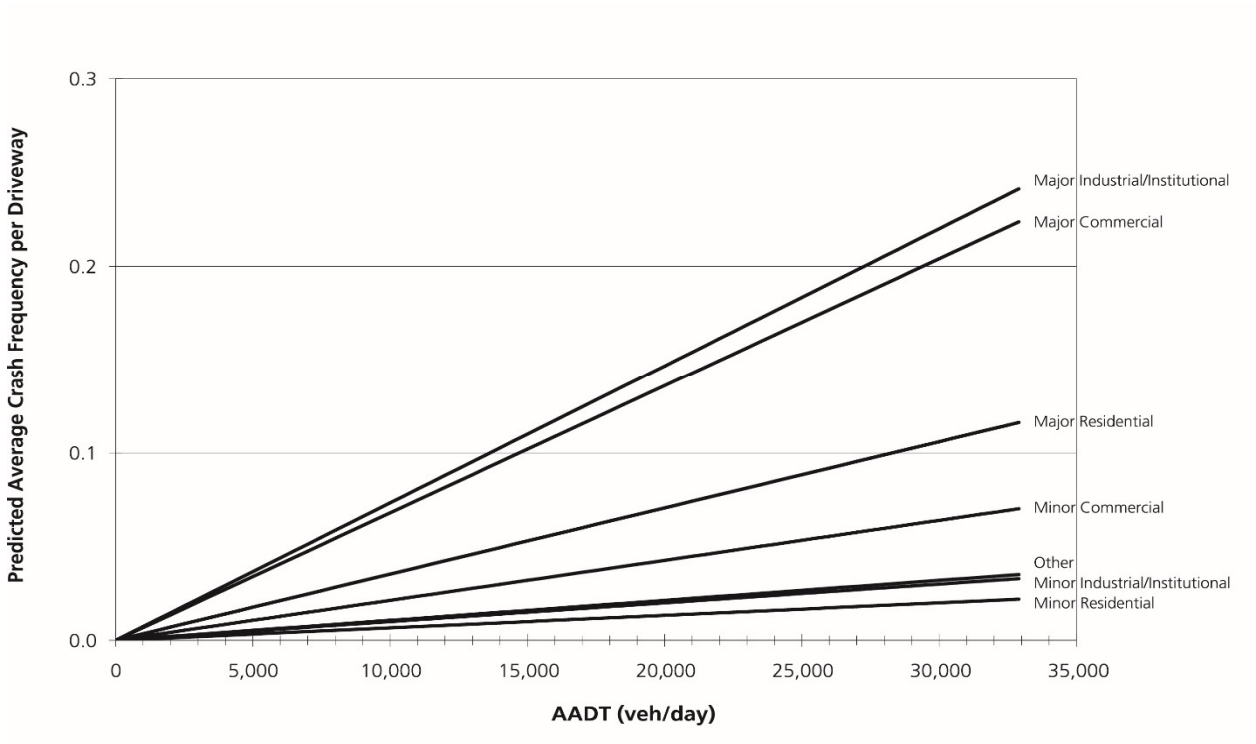
	Coefficients for Specific Roadway Types				
Driveway Type (*j*)	**2U**	**3T**	**4U**	**4D**	**5T**
Number of Driveway-Related Collisions per Driveway per Year (N_j)					
Major commercial	0.158	0.102	0.182	0.033	0.165
Minor commercial	0.050	0.032	0.058	0.011	0.053
Major industrial/institutional	0.172	0.110	0.198	0.036	0.181
Minor industrial/institutional	0.023	0.015	0.026	0.005	0.024
Major residential	0.083	0.053	0.096	0.018	0.087
Minor residential	0.016	0.010	0.018	0.003	0.016
Other	0.025	0.016	0.029	0.005	0.027
Regression Coefficient for AADT (*t*)					
All driveways	1.000	1.000	1.172	1.106	1.172
Overdispersion Parameter (*k*)					
All driveways	0.81	1.10	0.81	1.39	0.10
Proportion of Fatal-and-Injury Crashes (f_{dwy})					
All driveways	0.323	0.243	0.342	0.284	0.269
Proportion of Property-Damage-Only Crashes					
All driveways	0.677	0.757	0.658	0.716	0.731

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



▪ **Figure 12-5.** Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Two-Lane Undivided Arterials (2U) (from Equation 12-16 and Table 12-7)

▪



▪ **Figure 12-6.** Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Three-Lane Undivided Arterials (3T) (from Equation 12-16 and Table 12-7)

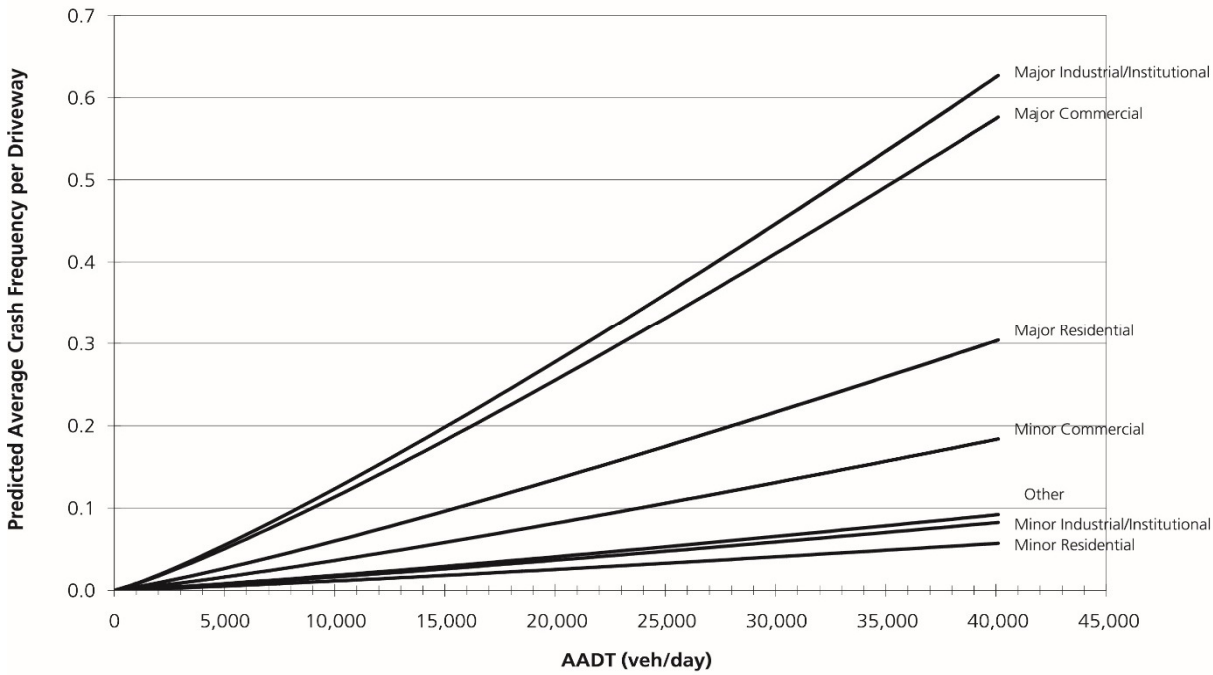
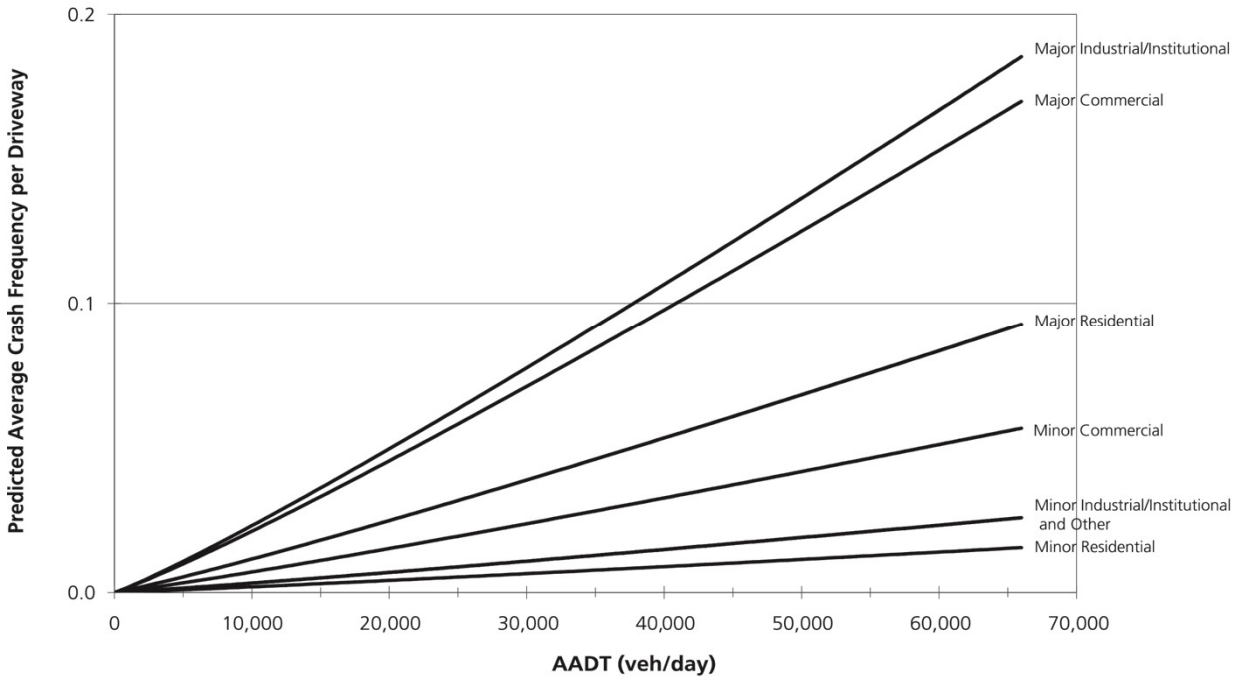
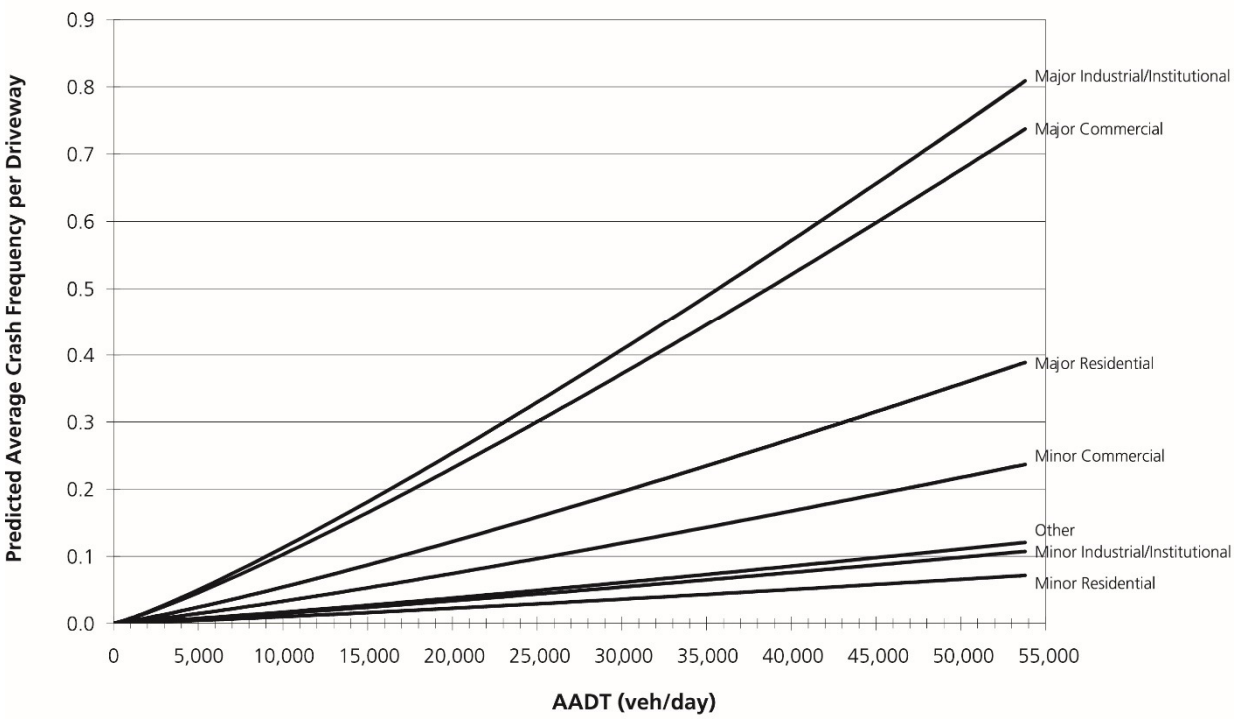


Figure 12-7. Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Four-Lane Undivided Arterials (4U) (from Equation 12-16 and Table 12-7)



▪ **Figure 12-8.** Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Four-Lane Divided Arterials (4D) (from Equation 12-16 and Table 12-7)

▪



▪ **Figure 12-9.** Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Five-Lane Arterials Including a Center Two-Way Left-Turn Lane (from Equation 12-16 and Table 12-7)

Driveway-related collisions can be separated into components by severity level as follows:

$$N_{brdwy(FI)} = N_{brdwy(\text{total})} \times f_{dwy} \tag{12-17}$$

$$N_{brdwy(PDO)} = N_{brdwy(\text{total})} - N_{brdwy(FI)} \tag{12-18}$$

Where:

f_{dwy} = proportion of driveway-related collisions that involve fatalities or injuries

The values of N_j and f_{dwy} are shown in Table 12-7.

Vehicle-Pedestrian Collisions

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

$$N_{pedr} = N_{br} \times f_{pedr} \tag{12-19}$$

Where:

f_{pedr} = pedestrian crash adjustment factor.

The value N_{br} used in Equation 12-19 is that determined with Equation 12-3.

Table 12-8 presents the values of f_{pedr} for use in Equation 12-19. All vehicle-pedestrian collisions are considered to be fatal-and-injury crashes. The values of f_{pedr} are likely to depend on the climate and the walking environment in particular states or communities. HSM users are encouraged to replace the values in Table 12-8 with suitable values for their own state or community through the calibration process (see Part C, Appendix A).

Table 12-8. Pedestrian Crash Adjustment Factor for Roadway Segments

	Pedestrian Crash Adjustment Factor (f_{pedr})	
Road Type	**Posted Speed 30 mph or Lower**	**Posted Speed Greater than 30 mph**
2U	0.036	0.005
3T	0.041	0.013
4U	0.022	0.009
4D	0.067	0.019
5T	0.030	0.023

Note: These factors apply to the methodology for predicting total crashes (all severity levels combined). All pedestrian collisions resulting from this adjustment factor are treated as fatal-and-injury crashes and none as property-damage-only crashes.
Source: HSIS data for Washington (2002–2006)

Vehicle-Bicycle Collisions

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

$$N_{biker} = N_{br} \times f_{biker} \tag{12-20}$$

Where:

f_{biker} = bicycle crash adjustment factor.

The value of N_{br} used in Equation 12-20 is determined with Equation 12-3.

Table 12-9 presents the values of fbiker for use in Equation 12-18. All vehicle-bicycle collisions are considered to be fatal-and-injury crashes. The values of fbiker are likely to depend on the climate and bicycling environment in particular states or communities. HSM users are encouraged to replace the values in Table 12-9 with suitable values for their own state or community through the calibration process (see Part C, Appendix A).

Table 12-9. Bicycle Crash Adjustment Factors for Roadway Segments

	Bicycle Crash Adjustment Factor (f_{biker})	
Road type	**Posted Speed 30 mph or Lower**	**Posted Speed Greater than 30 mph**
2U	0.018	0.004
3T	0.027	0.007
4U	0.011	0.002

4D	0.013	0.005
5T	0.050	0.012

Note: These factors apply to the methodology for predicting total crashes (all severity levels combined). All bicycle collisions resulting from this adjustment factor are treated as fatal-and-injury crashes and none as property-damage-only crashes.
Source: HSIS data for Washington (2002–2006)

Safety Performance Functions for Urban and Suburban Arterial Intersections

The predictive models for predicting the frequency of crashes related to an intersection is presented in Equations 12-5 through 12-7. The structure of the predictive models for intersections is similar to the predictive models for roadway segments.

The effect of traffic volume on predicted crash frequency for intersections is incorporated through SPFs, while the effect of geometric and traffic control features are incorporated through CMFs.

SPFs and adjustment factors have been developed for two types of signalized intersections, two types of stop-controlled intersections and four types of roundabouts, all on urban and suburban arterials. These are:

- Three-leg intersections with stop control on the minor-road approach (3ST)
- Three-leg signalized intersections (3SG)
- Four-leg intersections with stop control on the minor-road approaches (4ST)
- Four-leg signalized intersections (4SG)
- Three-leg roundabouts with one circulating lane (31R)
- Three-leg roundabouts with two circulating lanes (32R)
- Four-leg roundabouts with one circulating lane (41R)
- Four-leg roundabouts with two circulating lanes (42R)

Other types of intersections may be found on urban and suburban arterials but are not addressed by the Chapter 12 SPFs.

12.6.2.1 Signalized and Stop-Controlled Intersection Safety Performance Functions

The SPFs for each of the four intersection types identified above predict total crash frequency per year for crashes that occur within the limits of the intersection and intersection-related crashes. The SPFs and adjustment factors address the following four types of collisions, (the corresponding equations, tables, and figures are indicated in Table 12-2):

- multiple-vehicle collisions
- single-vehicle crashes
- vehicle-pedestrian collisions
- vehicle-bicycle collisions

Guidance on the estimation of traffic volumes for the major and minor road legs for use in the SPFs is presented in Step 3. The AADT(s) used in the SPF are the AADT(s) for the selected year of the evaluation period. The SPFs for intersections are applicable to the following AADT ranges:

3ST Intersections	$AADT_{maj}$: 0 to 45,700 vehicles per day and	$AADT_{min}$: 0 to 9,300 vehicles per day
4ST Intersections	$AADT_{maj}$: 0 to 46,800 vehicles per day and	$AADT_{min}$: 0 to 5,900 vehicles per day
3SG Intersections	$AADT_{maj}$: 0 to 58,100 vehicles per day and	$AADT_{min}$: 0 to 16,400 vehicles per day
4SG Intersections	$AADT_{maj}$: 0 to 67,700 vehicles per day and	$AADT_{min}$: 0 to 33,400 vehicles per day

4SG Intersections Pedestrian Models:

- $AADT_{maj}$: 80,200 vehicles per day
- $AADT_{min}$: 49,100 vehicles per day
- PedVol: 34,200 pedestrians per day crossing all four legs combined

Application to sites with AADTs substantially outside this range may not provide reliable results.

Multiple-Vehicle Collisions

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

$$N_{bimv} = exp\left(a + b \times \ln\left(AADT_{maj}\right) + c \times \ln\left(AADT_{min}\right)\right) \tag{12-21}$$

Where:

$AADT_{maj}$ = average daily traffic volume (vehicles/day) for major road (both directions of travel combined);

$AADT_{min}$ = average daily traffic volume (vehicles/day) for minor road (both directions of travel combined); and

a, b, c = regression coefficients.

Table 12-10 presents the values of the coefficients a, b, and c used in applying Equation 12-21. The SPF overdispersion parameter, k, is also presented in Table 12-10.

Equation 12-21 is first applied to determine N_{bimv} using the coefficients for total crashes in Table 12-10. N_{bimv} is then divided into components by crash severity level, $N_{bimv(FI)}$ for fatal-and-injury crashes and $N_{bimv(PDO)}$ for property-damage-only crashes. Preliminary values of $N_{bimv(FI)}$ and $N_{bimv(PDO)}$, designated as $N'_{bimv(FI)}$ and $N'_{bimv(PDO)}$ in Equation 12-22, are determined with Equation 12-21 using the coefficients for fatal-and-injury and property-damage-only crashes, respectively, in Table 12-10. The following adjustments are then made to assure that $N_{bimv(FI)}$ and $N_{bimv(PDO)}$ sum to N_{bimv}:

$$N_{bimv(FI)} = N_{bimv(\text{total})} \times \left(\frac{N'_{bimv(FI)}}{N'_{bimv(FI)} + N'_{bimv(PDO)}} \right) \tag{12-22}$$

$$N_{bimv(PDO)} = N_{bimv(\text{total})} - N_{bimv(FI)} \tag{12-23}$$

The proportions in Table 12-11 are used to separate $N_{bimv(FI)}$ and $N_{bimv(PDO)}$ into components by manner of collision.

Table 12-10. SPF Coefficients for Multiple-Vehicle Collisions at Intersections

	Coefficients Used in Equation 12-21			
Intersection Type	**Intercept (a)**	**$AADT_{maj}$ (b)**	**$AADT_{min}$ (c)**	**Overdispersion Parameter (k)**
Total Crashes				
3ST	−13.36	1.11	0.41	0.80
3SG	−12.13	1.11	0.26	0.33
4ST	−8.90	0.82	0.25	0.40
4SG	−10.99	1.07	0.23	0.39
Fatal-and-Injury Crashes				
3ST	−14.01	1.16	0.30	0.69
3SG	−11.58	1.02	0.17	0.30
4ST	−11.13	0.93	0.28	0.48
4SG	−13.14	1.18	0.22	0.33
Property-Damage-Only Crashes				
3ST	−15.38	1.20	0.51	0.77
3SG	−13.24	1.14	0.30	0.36
4ST	−8.74	0.77	0.23	0.40
4SG	−11.02	1.02	0.24	0.44

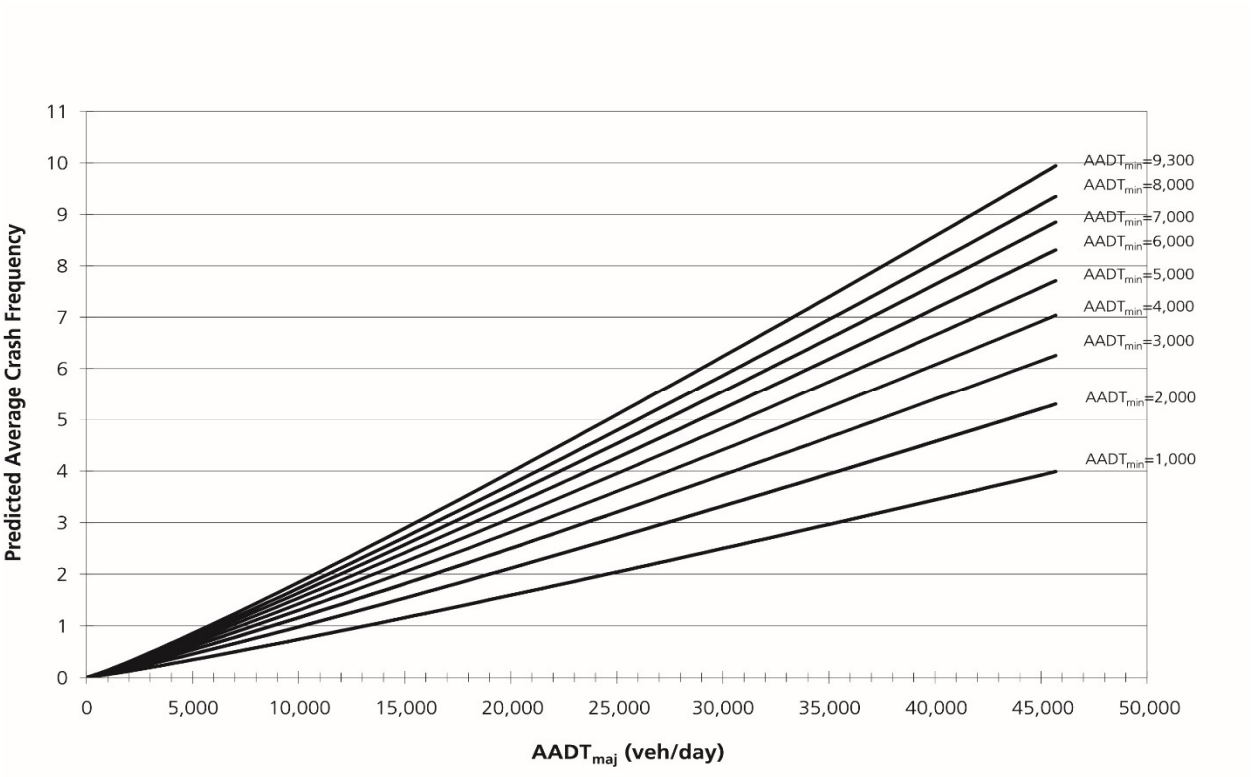
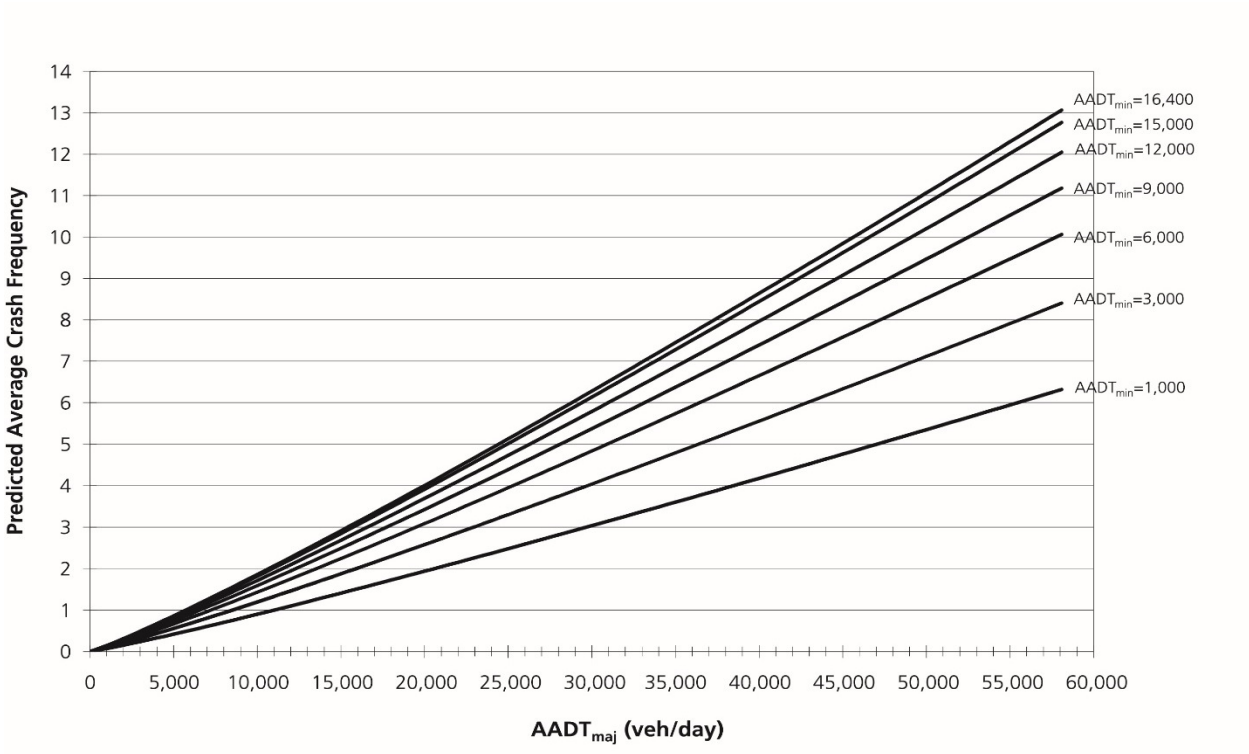
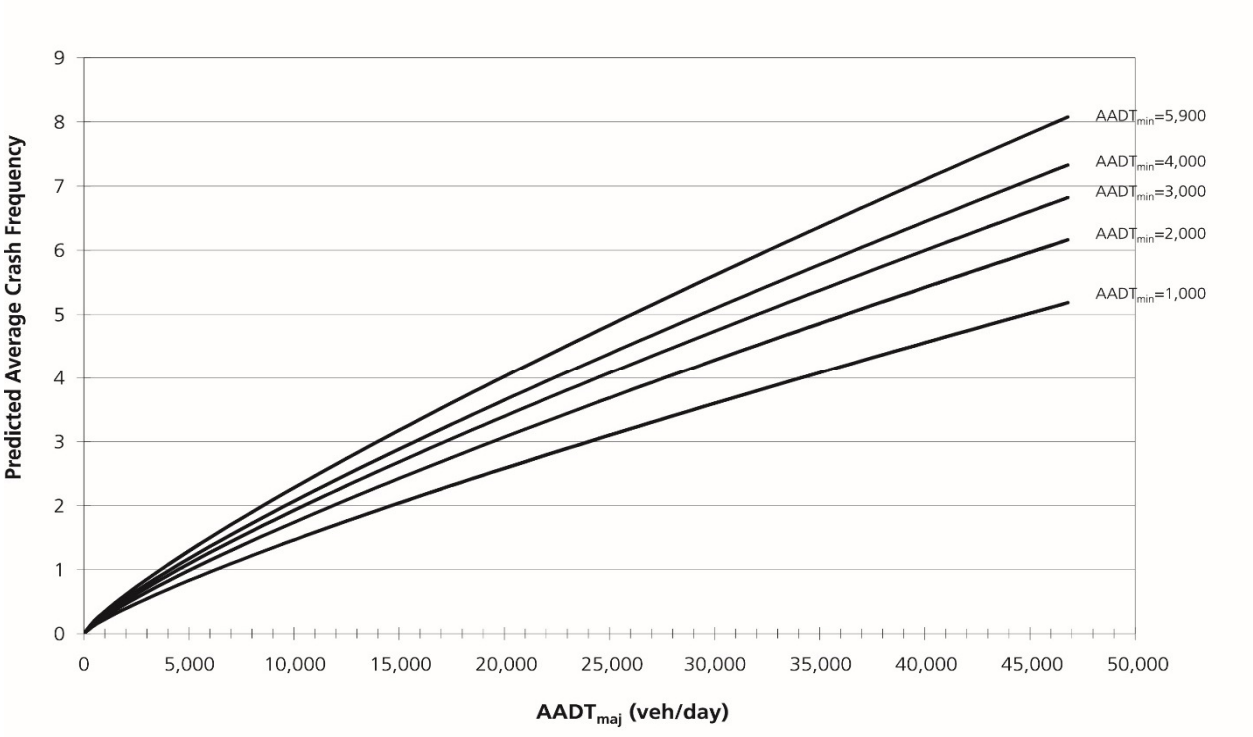


Figure 12-10. Graphical Form of the Intersection SPF for Multiple Vehicle Collisions on Three-Leg Intersections with Minor-Road Stop Control (3ST) (from Equation 12-21 and Table 12-10)



▪ **Figure 12-11.** Graphical Form of the Intersection SPF for Multiple Vehicle Collisions on Three-Leg Signalized Intersections (3SG) (from Equation 12-21 and Table 12-10)

▪



▪ **Figure 12-12.** Graphical Form of the Intersection SPF for Multiple Vehicle Collisions on Four-Leg Intersections with Minor-Road Stop Control (4ST) (from Equation 12-21 and Table 12-10)

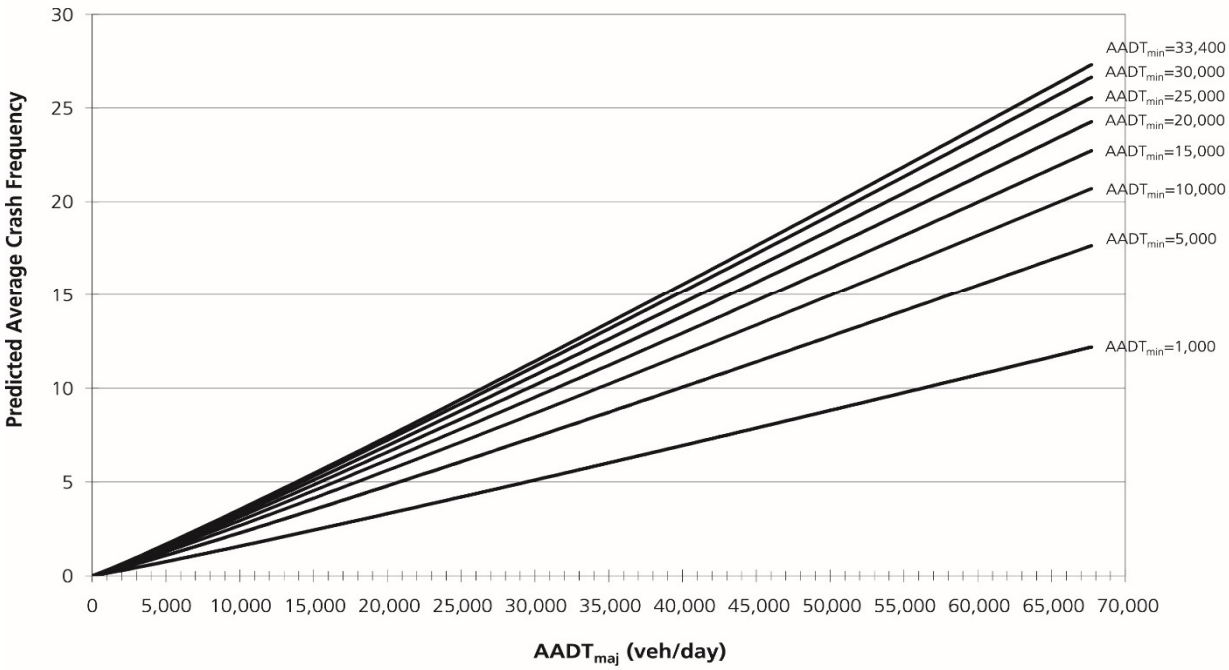


Figure 12-13. Graphical Form of the Intersection SPF for Multiple Vehicle Collisions on Four-Leg Signalized Intersections (4SG) (from Equation 12-21 and Table 12-10)

Table 12-11. Distribution of Multiple-Vehicle Collisions for Intersections by Collision Type

	Proportion of Crashes by Severity Level for Specific Intersections Types							
	3ST		3SG		4ST		4SG	
Manner of Collision	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**
Rear-end collision	0.421	0.440	0.549	0.546	0.338	0.374	0.450	0.483
Head-on collision	0.045	0.023	0.038	0.020	0.041	0.030	0.049	0.030
Angle collision	0.343	0.262	0.280	0.204	0.440	0.335	0.347	0.244
Sideswipe	0.126	0.040	0.076	0.032	0.121	0.044	0.099	0.032
Other multiple-vehicle collisions	0.065	0.235	0.057	0.198	0.060	0.217	0.055	0.211

Source: HSIS data for California (2002–2006)

Single-Vehicle Crashes

SPFs for single-vehicle crashes are applied as follows:

$$N_{bisv} = exp\left(a + b \times \ln\left(AADT_{maj}\right) + c \times \ln\left(AADT_{min}\right)\right) \quad (12\text{-}24)$$

Table 12-12 presents the values of the coefficients and factors used in Equation 12-24 for each roadway type. Equation 12-24 is first applied to determine N_{bisv} using the coefficients for total crashes in Table 12-12. N_{bisv} is then divided into components by severity level, $N_{bisv(FI)}$ for fatal-and-injury crashes and $N_{bisv(PDO)}$ for property-damage-only crashes. Preliminary values of $N_{bisv(FI)}$ and $N_{bisv(PDO)}$, designated as $N'_{bisv(FI)}$ and $N'_{bisv(PDO)}$ in Equation 12-25, are determined with Equation 12-24 using the coefficients for fatal-and-injury and property-damage-only crashes, respectively, in Table 12-12. The following adjustments are then made to assure that $N_{bisv(FI)}$ and $N_{bisv(PDO)}$ sum to N_{bisv}.

$$N_{bisv(FI)} = N_{bisv(\text{total})} \times \left(\frac{N'_{bisv(FI)}}{N'_{bisv(FI)} + N'_{bisv(PDO)}} \right) \tag{12-25}$$

$$N_{bisv(PDO)} = N_{bisv(\text{total})} - N_{bisv(FI)} \tag{12-26}$$

Table 12-12. SPF Coefficients for Single-Vehicle Crashes at Intersections

	Coefficients Used in Equation 12-24			
Intersection Type	**Intercept (a)**	**AADT$_{maj}$ (b)**	**AADT$_{min}$ (c)**	**Overdispersion Parameter (k)**
Total Crashes				
3ST	−6.81	0.16	0.51	1.14
3SG	−9.02	0.42	0.40	0.36
4ST	−5.33	0.33	0.12	0.65
4SG	−10.21	0.68	0.27	0.36
Fatal-and-Injury Crashes				
3ST				
3SG	−9.75	0.27	0.51	0.24
4ST				
4SG	−9.25	0.43	0.29	0.09
Property-Damage-Only Crashes				
3ST	−8.36	0.25	0.55	1.29
3SG	−9.08	0.45	0.33	0.53
4ST	−7.04	0.36	0.25	0.54
4SG	−11.34	0.78	0.25	0.44

Note: Where no models are available, Equation 12-27 is used.

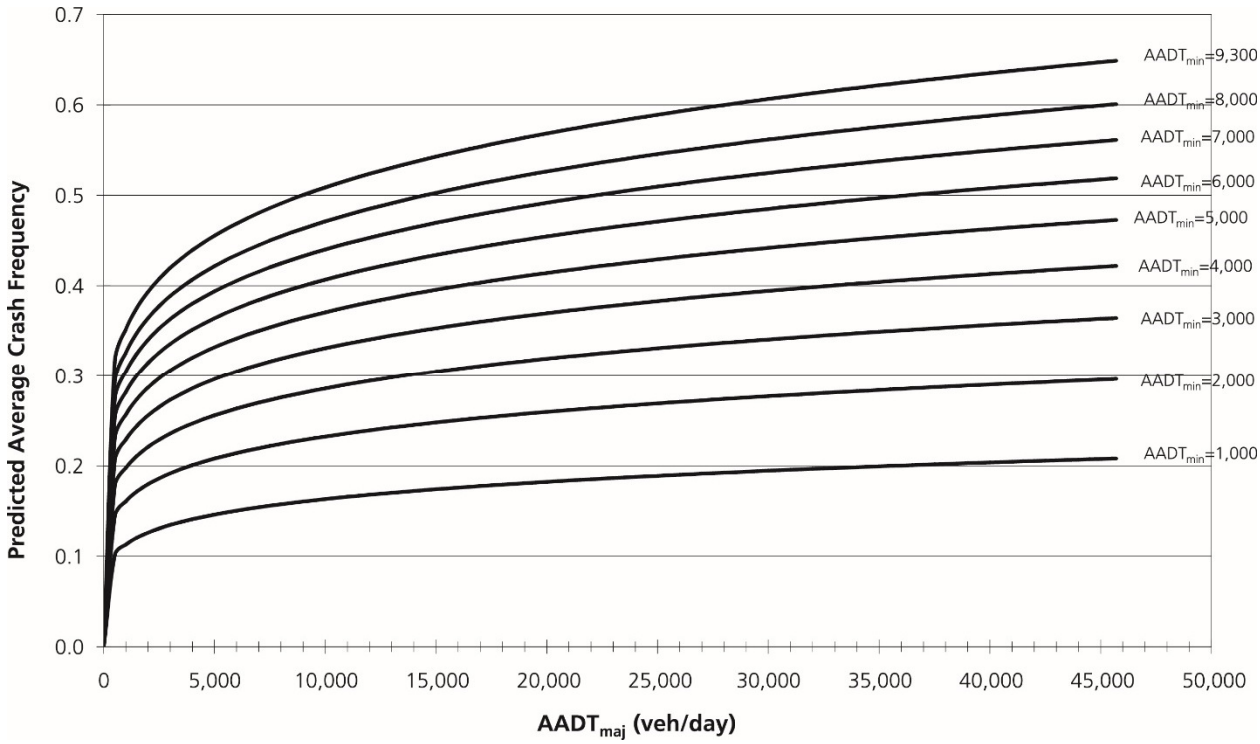


Figure 12-14. Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Three-Leg Intersections with Minor-Road Stop Control (3ST) (from Equation 12-24 and Table 12-12)

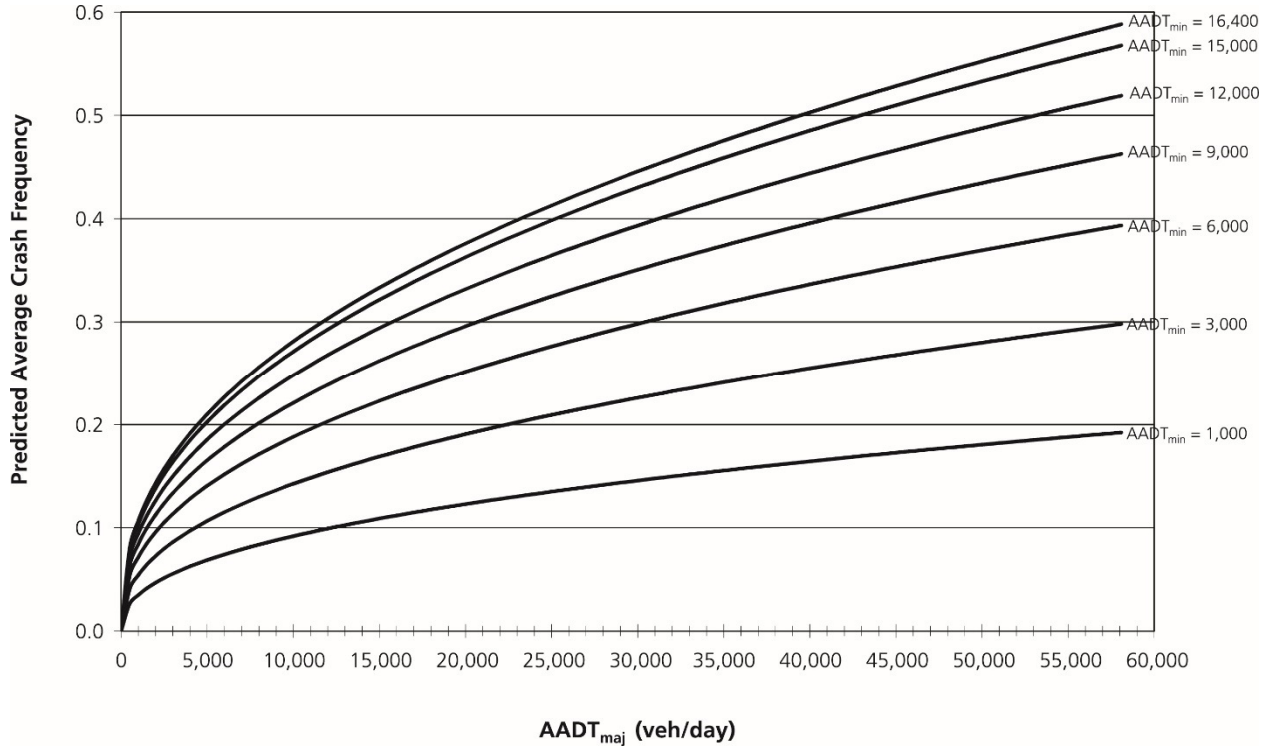


Figure 12-15. Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Three-Leg Signalized Intersections (3SG) (from Equation 12-24 and Table 12-12)

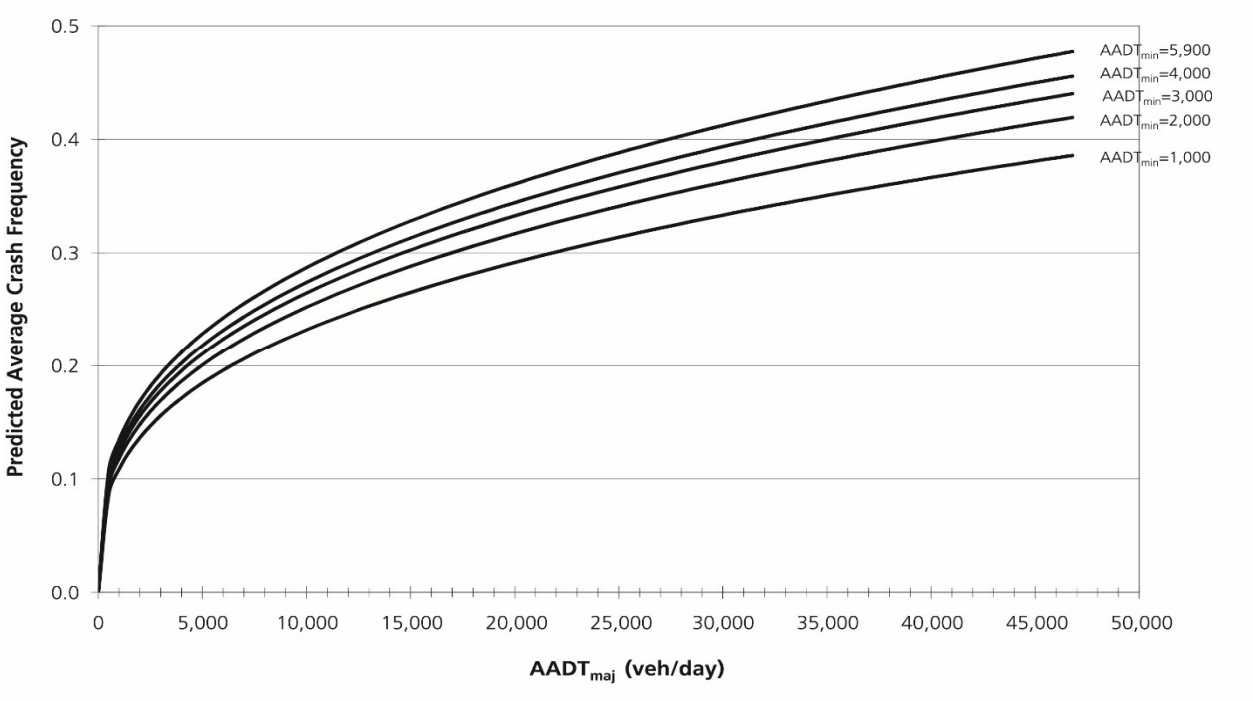
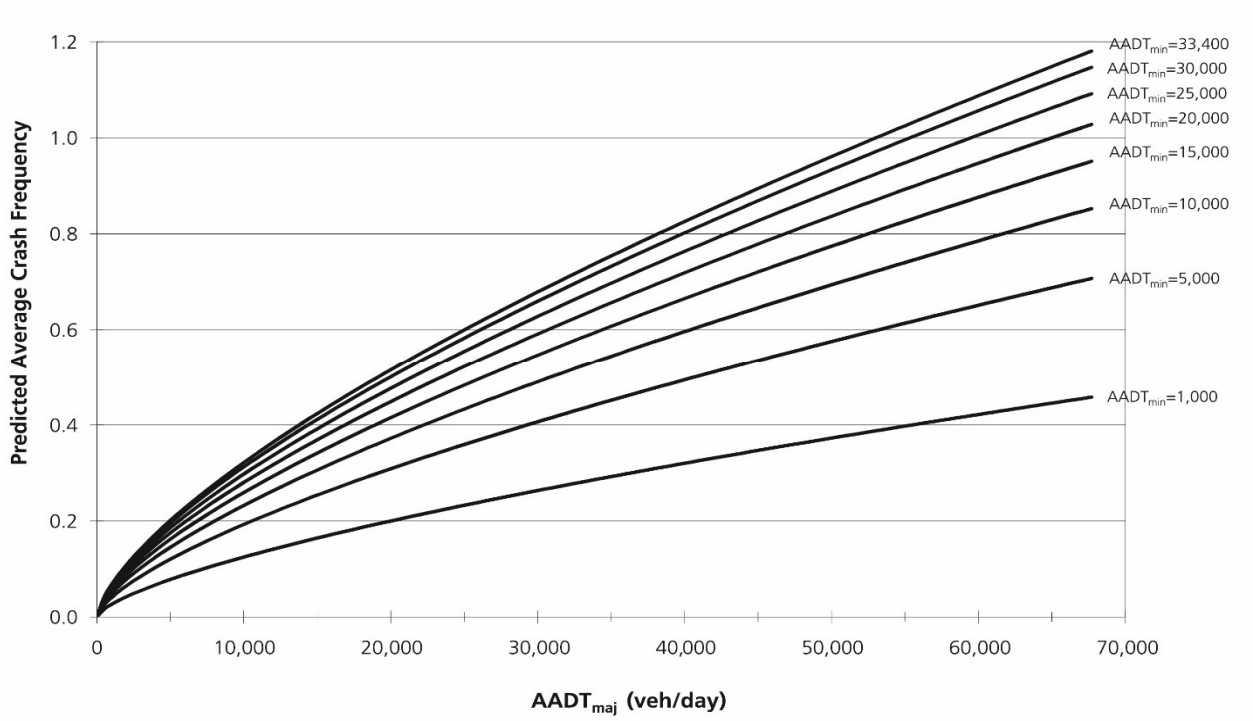


Figure 12-16. Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Four-Leg Stop Controlled Intersections (4ST) (from Equation 12-24 and Table 12-12)



▪ **Figure 12-17.** Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Four-Leg Signalized Intersections (4SG) (from Equation 12-24 and Table 12-12)

The proportions in Table 12-13 are used to separate $N_{bisv(FI)}$ and $N_{bisv(PDO)}$ into components by crash type.

Table 12-13. Distribution of Single-Vehicle Crashes for Intersection by Collision Type

	Proportion of Crashes by Severity Level for Specific Road Types							
	3ST		3SG		4ST		4SG	
Crash Type	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**	**FI**	**PDO**
Collision with parked vehicle	0.001	0.003	0.001	0.001	0.001	0.001	0.001	0.001
Collision with animal	0.003	0.018	0.001	0.003	0.001	0.026	0.002	0.002
Collision with fixed object	0.762	0.834	0.653	0.895	0.679	0.847	0.744	0.870
Collision with other object	0.090	0.092	0.091	0.069	0.089	0.070	0.072	0.070
Other single-vehicle collision	0.039	0.023	0.045	0.018	0.051	0.007	0.040	0.023
Noncollision	0.105	0.030	0.209	0.014	0.179	0.049	0.141	0.034

Source: HSIS data for California (2002–2006)

Since there are no models for fatal-and-injury crashes at three- and four-leg stop-controlled intersections in Table 12-12, Equation 12-25 is replaced with the following equation in these cases:

$$N_{bisv(FI)} = N_{bisv(\text{total})} \times f_{bisv} \tag{12-27}$$

Where:

f_{bisv} = proportion of fatal-and-injury crashes for combined sites.

The default value of f_{bisv} in Equation 12-27 is 0.31 for 3ST and 0.28 for 4ST intersections. It is recommended that these default values be updated based on locally available data.

SPFs for Vehicle-Pedestrian Collisions at Signalized Intersections

The number of vehicle-pedestrian collisions per year at a signalized intersection is estimated with a SPF and a set of CMFs that apply specifically to vehicle-pedestrian collisions. The model for estimating vehicle-pedestrian collisions at signalized intersections is:

$$N_{pedi} = N_{pedbase} \times CMF_{1p} \times CMF_{2p} \times CMF_{3p} \tag{12-28}$$

Where:

$N_{pedbase}$ = predicted number of vehicle-pedestrian collisions per year for base conditions at signalized intersections; and

$CMF_{1p}...CMF_{3p}$ = crash modification factors for vehicle-pedestrian collisions at signalized intersections.

The SPF for vehicle-pedestrian collisions at signalized intersections is:

$$N_{pedbase} = \exp\left(a + b \times \ln\left(AADT_{\text{total}}\right) + c \times \ln\left(\frac{AADT_{min}}{AADT_{maj}}\right) + d \times \ln\left(PedVol\right) + e \times n_{lanesx} \right) \quad (12\text{-}29)$$

Where:

$AADT_{\text{total}}$ = sum of the average daily traffic volumes (vehicles per day) for the major and minor roads (= $AADT_{maj} + AADT_{min}$);

$PedVol$ = sum of daily pedestrian volumes (pedestrians/day) crossing all intersection legs;

n_{lanesx} = maximum number of traffic lanes crossed by a pedestrian in any crossing maneuver at the intersection considering the presence of refuge islands; and

a, b, c, d, e = regression coefficients.

Determination of values for $AADT_{maj}$ and $AADT_{min}$ is addressed in the discussion of Step 3. Only pedestrian crossing maneuvers immediately adjacent to the intersection (e.g., at a marked crosswalk or along the extended path of any sidewalk present) are considered in determining the pedestrian volumes. Table 12-14 presents the values of the coefficients a, b, c, d, and e used in applying Equation 12-29.

The coefficient values in Table 12-14 are intended for estimating total vehicle-pedestrian collisions. All vehicle-pedestrian collisions are considered to be fatal-and-injury crashes.

The application of Equation 12-29 requires data on the total pedestrian volumes crossing the intersection legs. Reliable estimates will be obtained when the value of PedVol in Equation 12-29 is based on actual pedestrian volume counts. Where pedestrian volume counts are not available, they may be estimated using Table 12-15. Replacing the values in Table 12-15 with locally derived values is encouraged.

The value of n_{lanesx} in Equation 12-29 represents the maximum number of traffic lanes that a pedestrian must cross in any crossing maneuver at the intersection. Both through and turning lanes that are crossed by a pedestrian along the crossing path are considered. If the crossing path is broken by an island that provides a suitable refuge for the pedestrian so that the crossing may be accomplished in two (or more) stages, then the number of lanes crossed in each stage is considered separately. To be considered as a suitable refuge, an island must be raised or depressed; a flush or painted island is not treated as a refuge for purposes of determining the value of nlanesx.

Table 12-14. SPFs for Vehicle-Pedestrian Collisions at Signalized Intersections

	Coefficients used in Equation 12-29					
Intersection Type	**Intercept (a)**	**AADTtotal (b)**	**$AADT_{min}/AADT_{maj}$ (c)**	**PedVol (d)**	**n_{lanesx} (e)**	**Overdispersion Parameter (k)**
Total crashes						
3SG	–6.60	0.05	0.24	0.41	0.09	0.52
4SG	–9.53	0.40	0.26	0.45	0.04	0.24

Table 12-15. Estimates of Pedestrian Crossing Volumes Based on General Level of Pedestrian Activity

General Level of Pedestrian Activity	Estimate of PedVol (pedestrians/day) for Use in Equation 12-29

	3SG Intersections	4SG Intersections
High	1,700	3,200
Medium-high	750	1,500
Medium	400	700
Medium-low	120	240
Low	20	50

SPFs for Vehicle-Pedestrian Collisions Stop-Controlled Intersections

The number of vehicle-pedestrian collisions per year for a stop-controlled intersection is estimated as:

$$N_{pedi} = N_{bi} \times f_{pedi} \tag{12-30}$$

Where:

f_{pedi} = pedestrian crash adjustment factor.

The value of N_{bi} used in Equation 12-30 is that determined with Equation 12-6.

Table 12-16 presents the values of fpedi for use in Equation 12-30. All vehicle-pedestrian collisions are considered to be fatal-and-injury crashes. The values of f_{pedi} are likely to depend on the climate and walking environment in particular states or communities. HSM users are encouraged to replace the values in Table 12-16 with suitable values for their own state or community through the calibration process (see Part C, Appendix A).

Table 12-16. Pedestrian Crash Adjustment Factors for Stop-Controlled Intersections

Intersection Type	Pedestrian Crash Adjustment Factor (f_{pedi})
3ST	0.021
4ST	0.022

Note: These factors apply to the methodology for predicting total crashes (all severity levels combined). All pedestrian collisions resulting from this adjustment factor are treated as fatal-and-injury crashes and none as property-damage-only crashes.
Source: HSIS data for California (2002–2006)

Vehicle-Bicycle Collisions

The number of vehicle-bicycle collisions per year for a signalized or stop-controlled intersection is estimated as:

$$N_{bikei} = N_{bi} \times f_{bikei} \tag{12-31}$$

Where:

f_{bikei} = bicycle crash adjustment factor.

The value of N_{bi} used in Equation 12-31 is determined with Equation 12-6.

Table 12-17 presents the values of f_{bikei} for use in Equation 12-31. All vehicle-bicycle collisions are considered to be fatal-and-injury crashes. The values of f_{bikei} are likely to depend on the climate and bicycling environment in particular states or communities. HSM users are encouraged to replace the values in Table 12-17 with suitable values for their own state or community through the calibration process (see Part C, Appendix A).

Table 12-17. Bicycle Crash Adjustment Factors for Intersections

Intersection Type	Bicycle Crash Adjustment Factor (f_{bikei})
3ST	0.016
3SG	0.011
4ST	0.018
4SG	0.015

Note: These factors apply to the methodology for predicting total crashes (all severity levels combined). All bicycle collisions resulting from this adjustment factor are treated as fatal-and-injury crashes and none as property-damage-only crashes. Source: HSIS data for California (2002–2006)

12.6.2.2 Roundabout Safety Performance Functions

The intersection-level SPFs for each of the four roundabout types predict total crash frequency per year for crashes that occur within the limits of the intersection and intersection-related crashes. Separate SPFs are provided for fatal+injury and PDO crash severities.

The SPFs for intersections are applicable to the following AADT ranges (Table 12-?):

Table 12-?: Applicable AADT Ranges for Intersection-Level SPFs

3 leg 1 circulating lane	$AADT_{maj}$: 675 to 17,369 vehicles per day and	$AADT_{min}$: 409 to 14,000 vehicles per day
3 leg 2 circulating lanes	$AADT_{maj}$: 1,000 to 22,050 vehicles per day and	$AADT_{min}$: 476 to 15,108 vehicles per day
4 leg 1 circulating lane	$AADT_{maj}$: 1,000 to 19,733 vehicles per day and	$AADT_{min}$: 280 to 11,239 vehicles per day
4 leg 2 circulating lanes	$AADT_{maj}$: 1,000 to 28,333 vehicles per day and	$AADT_{min}$: 390 to 19,371 vehicles per day

The SPF model form is shown in Equation 12-?. The equation is used to compute the predicted crash frequency for base conditions. The SPF coefficients for this Equation are shown in Table 12-?.

Equation 12-?

$$N_{SPF,m} = exp[\mathrm{a} + \mathrm{b} \times LN(EntAADT\ /1000) + \mathrm{c} \times I_{rural}]$$

- where
- $N_{SPF,m}$ = predicted average crash frequency for base conditions on all legs, crashes/yr;
- $EntAADT$ = entering AADT for roundabout, veh/d; and
- I_{rural} = area type indicator variable (= 1.0 if area is rural, 0.0 otherwise).

The entering AADT for the roundabout represents the sum of the annual average daily volume (AADT) that enters the roundabout, when considering all legs. If a leg serves two-way traffic, then its entering volume can be estimated by multiplying a representative directional distribution factor by the leg AADT. If this factor is unknown, then a

default value of 0.5 can be used. If a leg serves one-way traffic inbound to the roundabout, then the leg's AADT can be included directly in the summation. If a leg serves one-way traffic outbound from the roundabout, then the leg's AADT should not be included in the summation.

Table 12-?. SPF Coefficients for Intersection-Level Roundabout Models

	Coefficients Used in Equation 12-?			
Intersection Type	**Intercept (a)**	**EntAADT/1000 (b)**	**I_{rural} (c)**	**Overdispersion Parameter (k)**
Fatal+Injury Crashes				
3 leg 1 circulating lane	-4.404	1.084	0.206	0.31
3 leg 2 circulating lanes	-3.887	1.306	0.250	0.36
4 leg 1 circulating lane	-3.503	0.915	0.206	0.33
4 leg 2 circulating lanes	-3.535	1.276	0.250	0.45
Property-Damage-Only Crashes				
3 leg 1 circulating lane	-1.720	0.486	0.168	0.54
3 leg 2 circulating lanes	-1.565	1.055	0.496	1.06
4 leg 1 circulating lane	-1.475	0.702	0.168	0.80
4 leg 2 circulating lanes	-1.536	1.131	0.496	0.79

The proportions in Tables 12-? to 12-? are used to separate $N_{SPF,m}$ into components by crash type.

- **Table 12-?. Fatal-and-injury crash type distribution – one circulating lane.**

	Multiple-Vehicle Crash Type					Single-Vehicle Crash Type				
Number of Legs	**Head On**	**Right Angle**	**Rear End**	**Side-swipe, Same Dir.**	**Other**	**Animal**	**Fixed Object**	**Other Object**	**Parked Vehicle**	**Other**
3	0.007	0.168	0.356	0.045	0.139	0.000	0.109	0.000	0.000	0.175
4	0.011	0.115	0.298	0.078	0.071	0.000	0.216	0.000	0.002	0.209

- **Table 12-?. Fatal-and-injury crash type distribution – two circulating lanes.**

	Multiple-Vehicle Crash Type					Single-Vehicle Crash Type				
Number of Legs	**Head On**	**Right Angle**	**Rear End**	**Side-swipe, Same Dir.**	**Other**	**Animal**	**Fixed Object**	**Other Object**	**Parked Vehicle**	**Other**
3	0.000	0.072	0.137	0.109	0.124	0.000	0.325	0.000	0.000	0.233
4	0.008	0.142	0.268	0.177	0.152	0.000	0.127	0.000	0.000	0.126

- **Table 6-10. Property-damage-only crash type distribution – one circulating lane.**

Number of Legs	Area Type	Multiple-Vehicle Crash Type					Single-Vehicle Crash Type				
		Head On	Right Angle	Rear End	Side-swipe, Same Dir.	Other	Animal	Fixed Object	Other Object	Parked Vehicle	Other
3	R	0.000	0.070	0.411	0.099	0.151	0.017	0.183	0.000	0.000	0.069
	U	0.008	0.121	0.226	0.053	0.241	0.008	0.225	0.002	0.000	0.117
4	R	0.004	0.149	0.248	0.136	0.070	0.014	0.261	0.000	0.003	0.116
	U	0.010	0.192	0.263	0.093	0.187	0.002	0.188	0.002	0.009	0.054

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Note: Area type: R = rural; U = urban or suburban.

- **Table 6-11. Property-damage-only crash type distribution – two circulating lanes.**

Number of Legs	Area Type	Multiple-Vehicle Crash Type					Single-Vehicle Crash Type				
		Head On	Right Angle	Rear End	Side-swipe, Same Dir.	Other	Animal	Fixed Object	Other Object	Parked Vehicle	Other
3	R	0.000	0.147	0.215	0.131	0.262	0.000	0.186	0.000	0.000	0.060
	U	0.002	0.072	0.227	0.256	0.131	0.005	0.178	0.000	0.000	0.128
4	R	0.025	0.164	0.216	0.230	0.258	0.005	0.076	0.001	0.000	0.025
	U	0.005	0.174	0.178	0.265	0.199	0.003	0.138	0.002	0.000	0.037

CHAPTER 13.

Note: Area type: R = rural; U = urban or suburban.

CHAPTER 14.

Prediction of Vehicle-Pedestrian and Vehicle-Bicycle Collisions at Roundabouts

SPFs for the prediction of vehicle-pedestrian and vehicle-bicycle crashes at roundabouts are not available in the HSM. Developing SPFs specific to predicting vehicle-pedestrian and vehicle-bicycle crashes is difficult because reported pedestrian and bicycle crashes are rare events at roundabouts.

In the data used to develop the intersection level roundabout SPFs, bicycle crashes account for approximately 1% of all reported crashes in the project database. Pedestrian crashes represent about 0.4% of all reported crashes in the database. The breakdown of these crashes by severity is shown in Table 12-?

- **Table 12-? Bicycle and Pedestrian Crashes by Severity**

	Crashes by Severity					
Crash Type	Fatal	Injury A	Injury B	Injury C	Injury (ABC)[1]	PDO

	Crashes by Severity					
Crash Type	Fatal	Injury A	Injury B	Injury C	Injury (ABC)[1]	PDO
Bicycle	0	6	36	21	3	8
Pedestrian	0	2	7	8	7	1

[1]Severity of injury unknown

Prediction by Crash Severity Using Severity Distribution Functions

An optional step for the intersection-level design models for roundabouts is to predict K, A, B or C severity level crashes using severity distribution functions applied to the SPF predictions for fatal+injury crashes. The predicted crash frequency is multiplied by the predicted proportion of crashes of a specific severity to obtain an estimate of the average frequency of crashes associated with the specified severity. This model can be used to predict the proportion of K (fatal), A (incapacitating injury), B (non-incapacitating injury), and C (possible injury) crashes. Then, these proportions would be used with the predicted FI crash frequency to estimate the frequency of K, A, B, and C crashes.

The prediction is accomplished in three steps:

Step 1 – Apply Severity Adjustment Factor. The calibrated adjustment factor is used to adjust the predicted severity distribution as a function of speed limit. It is a leg-specific factor. The speed limit adjustment factor is computed using the following equation. One factor value is computed for each roundabout leg.

- **Equation 12-?**

$$f_{j,sl} = exp\left[3.1187 \times \left\{\left(SL_j/100\right)^2 - (35/100)^2\right\}\right]$$

- where
- $f_{j,sl}$ = severity adjustment factor for the effect of speed limit on leg j (j = 1 to m);
- SL_j = speed limit on leg j (j = 1 to m) in mi/h.

This adjustment factor is applicable to roundabouts in urban, suburban, and rural areas. The leg speed limit data used to calibrate this factor ranged from 10 to 60 mi/h. This factor should not be used for speed limits outside of this range.

Step 2 – Aggregate Leg-Specific Adjustment Factor. The leg-specific adjustment factors need to have their effect aggregated to the overall intersection level. The aggregated adjustment factors for three- and four-leg roundabouts are computed using the following equations.

<u>Aggregated CMF for three-leg roundabouts (m = 3).</u>

$$S_K = \exp[F_K] \times [\, p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl}]$$

$$S_A = \exp[F_A] \times [p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl}]$$

$$S_B = \exp[F_B] \times [p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl}]$$

- **Equation 12-?**

$$\begin{aligned} S_K &= \exp[-3.4725] \times \left[p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl}\right] S_A \\ &= \exp[-1.1752] \times \left[p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl}\right] S_B \\ &= \exp[-0.0415] \times \left[p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl}\right] \end{aligned}$$

The proportion of total leg volume p_j is computed using Equation?

- <u>Aggregated CMF for four-leg roundabouts (m = 4).</u>
-

$$S_K = \exp[F_K] \times [p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl} + p_4 \times f_{4,sl}]$$

$$S_A = \exp[F_A] \times [p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl} + p_4 \times f_{4,sl}]$$

$$S_B = \exp[F_B] \times [p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl} + p_4 \times f_{4,sl}]$$

$$\begin{aligned} S_K &= \exp[F_K] \times \left[p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl} + p_4 \times f_{4,sl}\right] S_K \\ &= \exp[F_K] \times \left[p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl} + p_4 \times f_{4,sl}\right] S_K \\ &= \exp[-4.6216] \times \left[p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl} + p_4 \times f_{4,sl}\right] S_A \\ &= \exp[-2.3243] \times \left[p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl} + p_4 \times f_{4,sl}\right] S_B \\ &= \exp[-0.4627] \times \left[p_1 \times f_{1,sl} + p_2 \times f_{2,sl} + p_3 \times f_{3,sl} + p_4 \times f_{4,sl}\right] \end{aligned}$$

The proportion of total leg volume p_j is computed using Equation?.

- where
- S_l = distribution score for severity l (l = K, A, B); and
- p_j = proportion of total leg traffic volume associated with roundabout leg j (j = 1 to m).
-

Table 12-29. Calibrated base distribution score equations

Number of Legs	Number of Circulating Lanes	F_K	F_A	F_B
3	1	-3.4725	-1.1752	-0.0415
4	1	-4.6216	-2.3243	-0.4627
3	2	-3.3124	-1.0151	-0.3639
4	2	-4.4615	-2.1642	-0.7851

Step 3 – Apply Severity Distribution (optional). The distribution scores computed in the previous step are used in the following equations to compute the predicted distribution of fatal-and-injury crashes.

- **Equation 12-?**

$$P_K = \frac{S_K}{1 + S_K + S_A + S_B}$$

- **Equation 12-?**

$$P_A = \frac{S_A}{1 + S_K + S_A + S_B}$$

- **Equation 12-?**

$$P_A = \frac{S_B}{1 + S_K + S_A + S_B}$$

- **Equation 12-?**

$$P_C = 1 - (P_K + P_A + P_B)$$

- where
- P_l = probability of the occurrence of crash severity l (l = K, A, B, C); and
- S_l = distribution score for severity l (l = K, A, B).

Safety Performance Functions for Individual Intersection Legs

Leg-level models are intended to be used to address leg-specific design considerations but not to evaluate an overall intersection. Leg-level models in the HSM are currently only available for roundabouts.

The SPF model forms are shown in Equations 12-? to 12-?. The equations are used to compute the predicted crash frequency for base conditions $N_{SPF,i}$ for an individual leg, i, for various types of crashes by lane configuration and setting.

The SPFs for are applicable to the AADT ranges (Table 12-?) and the base conditions values indicated in Tables 12-?? to 12-??.

Table 12-?: Applicable AADT Ranges for Leg-Level SPFs

Crash Type/Severity/Setting	Entering/ (Exiting) Lanes	Circulating Lanes	Range for Applicable AADT			
			Entering AADT	Circulating AADT	Exiting AADT	Approach AADT
Entering-Circulating ALL	1	1	100 to 9,900	10 to 9,900	100 to 9,900	250 to 19750
	1	2	150 to 7,950	600 to 11,550	150 to 7,950	300 to 15,900
	2	1	600 to 11,200	50 to 9,150	500 to 12,700	1,200 to 23,450
	2	2	300 to 10.400	350 to 8,950	300 to 11,150	650 to 20,800
Entering-Circulating KABC	1	1 or 2	100 to 9,900	10 to 11,550	100 to 9,900	250 to 19,750
	2	1 or 2	300 to 11,200	50 to 9,150	300 to 12,700	650 to 23,450
Exiting-Circulating	(1)	2	150 to 7,950	600 to 11,550	150 to 7,950	300 to 15,900
	(2)	1	600 to 11,200	50 to 9,150	500 to 12,700	1,200 to 23,450
Rear-end (Approach)	1 or 2	1 or 2	140 to 11,200	10 to 11,550	100 to 12,700	250 to 23,450
Single-vehicle (Approach)	1 or 2	1 or 2	140 to 11,200	10 to 11,550	100 to 12,700	250 to 23,450
Single-vehicle (Approach+Circulating	1	1 or 2	100 to 9,900	10 to 11,550	100 to 9,900	250 to 19,750
Single-vehicle (Approach+Circulating	2	1 or 2	300 to 11,200	50 to 9,150	300 to 12,700	650 to 23,450
Circulating-Circulating	1 or 2	1 or 2	140 to 11,200	10 to 11,550	100 to 12,700	250 to 23,450
Total (Rural)	1	1	100 to 9,900	10 to 9,900	100 to 9,900	250 to 19750
	2	1	600 to 11,200	50 to 9,150	500 to 12,700	1,200 to 23,450
Total (Rural) (1 Exiting lane)	1	2	150 to 7,950	600 to 11,550	150 to 7,950	300 to 15,900
	2	2	300 to 10.400	350 to 8,950	300 to 11,150	650 to 20,800

Total (Rural) (2 Exiting lanes)	1	2	150 to 7,950	600 to 11,550	150 to 7,950	300 to 15,900
	2	2	300 to 10.400	350 to 8,950	300 to 11,150	650 to 20,800
Total (Urban)	1	1	100 to 9,900	10 to 9,900	100 to 9,900	250 to 19750
	2	1	600 to 11,200	50 to 9,150	500 to 12,700	1,200 to 23,450
Total (Urban) (1 Exiting lane)	1	2	150 to 7,950	600 to 11,550	150 to 7,950	300 to 15,900
	2	2	300 to 10.400	350 to 8,950	300 to 11,150	650 to 20,800
Total (Urban) (2 Exiting lanes)	1	2	150 to 7,950	600 to 11,550	150 to 7,950	300 to 15,900
	2	2	300 to 10.400	350 to 8,950	300 to 11,150	650 to 20,800

Table 12-? presents the values of the coefficients for various crash types, severities, and lane configurations and settings.

For entering-circulating crashes

- **Equation 12-?**

$$N_{SPF,i} = exp[\mathrm{a} + \mathrm{b} \times LN(EnteringAADT\ /1000) + \mathrm{c} \times LN(CirculatingAADT/1000)]$$

For exiting-circulating crashes

- **Equation 12-?**

$$N_{SPF,i} = exp[\mathrm{a} + \mathrm{b} \times LN(ExitingAADT\ /1000) + \mathrm{c} \times LN(CirculatingAADT/1000)]$$

-

For rear-end approach crashes

- **Equation 12-?**

$$N_{SPF,i} = exp[\mathrm{a} + \mathrm{b} \times LN(ApproachAADT\ \ \mathrm{c} \times LN(CirculatingAADT/1000)]$$

For single vehicle approach and single vehicle approach+circulating crashes

- **Equation 12-?**

$$N_{SPF,i} = exp[\mathrm{a} + \mathrm{b} \times LN(ApproachAADT\ /1000))]$$

For circulating-circulating crashes

- **Equation 12-?**

$$N_{SPF,i} = exp[\mathrm{a} + \mathrm{b} \times LN(CirculatingAADT/1000)]$$

For total crashes of all types

- **Equation 12-?**

$$N_{SPF,i} = exp[\mathrm{a} + \mathrm{b} \times LN(ApproachAADT/1000) + \mathrm{c} \times LN(CirculatingAADT/1000)]$$

Table 12-? SPF Coefficients for Roundabout Leg-Level Models

Crash Type/Severity/Setting	Entering/ (Exiting) Lanes	Circulating Lanes	A	Coefficient (Exponent) for Applicable AADT				Over-dispersion parameter k
				Entering AADT/ 1000	Circulating AADT/1000	Exiting AADT/ 1000	Approach AADT/ 1000	
Entering-Circulating ALL	1	1	-2.584	0.6091	0.3020			0.7470
	1	2	-0.314	0.9636	0.3917			0.6232
	2	1	-5.784	0.3608	0.6711			1.0734
	2	2	-3.006	0.8054	0.7398			0.7759
Entering-Circulating KABC	1	1 or 2	-5.590	0.9374	0.4749			0.4337
	2	1 or 2	-4.430	0.9374	0.4749			0.4337
Exiting-Circulating	(1)	2	-5.353		1.0853	0.4317		0.6159
	(2)	1	-6.582		0.5511	2.0150		1.8837
Rear-end (Approach)	1 or 2	1 or 2	-4.781		0.3034		1.0978	1.0659
Single-vehicle (Approach)	1 or 2	1 or 2	-2.696				0.3392	0.8153
Single-vehicle (Approach+Circulating	1	1 or 2	-2.553				0.4055	0.9410
Single-vehicle (Approach+Circulating	2	1 or 2	-2.007				0.4055	0.9410
Circulating-Circulating	1 or 2	1 or 2	-2.679		0.3963			1.4571
Total (Rural)	1	1	-2.619		0.2747		0.8197	0.6921
	2	1	-1.636		0.2747		0.8197	0.6921
Total (Rural) (1 Exiting lane)	1	2	-1.331		0.3306		0.4443	0.9429
	2	2	-1.036		0.3306		0.4443	0.9429

Total (Rural)	1	2	-0.950		0.3306		0.4443	0.9429
(2 Exiting lanes)	2	2	-0.655		0.3306		0.4443	0.9429
Total (Urban)	1	1	-2.986		0.2747		0.8197	0.6921
	2	1	-2.003		0.2747		0.8197	0.6921
Total (Urban)	1	2	-1.750		0.3306		0.4443	0.9429
(1 Exiting lane)	2	2	-1.160		0.3306		0.4443	0.9429
Total (Urban)	1	2	-1.370		0.3306		0.4443	0.9429
(2 Exiting lanes)	2	2	-1.075		0.3306		0.4443	0.9429

CRASH MODIFICATION FACTORS

In Step 10 of the predictive method shown in Section 12.4, crash modification factors are applied to the selected safety performance function (SPF), which was selected in Step 9. SPFs provided in Chapter 12 are presented in Section 12.6. A general overview of crash modification factors (CMFs) is presented in Section 3.5.3. The Part C—Introduction and Applications Guidance provides further discussion on the relationship of CMFs to the predictive method. This section provides details of the specific CMFs applicable to the SPFs presented in Section 12.6.

Crash modification factors (CMFs) are used to adjust the SPF estimate of predicted average crash frequency for the effect of individual geometric design and traffic control features, as shown in the general predictive model for Chapter 12 shown in Equation 12-1. The CMF for the SPF base condition of each geometric design or traffic control feature has a value of 1.00. Any feature associated with higher crash frequency than the base condition has a CMF with a value greater than 1.00; any feature associated with lower crash frequency than the base condition has a CMF with a value less than 1.00.

The CMFs used in Chapter 12 are consistent with the CMFs in Part D, although they have, in some cases, been expressed in a different form to be applicable to the base conditions of the SPFs. The CMFs presented in Chapter 12 and the specific SPFs which they apply to are summarized in Table 12-18.

Table 12-18. Summary of CMFs in Chapter 12 and the Corresponding SPFs

Applicable SPF	CMF	CMF Description	CMF Equations and Tables
Roadway Segments	CMF_{1r}	On-Street Parking	Equation 12-32 and Table 12-19
	CMF_{2r}	Roadside Fixed Objects	Equation 12-33 and Tables 12-20 and 12-21
	CMF_{3r}	Median Width	Table 12-22
	CMF_{4r}	Lighting	Equation 12-34 and Table 12-23
	CMF_{5r}	Automated Speed Enforcement	See text
Multiple-Vehicle Collisions and Single-Vehicle Crashes at Intersections	CMF_{1i}	Intersection Left-Turn Lanes	Table 12-24
	CMF_{2i}	Intersection Left-Turn Signal Phasing	Table 12-25

	CMF_{3i}	Intersection Right-Turn Lanes	Table 12-26
	CMF_{4i}	Right-Turn-on-Red	Equation 12-35
	CMF_{5i}	Lighting	Equation 12-36 and Table 12-27
	CMF_{6i}	Red-Light Cameras	Equations 12-37, 12-38, 12-39
Vehicle-Pedestrian Collisions at Signalized Intersections	CMF_{1p}	Bus Stops	Table 12-28
	CMF_{2p}	Schools	Table 12-29
	CMF_{3p}	Alcohol Sales Establishments	Table 12-30
Intersection-Level Roundabouts	CMF_{ICD}	Inscribed Circle Diameter	Equation 12-? and Table 12-?
	CMF_{outbd}	Outbound-Only Leg	Table 12-?
	CMF_{bypass}	Right-Turn Bypass Lane	Equation 12-? and Table 12-?
	$CMF_{ap,i}$	Access Point Frequency	Equation 12-? and Table 12-?
	$CMF_{ew,i}$	Entry Width	Equation 12-? and Table 12-?
	$CMF_{cl,i}$	Circulating Width	Equation 12-? and Table 12-?
Leg-Level Roundabouts	CMF_{ICD}	Inscribed Circle Diameter	Table 12-?
	$CMF_{Ang,i}$	Angle	Table 12-?
	$CMF_{cl,i}$	Circulating Width	Table 12-?
	$CMF_{Acc,i}$	Number of Access points	Table 12-?
	$CMF_{Lum,i}$	Number of Luminaires	Table 12-?
	$CMF_{SPD,i}$	Posted speed	Table 12-?
	$CMF_{bypass,i}$	Right-Turn Bypass Lane	Table 12-?

Crash Modification Factors for Roadway Segments

The CMFs for geometric design and traffic control features of urban and suburban arterial roadway segments are presented below. These CMFs are determined in Step 10 of the predictive method and used in Equation 12-3 to adjust the SPF for urban and suburban arterial roadway segments to account for differences between the base conditions and the local site conditions.

CMF_{1r}—On-Street Parking

The CMF for on-street parking, where present, is based on research by Bonneson (1). The base condition is the absence of on-street parking on a roadway segment. The CMF is determined as:

$$CMF_{1r} = 1 + p_{pk} \times \left(f_{pk} - 1.0 \right) \tag{12-32}$$

Where:

CMF_{1r} = crash modification factor for the effect of on-street parking on total crashes;

f_{pk} = factor from Table 12-19;

p_{pk} = proportion of curb length with on-street parking = (0.5 L_{pk}/L); and

L_{pk} = sum of curb length with on-street parking for both sides of the road combined (miles); and

L = length of roadway segment (miles).

This CMF applies to total roadway segment crashes.

The sum of curb length with on-street parking (L_{pk}) can be determined from field measurements or video log review to verify parking regulations. Estimates can be made by deducting from twice the roadway segment length allowances for intersection widths, crosswalks, and driveway widths.

Table 12-19. Values of f_{pk} Used in Determining the Crash Modification Factor for On-Street Parking

	Type of Parking and Land Use			
	Parallel Parking		Angle Parking	
Road Type	Residential/Other	Commercial or Industrial/Institutional	Residential/Other	Commercial or Industrial/Institutional
2U	1.465	2.074	3.428	4.853
3T	1.465	2.074	3.428	4.853
4U	1.100	1.709	2.574	3.999
4D	1.100	1.709	2.574	3.999
5T	1.100	1.709	2.574	3.999

CMF_{2r}—Roadside Fixed Objects

The base condition is the absence of roadside fixed objects on a roadway segment. The CMF for roadside fixed objects, where present, has been adapted from the work of Zegeer and Cynecki (15) on predicting utility pole crashes. The CMF is determined with the following equation:

$$CMF_{2r} = f_{\text{offset}} \times D_{fo} \times p_{fo} + \left(1.0 - p_{fo}\right) \quad (12\text{-}33)$$

Where:

CMF_{2r} = crash modification factor for the effect of roadside fixed objects on total crashes;

f_{offset} = fixed-object offset factor from Table 12-20;

D_{fo} = fixed-object density (fixed objects/mi) for both sides of the road combined; and

p_{fo} = fixed-object collisions as a proportion of total crashes from Table 12-21.

This CMF applies to total roadway segment crashes. If the computed value of CMF_{2r} is less than 1.00, it is set equal to 1.00. This can only occur for very low fixed object densities.

In estimating the density of fixed objects (D_{fo}), only point objects that are 4 inches or more in diameter and do not have breakaway design are considered. Point objects that are within 70 ft of one another longitudinally along the road are counted as a single object. Continuous objects that are not behind point objects are counted as one point object for each 70 ft of length. The offset distance (O_{fo}) shown in Table 12-20 is an estimate of the average distance from the edge of the traveled way to roadside objects over an extended roadway segment. If the average offset to fixed objects exceeds 30 ft, use the value of foffset for 30 ft. Only fixed objects on the roadside on the right side of the roadway in each direction of travel are considered; fixed objects in the roadway median on divided arterials are not considered.

Table 12-20. Fixed-Object Offset Factor

Offset to Fixed Objects (O_{fo}) (ft)	Fixed-Object Offset Factor (f_{offset})
2	0.232
5	0.133
10	0.087
15	0.068
20	0.057
25	0.049
30	0.044

Table 12-21. Proportion of Fixed-Object Collisions

Road Type	Proportion of Fixed-Object Collisions (p_{fo})
2U	0.059
3T	0.034
4U	0.037
4D	0.036

5T	0.016

CMF_{3r}—Median Width

A CMF for median widths on divided roadway segments of urban and suburban arterials is presented in Table 12-22 based on the work of Harkey et al. (6). The base condition for this CMF is a median width of 15 ft. The CMF applies to total crashes and represents the effect of median width in reducing cross-median collisions; the CMF assumes that nonintersection collision types other than cross-median collisions are not affected by median width. The CMF in Table 12-22 has been adapted from the CMF in Table 13-12 based on the estimate by Harkey et al. (6) that cross-median collisions represent 12.0 percent of crashes on divided arterials.

This CMF applies only to traversable medians without traffic barriers; it is not applicable to medians serving as TWLTLs (a CMF for TWLTLs is provided in Chapter 16). The effect of traffic barriers on safety would be expected to be a function of barrier type and offset, rather than the median width; however, the effects of these factors on safety have not been quantified. Until better information is available, a CMF value of 1.00 is used for medians with traffic barriers. The value of this CMF is 1.00 for undivided facilities.

Table 12-22. CMFs for Median Widths on Divided Roadway Segments without a Median Barrier (CMF_{3r})

Median Width (ft)	CMF
10	1.01
15	1.00
20	0.99
30	0.98
40	0.97
50	0.96
60	0.95
70	0.94
80	0.93
90	0.93
100	0.92

CMF_{4r}—Lighting

The base condition for lighting is the absence of roadway segment lighting ($CMF_{4r} = 1.00$). The CMF for lighted roadway segments is determined, based on the work of Elvik and Vaa (3), as:

$$CMF_{4r} = 1.0 - \left(p_{nr} \times \left(1.0 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}\right)\right) \quad (12\text{-}34)$$

Where:

CMF_{4r} = crash modification factor for the effect of roadway segment lighting on total crashes;

p_{inr} = proportion of total nighttime crashes for unlighted roadway segments that involve a fatality or injury;

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

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

CMF_{4r} applies to total roadway segment crashes. Table 12-23 presents default values for the nighttime crash proportions p_{inr}, p_{pnr}, and p_{nr}. Replacement of the estimates in Table 12-23 with locally derived values is encouraged. If lighting installation increases the density of roadside fixed objects, the value of CMF_{2r} is adjusted accordingly.

Table 12-23. Nighttime Crash Proportions for Unlighted Roadway Segments

	Proportion of Total Nighttime Crashes by Severity Level		Proportion of Crashes that Occur at Night
Roadway Segment Type	Fatal and Injury p_{inr}	PDO p_{pnr}	p_{nr}
2U	0.424	0.576	0.316
3T	0.429	0.571	0.304
4U	0.517	0.483	0.365
4D	0.364	0.636	0.410
5T	0.432	0.568	0.274

CMF_{5r}—Automated Speed Enforcement

Automated speed enforcement systems use video or photographic identification in conjunction with radar or lasers to detect speeding drivers. These systems automatically record vehicle identification information without the need for police officers at the scene. The base condition for automated speed enforcement is that it is absent. Chapter 17 presents a CMF of 0.83 for the reduction of all types of fatal-and-injury crashes from implementation of automated speed enforcement. This CMF is assumed to apply to roadway segments between intersections with fixed camera sites where the camera is always present or where drivers have no way of knowing whether the camera is present or not. No information is available on the effect of automated speed enforcement on noninjury crashes. With the conservative assumption that automated speed enforcement has no effect on noninjury crashes, the value of the CMF for automated speed enforcement would be 0.95.

Crash Modification Factors for Intersections

Signalized and Stop-Controlled Intersection CMFs

The effects of individual geometric design and traffic control features of intersections are represented in the predictive models by CMFs. CMF_{1i} through CMF_{6i} are applied to multiple-vehicle collisions and single-vehicle crashes at intersections, but not to vehicle-pedestrian and vehicle-bicycle collisions. CMF_{1p} through CMF_{3p} are applied to vehicle-pedestrian collisions at four-leg signalized intersections (4SG), but not to multiple-vehicle collisions and single-vehicle crashes and not to other intersection types.

CMF_{1i}—Intersection Left-Turn Lanes

The base condition for intersection left-turn lanes is the absence of left-turn lanes on the intersection approaches. The CMFs for presence of left-turn lanes are presented in Table 12-24. These CMFs apply to installation of left-turn lanes on any approach to a signalized intersection but only on uncontrolled major-road approaches to stop-controlled intersections. The CMFs for installation of left-turn lanes on multiple approaches to an intersection are equal to the

corresponding CMF for installation of a left-turn lane on one approach raised to a power equal to the number of approaches with left-turn lanes. There is no indication of any change in crash frequency for providing a left-turn lane on an approach controlled by a stop sign, so the presence of a left-turn lane on a stop-controlled approach is not considered in applying Table 12-24. The CMFs in the table apply to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions). The CMFs for installation of left-turn lanes are based on research by Harwood et al. (7). A CMF of 1.00 is always used when no left-turn lanes are present.

Table 12-24. Crash Modification Factor (CMF_{1i}) for Installation of Left-Turn Lanes on Intersection Approaches

		Number of Approaches with Left-Turn Lanes[a]			
Intersection Type	**Intersection Traffic Control**	**One Approach**	**Two Approaches**	**Three Approaches**	**Four Approaches**
Three-leg intersection	Minor-road stop control[b]	0.67	0.45	—	—
	Traffic signal	0.93	0.86	0.80	—
Four-leg intersection	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.

CMF_{2i}—Intersection Left-Turn Signal Phasing

The CMF for left-turn signal phasing is based on the results of work by Hauer (10), as modified in a study by Lyon et al. (11). Types of left-turn signal phasing considered include permissive, protected, protected/permissive, and permissive/protected. Protected/permissive operation is also referred to as a leading left-turn signal phase; permissive/protected operation is also referred to as a lagging left-turn signal phase. The CMF values are presented in Table 12-25. The base condition for this CMF is permissive left-turn signal phasing. This CMF applies to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions) and is applicable only to signalized intersections. A CMF value of 1.00 is always used for unsignalized intersections.

If several approaches to a signalized intersection have left-turn phasing, the values of CMF_{2i} for each approach are multiplied together.

Table 12-25. Crash Modification Factor (CMF_{2i}) for Type of Left-Turn Signal Phasing

Type of Left-Turn Signal Phasing	CMF_{2i}
Permissive	1.00
Protected/permissive or permissive/protected	0.99
Protected	0.94

Note: Use $CMF_{2i} = 1.00$ for all unsignalized intersections. If several approaches to a signalized intersection have left-turn phasing, the values of CMF_{2i} for each approach are multiplied together.

CMF_{3i}—Intersection Right-Turn Lanes

The base condition for intersection right-turn lanes is the absence of right-turn lanes on the intersection approaches. The CMFs for presence of right-turn lanes based on research by Harwood et al. (7) are presented in Table 12-26. These CMFs apply to installation of right-turn lanes on any approach to a signalized intersection, but only on uncontrolled major-road approaches to stop-controlled intersections. The CMFs for installation of right-turn lanes on

multiple approaches to an intersection are equal to the corresponding CMF for installation of a right-turn lane on one approach raised to a power equal to the number of approaches with right-turn lanes. There is no indication of any change in crash frequency for providing a right-turn lane on an approach controlled by a stop sign, so the presence of a right-turn lane on a stop-controlled approach is not considered in applying Table 12-26.

The CMFs in Table 12-26 apply to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions). A CMF value of 1.00 is always used when no right-turn lanes are present. This CMF applies only to right-turn lanes that are identified by marking or signing. The CMF is not applicable to long tapers, flares, or paved shoulders that may be used informally by right-turn traffic.

Table 12-26. Crash Modification Factor (CMF_{3i}) for Installation of Right-Turn Lanes on Intersection Approaches

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

CMF_{4i}—Right-Turn-on-Red

The CMF for prohibiting right-turn-on-red on one or more approaches to a signalized intersection has been derived from a study by Clark (2) and from the CMFs for right-turn-on-red operation shown in Chapter 14. The base condition for CMF_{4i} is permitting a right-turn-on-red at all approaches to a signalized intersection. The CMF is determined as:

$$CMF_{4i} = 0.98^{(n_{prohib})} \tag{12-35}$$

Where:

CMF_{4i} = crash modification factor for the effect of prohibiting right turns on red on total crashes; and

n_{prohib} = number of signalized intersection approaches for which right-turn-on-red is prohibited.

This CMF applies to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions) and is applicable only to signalized intersections. A CMF value of 1.00 is used for unsignalized intersections.

CMF_{5i}—Lighting

The base condition for lighting is the absence of intersection lighting. The CMF for lighted intersections is adapted from the work of Elvik and Vaa (3), as:

$$CMF_{5i} = 1 - 0.38 \times p_{ni} \tag{12-36}$$

Where:

CMF_{5i} = crash modification factor for the effect of intersection 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 collisions). Table 12-27 presents default values for the nighttime crash proportion, p_{ni}. HSM users are encouraged to replace the estimates in Table 12-27 with locally derived values.

Table 12-27. Nighttime Crash Proportions for Unlighted Intersections

	Proportion of Crashes that Occur at Night
Intersection Type	p_{ni}
3ST	0.238
4ST	0.229
3SG and 4SG	0.235

CMF$_{6i}$—Red-Light Cameras

The base condition for red light cameras is their absence. The CMF for installation of a red light camera for enforcement of red signal violations at a signalized intersection is based on an evaluation by Persaud et al. (12). As shown in Chapter 14, this study indicates a CMF for red light camera installation of 0.74 for right-angle collisions and a CMF of 1.18 for rear-end collisions. In other words, red light cameras would typically be expected to reduce right-angle collisions and increase rear-end collisions. There is no evidence that red light camera installation affects other collision types. Therefore, a CMF for the effect of red light camera installation on total crashes can be computed with the following equations:

$$CMF_{6i} = 1 - p_{ra} \times (1 - 0.74) - p_{re} \times (1 - 1.18) \quad (12\text{-}37)$$

$$p_{ra} = \frac{p_{ramv(FI)} \times N_{bimv(FI)} + p_{ramv(PDO)} \times N_{bimv(PDO)}}{\left(N_{bimv(FI)} + N_{bimv(PDO)} + N_{bisv}\right)} \quad (12\text{-}38)$$

$$p_{re} = \frac{p_{remv(FI)} \times N_{bimv(FI)} + p_{remv(PDO)} \times N_{bimv(PDO)}}{\left(N_{bimv(FI)} + N_{bimv(PDO)} + N_{bisv}\right)} \quad (12\text{-}39)$$

Where:

CMF_{6i} = crash modification factor for installation of red light cameras at signalized intersections;

p_{ra} = proportion of crashes that are multiple-vehicle, right-angle collisions;

p_{re} = proportion of crashes that are multiple-vehicle, rear-end collisions;

$p_{ramv(FI)}$ = proportion of multiple-vehicle fatal-and-injury crashes represented by right-angle collisions;

$p_{ramv(PDO)}$ = proportion of multiple-vehicle property-damage-only crashes represented by right-angle collisions;

$p_{remv(FI)}$ = proportion of multiple-vehicle fatal-and-injury crashes represented by rear-end collisions; and

$p_{remv(PDO)}$ = proportion of multiple-vehicle property-damage-only crashes represented by rear-end collisions.

The values of $N_{bimv(FI)}$ is available from Equation 12-22, the value of $N_{bimv(PDO)}$ is available from Equation 12-23, and the value of N_{bisv} is available from Equation 12-24. The values of $p_{ramv(FI)}$, $p_{ramv(PDO)}$, $p_{remv(FI)}$, and $p_{remv(PDO)}$ can be determined from data for the applicable intersection type in Table 12-11. The values in Table 12-11 may be updated with data for a particular jurisdiction as part of the calibration process presented in Part C, Appendix A. The data in Table 12-11, by definition, represent average values for a broad range of signalized intersections. Because jurisdictions are likely to implement red-light cameras at intersections with higher than average proportions of right-angle collisions, it is acceptable to replace the values in Table 12-11 with estimate based on data for a specific intersection when determining the value of the red light camera CMF.

Crash Modification Factors for Vehicle-Pedestrian Collisions at Signalized Intersections

The CMFs for vehicle-pedestrian collisions at signalized intersections are presented below.

CMF_{1p}—Bus Stops

The CMFs for the number of bus stops within 1,000 ft of the center of the intersection are presented in Table 12-28. The base condition for bus stops is the absence of bus stops near the intersection. These CMFs apply to total vehicle-pedestrian collisions and are based on research by Harwood et al. (8).

Table 12-28. Crash Modification Factor (CMF_{1p}) for the Presence of Bus Stops near the Intersection

Number of Bus Stops within 1,000 ft of the Intersection	CMF_{1p}
0	1.00
1 or 2	2.78
3 or more	4.15

In applying Table 12-28, multiple bus stops at the same intersection (i.e., bus stops in different intersection quadrants or located some distance apart along the same intersection leg) are counted separately. Bus stops located at adjacent intersections would also be counted as long as any portion of the bus stop is located within 1,000 ft of the intersection being evaluated.

CMF_{2p}—Schools

The base condition for schools is the absence of a school near the intersection. The CMF for schools within 1,000 ft of the center of the intersection is presented in Table 12-29. A school may be counted if any portion of the school grounds is within 1,000 ft of the intersection. Where one or more schools are located near the intersection, the value of the CMF is independent of the number of schools present. This CMF applies to total vehicle-pedestrian collisions and is based on research by Harwood et al. (8).

This CMF indicates that an intersection with a school nearby is likely to experience more vehicle-pedestrian collisions than an intersection without schools even if the traffic and pedestrian volumes at the two intersections are identical. Such increased crash frequencies indicate that school children are at higher risk than other pedestrians.

Table 12-29. Crash Modification Factor (CMF_{2p}) for the Presence of Schools near the Intersection

Presence of Schools within 1,000 ft of the Intersection	CMF_{2p}

No school present	1.00
School present	1.35

CMF_{3p}—Alcohol Sales Establishments

The base condition for alcohol sales establishments is the absence of alcohol sales establishments near the intersection. The CMF for the number of alcohol sales establishments within 1,000 ft of the center of an intersection is presented in Table 12-30. Any alcohol sales establishment wholly or partly within 1,000 ft of the intersection may be counted. The CMF applies to total vehicle-pedestrian collisions and is based on research by Harwood et al. (8).

This CMF indicates that an intersection with alcohol sales establishments nearby is likely to experience more vehicle-pedestrian collisions than an intersection without alcohol sales establishments even if the traffic and pedestrian volumes at the two intersections are identical. This indicates the likelihood of higher risk behavior on the part of either pedestrians or drivers near alcohol sales establishments. The CMF includes any alcohol sales establishment which may include liquor stores, bars, restaurants, convenience stores, or grocery stores. Alcohol sales establishments are counted if they are on any intersection leg or even on another street, as long as they are within 1,000 ft of the intersection being evaluated.

Table 12-30. Crash Modification Factor (CMF_{3p}) for the Number of Alcohol Sales Establishments near the Intersection

Number of Alcohol Sales Establishments within 1,000 ft of the Intersection	CMF_{3p}
0	1.00
1–8	1.12
9 or more	1.56

Roundabout Intersection-Level CMFs

There are two types of crash modification factors applied to the intersection-level roundabout models. The first apply to the roundabout as a whole and are applied directly. The second apply to each individual roundabout leg and are then aggregated to the roundabout as a whole prior to application. Guidance is given in this section on how to aggregate the leg-specific CMFs in order to apply to the roundabout base models.

CMF_{ICD}—Inscribed Circle Diameter

The inscribed circle diameter CMF is computed using the following equation.

$$CMF_{ICD} = exp[\mathrm{a} \times (ICD - 125)]$$

- where
- CMF_{ICD} = crash modification factor for inscribed circle diameter at urban roundabouts; and
- ICD = inscribed circle diameter, ft.

Table 12-??. Crash Modification Factor (CMF_{ICD}) for Inscribed Circle Diameter

Circulating Lanes	Severity	a
1	FI	-0.00621

This CMF is applicable to roundabouts in an urban or suburban area with one circulating lane. It is not applicable to roundabouts in rural areas. The diameters used to calibrate this CMF ranged from 90 to 160 ft.

CMF_{outbd}—Outbound-Only Leg

The outbound-only leg CMFs are applicable to any roundabout in an urban, suburban, or rural area. They are only applicable to interchange crossroad ramp terminal roundabouts with one outbound-only leg. They are not applicable to roundabouts with two or more outbound-only legs.

Table 12-??. Crash Modification Factor (CMF_{outbd}) for Outbound Leg

Circulating Lanes	Severity	CMF_{outbd}
1	FI	0.426
2	FI	0.455

$CMF_{bypass,j}$—Right-Turn Bypass Lane

The right-turn bypass lane CMF (CMF_{bypass}) is a leg-specific CMF. It is applicable to any roundabout in an urban, suburban, or rural area. It is rationalized that this CMF can be reasonably extended to roundabouts with a bypass lane present on every leg given the independent operation of each bypass lane. This CMF is applicable when the bypass lane has add-lane, merge, or yield control.

Table 12-??. Crash Modification Factor ($CMF_{bypass,j}$) for Right-Turn Bypass Lane

Circulating Lanes	Severity	CMF_{bypass}
1	FI	0.335
2	FI	0.432

$CMF_{ap,j}$—Access Point Frequency

The access point frequency CMF is a leg-specific CMF. It is described using the following equation.

- **Equation 12-?**

$$CMF_{ap,j} = exp\left[\mathrm{a} \times n_{ap,j}\right]$$

- where
- $CMF_{ap,j}$ = crash modification factor presence of driveways or unsignalized access points on leg j (j = 1 to m); and
- $n_{ap,j}$ = number of driveways or unsignalized access points on leg j (j = 1 to m) (i.e., within 250 ft of yield line).

Table 12-??. Crash Modification Factor ($CMF_{ap,j}$) for Access Point Frequency

Circulating Lanes	Severity	*a*
1	FI	0.0659
1	PDO	0.0885

This CMF is applicable to a roundabout in an urban, suburban, or rural area. The count of access points on a leg represents the number of driveways or unsignalized access points on the leg (either side) within 250 ft of the yield line. The number of access points in the data used to calibrate this CMF ranged from 0 to 8 access points per leg.

$CMF_{ew,j}$—Entry Width

The entry width CMF is a leg-specific CMF. It is described using the following equation.

- **Equation 12-?**

$$CMF_{ew,j} = exp\left[\mathrm{a} \times \left(W_{ew,j} - W_{ew,b,j}\right)\right]$$

This CMF is applicable to roundabouts in an urban, suburban, or rural area with two circulating lanes. The base entry width $W_{ew,b}$ equals 29 ft. The entry widths used to calibrate this CMF ranged from 24 to 34 ft.

Table 12-??. Crash Modification Factor ($CMF_{ew,j}$) for Entry Width

Circulating Lanes	Severity	*a*
2	FI	-0.0300
2	PDO	-0.0390

$CMF_{cl,j}$—Circulating Lane CMF

The circulating lane CMF is a leg-specific CMF. It is described using the following equation.

Equation 12-?

$$CMF_{cl,j} = exp\left[a \times \left(n_{cl,j} \times n_{el,j} - 4\right)\right]$$

This CMF is applicable to a roundabout in an urban, suburban, or rural area. The number of circulating lanes n_{cl} used to calibrate this CMF ranged from 1 to 2. The number of entering lanes n_{el} ranged from 1 to 2. This CMF is not applicable to outbound-only legs.

Table 12-??. Crash Modification Factor ($CMF_{cl,j}$) for Circulating Width

Circulating Lanes	Severity	*a*
2	FI	0.1960
2	PDO	0.2190

Aggregating Leg-Specific CMFs

The leg-specific CMFs of interest need to have their effect aggregated to the overall intersection level (and thereby, consistent with the SPFs which are also applicable to the overall intersection). These include the CMFs $CMF_{bypass,j}$, $CMF_{ap,j}$, $CMF_{ew,j}$ and $CMF_{cl,j}$.

As a first activity, for each roundabout leg *j* with one or more leg-specific CMFs, the following equation is used to compute the leg CMF.

Equation 12-?

$$CMF_j = CMF_{bypass,j} \times CMF_{ap,j} \times CMF_{ew,j} \times CMF_{cl,j}$$

where CMF_j is the combined crash modification factor for leg *j* (*j* = 1 to *m*). Only the leg-specific CMFs applicable are applied.

Then, the aggregated CMFs for three- and four-leg roundabouts are computed using Equation 12-? and Equation 12-?, respectively.

<u>Aggregated CMF for three-leg roundabouts (*m* = 3).</u>

Equation 12-?

$$CMF_{legs} = (p_1 \times CMF_1) + (p_2 \times CMF_2) + (p_3 \times CMF_3)$$

Equation 12-?

$$p_j = \frac{AADT_j}{AADT_1 + AADT_2 + AADT_3}$$

<u>Aggregated CMF for four-leg roundabouts (*m* = 4).</u>

Equation 12-?

$$CMF_{legs} = (p_1 \times CMF_1) + (p_2 \times CMF_2) + (p_3 \times CMF_3) + (p_4 \times CMF_4)$$

Equation 12-?

$$p_j = \frac{AADT_j}{AADT_1 + AADT_2 + AADT_3 + AADT_4}$$

where

CMF_{legs} = aggregate crash modification factor for all legs;

p_j = proportion of total leg traffic volume associated with roundabout leg j (j = 1 to m);

CMF_j = combined crash modification factor for leg j (j = 1 to m); and

$AADT_j$ = annual average daily traffic (AADT) volume for roundabout leg j (j= 1 to m), veh/d.

12.7.6 Roundabout Leg-Level CMFs

Tables 12-? to 12-? show the CMFs that can be applied to the base condition SPFs in Table -?. The variable definitions are as follows:

ICD – Inscribed Circle Diameter in ft.

Angle – Angle in degrees

CircWidth – Circulating Width in ft.

NumberAccess – Number of access points within 250 ft. of entry point on arm

Luminaires – Number of luminaires within 250 ft. of entry point on arm

PostedSpeed – Posted speed on approach in mph

Table 12-?. CMFs for entering-circulating crashes.

Variable	Circulating Lanes	Entering Lanes	Variable Levels			CMFs for each unit increase	CMF s.e.
			Base Value	minimum	maximum		
Circulating Width	1	2	30 ft.	15 ft.	42 ft.	1.0324	0.0190
	2	1		25 ft.	45 ft.	0.9860	0.0643
	2	2		24 ft.	45 ft.	0.8715	0.0504
ICD	1	1	150 ft.	65 ft.	236 ft.	0.9932	0.0043
	1	2		110 ft.	314 ft.	0.9918	0.0046
	2	1		135 ft.	426 ft.	0.9853	0.0060

Angle	1	2	90°	53°	182°	0.9769	0.0067
	2	2		69°	182°	0.9867	0.0052
Bypass Lane Present	1	1	None			0.3685	0.1856

- **Table 12-?. CMFs for KABC entering-circulating crashes.**

Variable	Circulating Lanes	Entering Lanes	Variable Levels			CMFs for each unit increase	CMF s.e.
			Base value	minimum	maximum		
ICD	All	All	150 ft.	65	426	0.9951	0.0025
Angle	All	All	90°	37°	186°	0.9825	0.0081

- **Table 12-? CMFs for exiting-circulating crashes.**

Variable	Circulating Lanes	Exiting Lanes	Variable Levels			CMFs/unit increase	CMF s.e.
			Base value	minimum	maximum		
Circulating Width	1	2	30 ft.	15 ft.	42 ft.	1.198	0.0441
	2	1		25 ft.	45 ft.	0.772	0.0667
ICD	2	1	150 ft.	110 ft.	426 ft.	0.985	0.0048

-

- **Table 12-?. CMFs for rear-end approach crashes.**

Variable	Variable Levels			CMFs for each unit increase	CMF s.e.
	Base value	Minimum	maximum		
NumberAccess	1	0	8	1.094	0.0609
Luminaires	2	0	8	0.937	0.0395

- **Table 12-? CMFs for single-vehicle approach crashes.**

Variable	Base valus	Variable min	Variable max	CMFs for each unit increase	CMF s.e.
PostedSpeed	40 mph	10 mph	60 mph	1.0451	0.0103

- **Table 12-?. CMFs for circulating-circulating crashes.**

Variable	Variable Levels			CMFs for each unit increase	CMF s.e.
	Base value	minimum	maximum		
CircWidth	30 ft.	24 ft.	45 ft.	0.917	0.0490

- **Table 12-?. CMFs for single-vehicle circulating plus approach crashes.**

Variable				CMFs for each unit increase	CMF s.e.
	Base value	minimum	maximum		
PostedSpeed	40 mph	10 mph	60 mph	1.0356	0.0078
CircWidth	30 ft.	14 ft.	45 ft.	0.9771	0.0098

12.8 CALIBRATION OF THE SPFS TO LOCAL CONDITIONS

In Step 10 of the predictive method, presented in Section 12.4, the predictive model is calibrated to local state or geographic conditions. Crash frequencies, even for nominally similar roadway segments or intersections, can vary widely from one jurisdiction to another. Geographic regions differ markedly in climate, animal population, driver populations, crash reporting threshold, and crash reporting practices. These variations may result in some jurisdictions experiencing a different number of reported traffic crashes on urban and suburban arterial highways than others. Calibration factors are included in the methodology to allow highway agencies to adjust the SPFs to match actual local conditions.

The calibration factors for roadway segments and intersections (defined below as C_r and C_i, respectively) will have values greater than 1.0 for roadways that, on average, experience more crashes than the roadways used in the development of the SPFs. The calibration factors for roadways that experience fewer crashes on average than the roadways used in the development of the SPFs will have values less than 1.0. The calibration procedures are presented in Part C, Appendix A.

Calibration factors provide one method of incorporating local data to improve estimated crash frequencies for individual agencies or locations. Several other default values used in the methodology, such as collision type distribution, can also be replaced with locally derived values. The derivation of values for these parameters is addressed in the calibration procedure in Part C, Appendix A.

LIMITATIONS OF PREDICTIVE METHOD IN CHAPTER 12

The limitations of the predictive method which apply generally across all of the Part C chapters are discussed in Section C.8. This section discusses limitations of the specific predictive models and the application of the predictive method in Chapter 12.

Where urban and suburban arterials intersect access-controlled facilities (i.e., freeways), the grade-separated interchange facility, including the arterial facility within the interchange area, cannot be addressed with the predictive method for urban and suburban arterials.

APPLICATION OF CHAPTER 12 PREDICTIVE METHOD

The predictive method presented in Chapter 12 applies to urban and suburban arterials. The predictive method is applied to by following the 18 steps presented in Section 12.4. Appendix 12A provides a series of worksheets for applying the predictive method and the predictive models detailed in this chapter. All computations within these worksheets are conducted with values expressed to three decimal places. This level of precision is needed for consistency in computations. In the last stage of computation, rounding the final estimate expected average crash frequency to one decimal place.

SUMMARY

The predictive method is used to estimate the expected average crash frequency for a series of contiguous sites (entire urban or suburban arterial facility), or a single individual site. An urban or suburban facility is defined in Section 12.3.

The predictive method for urban and suburban arterial highways is applied by following the 18 steps of the predictive method presented in Section 12.4. Predictive models, developed for urban and suburban arterial facilities, are applied in Steps 9, 10, and 11 of the method. These models have been developed to estimate the predicted average crash frequency of an individual intersection or homogenous roadway segment. The facility is divided into these individual sites in Step 5 of the predictive method.

Where observed data are available, the EB Method may be applied in Step 13 or 15 of the predictive method to improve the reliability of the estimate. The EB Method can be applied at the site-specific level or at the project specific level. It may also be applied to a future time period if site conditions will not change in the future period. The EB Method is described in Part C, Appendix A.2.

Each predictive model in Chapter 12 consists of a safety performance function (SPF), crash modification factors (CMFs), a calibration factor, and pedestrian and bicyclist factors. The SPF is selected in Step 9 and is used to estimate the predicted average crash frequency for a site with base conditions. This estimate can be for either total crashes or organized by crash-severity or collision-type distribution. In order to account for differences between the base conditions of the SPF and the actual conditions of the local site, CMFs are applied in Step 10 which adjust the predicted number of crashes according the geometric conditions of the site.

In order to account for the differences in state or regional crash frequencies, the SPF is calibrated to the specific state and or geographic region to which they apply. The process for determining calibration factors for the predictive models is described in Part C, Appendix A.1.

Section 12.13 presents six sample problems which detail the application of the predictive method. A series of template worksheets have been developed to assist with applying the predictive method in Chapter 12. These worksheets are utilized to solve the sample problems in Section 12.13, and Appendix 12A contains blank versions of the worksheets.

SAMPLE PROBLEMS

In this section, six sample problems are presented using the predictive method steps for urban and suburban arterials. Sample Problems 1 and 2 illustrate how to calculate the predicted average crash frequency for urban and suburban arterial roadway segments. Sample Problem 3 illustrates how to calculate the predicted average crash frequency for a stop-controlled intersection. Sample Problem 4 illustrates a similar calculation for a signalized intersection. Sample Problem 5 illustrates how to combine the results from Sample Problems 1 through 4 in a case where site-specific observed crash data are available (i.e., using the site-specific EB Method). Sample Problem 6 illustrates

how to combine the results from Sample Problems 1 through 4 in a case where site-specific observed crash data are not available (i.e., using the project-level EB Method).

Table 12-31. List of Sample Problems in Chapter 12

Problem No.	Page No.	Description
1	12-49	Predicted average crash frequency for a three-lane TWLTL arterial roadway segment
2	12-63	Predicted average crash frequency for a four-lane divided arterial roadway segment
3	12-74	Predicted average crash frequency for a three-leg stop-controlled intersection
4	12-86	Predicted average crash frequency for a four-leg signalized intersection
5	12-97	Expected average crash frequency for a facility when site-specific observed crash data are available
6	12-101	Expected average crash frequency for a facility when site-specific observed crash data are not available

Sample Problem 1

The Site/Facility
A three-lane urban arterial roadway segment with a center two-way left-turn lane (TWLTL).

- **The Question**

What is the predicted average crash frequency of the roadway segment for a particular year?

- **The Facts**
- 1.5-mi length
- 11,000 veh/day
- 1.0 mi of parallel on-street commercial parking on each side of street
- 30 driveways (10 minor commercial, 2 major residential, 15 minor residential, 3 minor industrial/institutional)
- 10 roadside fixed objects per mile
- 6-ft offset to roadside fixed objects
- Lighting present
- 35-mph posted speed

- **Assumptions**

Collision type distributions used are the default values presented in Tables 12-4 and 12-6 and Equations 12-19 and 12-20.

The calibration factor is assumed to be 1.00.

- **Results**

Using the predictive method steps as outlined below, the predicted average crash frequency for the roadway segment in Sample Problem 1 is determined to be 7.0 crashes per year (rounded to one decimal place).

- **Steps**

Step 1 through 8
To determine the predicted average crash frequency of the roadway segment in Sample Problem 1, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9—For the selected site, determine and apply the appropriate safety performance function (SPF) for the site's facility type and traffic control features.
For a three-lane urban arterial roadway segment with TWLTL, SPF values for multiple-vehicle nondriveway, single-vehicle, multiple-vehicle driveway-related, vehicle-pedestrian, and vehicle-bicycle collisions are determined. The calculations for vehicle-pedestrian and vehicle-bicycle collisions are shown in Step 10 since the CMF values are needed for these models.

Multiple-Vehicle Nondriveway Collisions
The SPF for multiple-vehicle nondriveway collisions for the roadway segment is calculated from Equation 12-10 and Table 12-3 as follows:

$$
\begin{aligned}
N_{brmv} &= exp\left(a + b \times \ln\left(AADT\right) + \ln\left(L\right)\right) \\
N_{brmv(\text{total})} &= exp\left(-12.40 + 1.41 \times \ln\left(11{,}000\right) + \ln\left(1.5\right)\right) \\
&= 3.805 \text{ crashes/year} \\
N_{brmv(FI)} &= \exp\left(-16.45 + 1.69 \times \ln\left(11{,}000\right) + \ln\left(1.5\right)\right) \\
&= 0.728 \text{ crashes/year} \\
N_{brmv(PDO)} &= exp\left(-11.95 + 1.33 \times \ln\left(11{,}000\right) + \ln\left(1.5\right)\right) \\
&= 2.298 \text{ crashes/year}
\end{aligned}
$$

These initial values for fatal-and-injury (FI) and property-damage-only (PDO) crashes are then adjusted using Equations 12-11 and 12-12 to assure that they sum to the value for total crashes as follows:

$$
\begin{aligned}
N_{brmv(FI)} &= N_{brmv(\text{total})} \left(\frac{N'_{brmv(FI)}}{N'_{brmv(FI)} + N'_{brmv(PDO)}} \right) \\
&= 3.085 \left(\frac{0.728}{0.728 + 2.298} \right) \\
&= 0.742 \text{ crashes/year}
\end{aligned}
$$

$$
\begin{aligned}
N_{brmv(PDO)} &= N_{brmv(\text{total})} - N_{brmv(FI)} \\
&= 3.085 - 0.742 \\
&= 2.343 \text{ crashes/year}
\end{aligned}
$$

Single-Vehicle Crashes
The SFP for single-vehicle crashes for the roadway segments is calculated from Equation 12-13 and Table 12-5 as follows:

$$N_{brsv} = exp\left(a + b \times \ln\left(AADT\right) + \ln\left(L\right)\right)$$

$$N_{brsv(\text{total})} = exp\left(-5.74 + 0.54 \times \ln\left(11{,}000\right) + \ln\left(1.5\right)\right)$$

$$= 0.734 \text{ crashes/year}$$

$$N_{brsv(FI)} = exp\left(-6.37 + 0.47 \times \ln\left(11{,}000\right) + \ln\left(1.5\right)\right)$$

$$= 0.204 \text{ crashes/year}$$

$$N_{brsv(PDO)} = exp\left(-6.29 + 0.56 \times \ln\left(11{,}000\right) + \ln\left(1.5\right)\right)$$

$$= 0.510 \text{ crashes/year}$$

These initial values for fatal-and-injury (FI) and property-damage-only (PDO) crashes are then adjusted using Equations 12-14 and 12-15 to assure that they sum to the value for total crashes as follows:

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

$$= 0.734 \times \left(\frac{0.204}{0.204 + 0.510}\right)$$

$$= 0.210 \text{ crashes/year}$$

$$N_{brsv(PDO)} = N_{brsv(\text{total})} - N_{brsv(FI)}$$

$$= 0.734 - 0.210$$

$$= 0.524 \text{ crashes/year}$$

Multiple-Vehicle Driveway-Related Collisions

The SPF for multiple-vehicle driveway-related collisions for the roadway segment is calculated from Equation 12-16 as follows:

$$N_{brdwy(\text{total})} = \sum_{\substack{\text{all} \\ \text{driveway} \\ \text{types}}} n_j \times N_j \times \left(\frac{AADT}{15{,}000}\right)^{(t)}$$

The number of driveways within the roadway segment, n_j, for Sample Problem 1 is 10 minor commercial, two major residential, 15 minor residential, and three minor industrial/institutional.

The number of driveway-related collisions, N_j, and the regression coefficient for AADT, t, for a three-lane arterial are provided in Table 12-7.

$$N_{brdwy(\text{total})} = 10 \times 0.032 \times \left(\frac{11{,}000}{15{,}000}\right)^{(1.0)} + 2 \times 0.053 \times \left(\frac{11{,}000}{15{,}000}\right)^{(1.0)}$$

$$+ 15 \times 0.010 \times \left(\frac{11{,}000}{15{,}000}\right)^{(1.0)} + 3 \times 0.015 \times \left(\frac{11{,}000}{15{,}000}\right)^{(1.0)}$$

$$= 0.455 \text{ crashes/year}$$

Driveway-related collisions can be separated into components by severity level using Equations 12-17 and 12-18 as follows:

From Table 12-7, for a three-lane arterial the proportion of driveway-related collisions that involve fatalities and injuries, $f_{dwy} = 0.243$

$$\begin{aligned} N_{brdwy(FI)} &= N_{brdwy(\text{total})} \times f_{dwy} \\ &= 0.455 \times 0.243 \\ &= 0.111 \text{ crashes/year} \end{aligned}$$

$$\begin{aligned} N_{brdwy(PDO)} &= N_{brdwy(\text{total})} - N_{brdwy(FI)} \\ &= 0.455 - 0.111 \\ &= 0.344 \text{ crashes/year} \end{aligned}$$

Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site specific geometric design and traffic control features.
Each CMF used in the calculation of the predicted average crash frequency of the roadway segment is calculated below:

On-Street Parking (CMF_{1r})
CMF_{1r} is calculated from Equation 12-32 as follows:

$$CMF_{1r} = 1 + p_{pk} \times \left(f_{pk} - 1.0 \right)$$

The proportion of curb length with on-street parking, p_{pk}, is determined as follows:

$$p_{pk} = 0.5 \times \frac{L_{pk}}{L}$$

Since 1.0 mile of on-street parking on each side of the road is provided, the sum of curb length with on-street parking for both sides of the road combined, $L_{pk} = 2$.

$$p_{pk} = 0.5 \times \frac{2}{1.5} = 0.66$$

From Table 12-19, $f_{pk} = 2.074$.

$$\begin{aligned} CMF_{1r} &= 1 + 0.66 \times \left(2.074 - 1.0 \right) \\ &= 1.71 \end{aligned}$$

Roadside Fixed Objects (CMF_{2r})
CMF_{2r} is calculated from Equation 12-33 as follows:

$$CMF_{2r} = f_{\text{offset}} \times D_{fo} \times p_{fo} + \left(1.0 - p_{fo} \right)$$

From Table 12-20, for a roadside fixed object with a 6-ft offset, the fixed-object offset factor, f_{offset}, is interpolated as 0.124.

From Table 12-21, for a three-lane arterial the proportion of total crashes, $p_{fo} = 0.034$.

$$CMF_{2r} = 0.124 \times 10 \times 0.034 + (1.0 - 0.034)$$
$$= 1.01$$

Median Width (CMF_{3r})

The value of CMF_{3r} is 1.00 for undivided facilities (see Section 12.7.1). It is assumed that a roadway with TWLTL is undivided.

Lighting (CMF_{4r})

CMF_{4r} is calculated from Equation 12-34 as follows:

$$CMF_{4r} = 1.0 - \left(p_{nr} \times \left(1.0 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}\right)\right)$$

For a three-lane arterial, $p_{inr} = 0.429$, $p_{pnr} = 0.571$, and $p_{nr} = 0.304$ (see Table 12-23).

$$CMF_{4r} = 1.0 - \left(0.304 \times \left(1.0 - 0.72 \times 0.429 - 0.83 \times 0.571\right)\right)$$
$$= 0.93$$

Automated Speed Enforcement (CMF_{5r})

Since there is no automated speed enforcement in Sample Problem 1, $CMF_{5r} = 1.00$ (i.e., the base condition for CMF_{5r} is the absent of automated speed enforcement).

The combined CMF value for Sample Problem 1 is calculated below.

$$CMF_{comb} = 1.71 \times 1.01 \times 0.93$$
$$= 1.61$$

Vehicle-Pedestrian and Vehicle-Bicycle Collisions

The predicted average crash frequency of an individual roadway segment (excluding vehicle-pedestrian and vehicle-bicycle collisions) for SPF base conditions, N_{br}, is calculated first in order to determine vehicle-pedestrian and vehicle-bicycle crashes. N_{br} is determined from Equation 12-3 as follows:

$$N_{br} = N_{spf\ rs} \times \left(CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{nr}\right)$$

From Equation 12-4, $N_{spf\ rs}$ can be calculated as follows:

$$N_{spf\ rs} = N_{brmv} + N_{brsv} + N_{brdwy}$$
$$= 3.085 + 0.734 + 0.455$$
$$= 4.274 \text{ crashes/year}$$

The combined CMF value for Sample Problem 1 is 1.61.

$$N_{br} = 4.274 \times (1.61)$$
$$= 6.881 \text{ crashes/year}$$

The SPF for vehicle-pedestrian collisions for the roadway segment is calculated from Equation 12-19 as follows:

$$N_{pedr} = N_{br} \times f_{pedr}$$

From Table 12-8, for a posted speed greater than 30 mph on three-lane arterials the pedestrian crash adjustment factor, $f_{pedr} = 0.013$.

$$N_{pedr} = 6.881 \times 0.013$$
$$= 0.089 \text{ crashes/year}$$

The SPF for vehicle-bicycle collisions is calculated from Equation 12-20 as follows:

$$N_{biker} = N_{br} \times f_{biker}$$

From Table 12-9, for a posted speed greater than 30 mph on three-lane arterials the bicycle crash adjustment factor, $f_{biker} = 0.007$.

$$N_{biker} = 6.881 \times 0.007$$
$$= 0.048 \text{ crashes/year}$$

Step 11—Multiply the result obtained in Step 10 by the appropriate calibration factor.
It is assumed in that a calibration factor, C_r, of 1.00 has been determined for local conditions. See Part C, Appendix A.1 for further discussion on calibration of the predicted models.

Calculation of Predicted Average Crash Frequency
The predicted average crash frequency is calculated using Equation 12-2 based on the results obtained in Steps 9 through 11 as follows:

$$N_{\text{predicted } rs} = C_r \times \left(N_{br} + N_{pedr} + N_{biker}\right)$$
$$= 1.00 \times \left(6.881 + 0.089 + 0.048\right)$$
$$= 7.018 \text{ crashes/year}$$

WORKSHEETS

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for a roadway segment. To apply the predictive method steps to multiple segments, a series of 12 worksheets are provided for determining the predicted average crash frequency. The 12 worksheets include:

- *Worksheet SP1A (Corresponds to Worksheet 1A)*—General Information and Input Data for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1B (Corresponds to Worksheet 1B)*—Crash Modification Factors for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1C (Corresponds to Worksheet 1C)*—Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1D (Corresponds to Worksheet 1D)*—Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1E (Corresponds to Worksheet 1E)*—Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1F (Corresponds to Worksheet 1F)*—Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Roadway Segments

- *Worksheet SP1G (Corresponds to Worksheet 1G)*—Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1H (Corresponds to Worksheet 1H)*—Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1I (Corresponds to Worksheet 1I)*—Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1J (Corresponds to Worksheet 1J)*—Vehicle-Bicycle Collisions for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1K (Corresponds to Worksheet 1K)*—Crash Severity Distribution for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP1L (Corresponds to Worksheet 1L)*—Summary Results for Urban and Suburban Arterial Roadway Segments

Details of these sample problem worksheets are provided below. Blank versions of the corresponding worksheets are provided in Appendix 12A.

▪ Worksheet SP1A—General Information and Input Data for Urban and Suburban Roadway Segments

Worksheet SP1A is a summary of general information about the roadway segment, analysis, input data (i.e., "The Facts"), and assumptions for Sample Problem 1.

Worksheet SP1A. General Information and Input Data for Urban and Suburban Roadway Segments

General Information		Location Information	
Analyst		Roadway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	Base Conditions	Site Conditions
Road type (2U, 3T, 4U, 4D, 5T)	—	3T
Length of segment, L (mi)	—	1.5
AADT (veh/day)	—	11,000
Type of on-street parking (none/parallel/angle)	none	parallel-commercial
Proportion of curb length with on-street parking	—	0.66
Median width (ft)	15	not present
Lighting (present/not present)	not present	present
Auto speed enforcement (present/not present)	not present	not present
Major commercial driveways (number)	—	0
Minor commercial driveways (number)	—	10
Major industrial/institutional driveways (number)	—	0

Minor industrial/institutional driveways (number)	—	3
Major residential driveways (number)	—	2
Minor residential driveways (number)	—	15
Other driveways (number)	—	0
Speed Category	—	intermediate or high speed (>30 mph)
Roadside fixed object density (fixed objects/mi)	not present	10
Offset to roadside fixed objects (ft)	not present	6
Calibration Factor, C_r	1.0	1.0

- **Worksheet SP1B. Crash Modification Factors for Urban and Suburban Roadway Segments**

In Step 10 of the predictive method, crash modification factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the CMF values. Once the value for each CMF has been determined, all of the CMFs are multiplied together in Column 6 of Worksheet SP1B which indicates the combined CMF value.

Worksheet SP1B. Crash Modification Factors for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
CMF for On-Street Parking	**CMF for Roadside Fixed Objects**	**CMF for Median Width**	**CMF for Lighting**	**CMF for Auto Speed Enforcement**	**Combined CMF**
CMF_{1r}	CMF_{2r}	CMF_{3r}	CMF_{4r}	CMF_{5r}	CMF_{comb}
from Equation 12-32	from Equation 12-33	from Table 12-22	from Equation 12-34	from Section 12.7.1	(1)*(2)*(3)*(4)*(5)
1.71	1.01	1.00	0.93	1.00	1.61

- **Worksheet SP1C—Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments**

The SPF for multiple-vehicle nondriveway collisions along the roadway segment in Sample Problem 1 is calculated using Equation 12-10 and entered into Column 4 of Worksheet SP1C. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 1 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal-and-injury (FI) and property-damage-only (PDO) crashes sum to the total crashes as illustrated in Column 6. Column 7 represents the combined CMF (from Column 6 in Worksheet SP1B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle nondriveway crashes using the values in Column 6, the combined CMF in Column 7, and the calibration factor in Column 8.

Worksheet SP1C. Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients		Overdispersion Parameter, k	Initial N_{brmv}		Adjusted N_{brmv}	Combined CMFs	Calibration Factor	Predicted N_{brmv}
	from Table 12-3								
Crash Severity Level	a	b	from Table 12-3	from Equation 12-10	Proportion of Total Crashes	$(4)_{total}$*(5)	(6) from Worksheet SP1B	C_r	(6)*(7)*(8)
Total	−12.40	1.41	0.66	3.085	1.000	3.085	1.61	1.00	4.967
Fatal and injury (FI)	−16.45	1.69	0.59	0.728	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$	0.743	1.61	1.00	1.196
					0.241				
Property damage only (PDO)	−11.95	1.33	0.59	2.298	$(5)_{total}-(5)_{FI}$	2.342	1.61	1.00	3.771
					0.759				

- **Worksheet SP1D—Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments**

Worksheet SP1D presents the default proportions for collision type (from Table 12-4) by crash severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for multiple-vehicle nondriveway crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for multiple-vehicle nondriveway crashes (from Column 9, Worksheet SP1C) into components by crash severity and collision type.

Worksheet SP1D. Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	Predicted $N_{brmv\ (FI)}$ (crashes/year)	Proportion of Collision Type $_{(PDO)}$	Predicted $N_{brmv\ (PDO)}$ (crashes/year)	Predicted $N_{brmv\ (total)}$ (crashes/year)
Collision Type	from Table 12-4	$(9)_{FI}$ from Worksheet SP1C	from Table 12-4	$(9)_{PDO}$ from Worksheet SP1C	$(9)_{total}$ from Worksheet SP1C
Total	1.000	1.196 $(2)*(3)_{FI}$	1.000	3.771 $(4)*(5)_{PDO}$	4.967 (3)+(5)
Rear-end collision	0.845	1.011	0.842	3.175	4.186
Head-on collision	0.034	0.041	0.020	0.075	0.116
Angle collision	0.069	0.083	0.020	0.075	0.158
Sideswipe, same direction	0.001	0.001	0.078	0.294	0.295
Sideswipe, opposite direction	0.017	0.020	0.020	0.075	0.095
Other multiple-vehicle collision	0.034	0.041	0.020	0.075	0.116

- **Worksheet SP1E—Single-Vehicle Crashes by Severity Level for Urban and Suburban Roadway Segments**

The SPF for single-vehicle crashes along the roadway segment in Sample Problem 1 is calculated using Equation 12-13 and entered into Column 4 of Worksheet SP1E. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 1 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal-and-injury (FI) and property-damage-only (PDO) crashes sum to the total crashes as illustrated in Column 6. Column 7 represents the combined CMF (from Column 6 in Worksheet SP1B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle nondriveway crashes using the values in Column 6, the combined CMF in Column 7, and the calibration factor in Column 8.

Worksheet SP1E. Single-Vehicle Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients		Overdispersion Parameter, k	Initial N_{brsv}		Adjusted N_{brsv}	Combined CMFs	Calibration Factor	Predicted N_{brsv}
Crash Severity Level	from Table 12-5								
	a	b	from Table 12-5	from Equation 12-13	Proportion of Total Crashes	$(4)_{total}$*(5)	(6) from Worksheet SP1B	C_r	(6)*(7)*(8)
Total	−5.74	0.54	1.37	0.734	1.000	0.734	1.61	1.00	1.182
Fatal and injury (FI)	−6.37	0.47	1.06	0.204	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.286	0.210	1.61	1.00	0.338
Property damage only (PDO)	−6.29	0.56	1.93	0.510	$(5)_{total}-(5)_{FI}$ 0.714	0.524	1.61	1.00	0.844

- **Worksheet SP1F—Single-Vehicle Crashes by Collision Type for Urban and Suburban Roadway Segments**

Worksheet SP1F presents the default proportions for collision type (from Table 12-5) by crash severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for single-vehicle crashes by collision type is presented in 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and Columns 6 (Total).

These proportions may be used to separate the predicted average crash frequency for single-vehicle crashes (from Column 9, Worksheet SP1E) into components by crash severity and collision type.

Worksheet SP1F. Single-Vehicle Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted** $N_{brsv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{brsv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{brsv\ (total)}$ **(crashes/year)**
Collision Type	**from Table 12-6**	**(9)**$_{FI}$ **from Worksheet SP1E**	**from Table 12-6**	**(9)**$_{PDO}$ **from Worksheet SP1E**	**(9)**$_{total}$ **from Worksheet SP1E**
Total	1.000	0.338 (2)*(3)$_{FI}$	1.000	0.844 (4)*(5)$_{PDO}$	1.182 (3)+(5)
Collision with animal	0.001	0.000	0.001	0.001	0.001
Collision with fixed object	0.688	0.233	0.963	0.813	1.046
Collision with other object	0.001	0.000	0.001	0.001	0.001
Other single-vehicle collision	0.310	0.105	0.035	0.030	0.135

▪ Worksheet SP1G—Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

Worksheet SP1G determines and presents the number of driveway-related multiple-vehicle collisions. The number of driveways along both sides of the road is entered in Column 2 by driveway type (Column 1). The associated number of crashes per driveway per year by driveway type as found in Table 12-7 is entered in Column 3. Column 4 contains the regression coefficient for AADT also found in Table 12-7. The initial average crash frequency of multiple-vehicle driveway-related crashes is calculated from Equation 12-16 and entered into Column 5. The overdispersion parameter from Table 12-7 is entered into Column 6; however, the overdispersion parameter is not needed for Sample Problem 1 (as the EB Method is not utilized).

Worksheet SP1G. Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
		Crashes per Driveway per Year, N_j	Coefficient for Traffic Adjustment, t	Initial N_{brdwy}	Overdispersion Parameter, k
Driveway Type	Number of Driveways, n_j	from Table 12-7	from Table 12-7	Equation 12-16 n_j*N_j*(AADT/15,000)t	from Table 12-7
Major commercial	0	0.102	1.000	0.000	
Minor commercial	10	0.032	1.000	0.235	
Major industrial/institutional	0	0.110	1.000	0.000	
Minor industrial/institutional	3	0.015	1.000	0.033	—
Major residential	2	0.053	1.000	0.078	
Minor residential	15	0.010	1.000	0.110	
Other	0	0.016	1.000	0.000	
Total	—	—	—	0.456	1.10

▪ Worksheet SP1H—Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

The initial average crash frequency of multiple-vehicle driveway-related crashes from Column 5 of Worksheet SP1G is entered in Column 2. This value is multiplied by the proportion of crashes by severity (Column 3) found in Table 12-7 and the adjusted value is entered into Column 4. Column 5 represents the combined CMF (from Column 6 in Worksheet SP1B), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency of multiple-vehicle driveway-related crashes using the values in Column 4, the combined CMF in Column 5, and the calibration factor in Column 6.

Worksheet SP1H. Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Initial N_{brdwy}	Proportion of Total Crashes (f_{dwy})	Adjusted N_{brdwy}	Combined CMFs		Predicted N_{brdwy}
Crash Severity Level	$(5)_{total}$ from Worksheet SP1G	from Table 12-7	$(2)_{total}$ *(3)	(6) from Worksheet SP1B	Calibration Factor, C_r	(4)*(5)*(6)
Total	0.456	1.000	0.456	1.61	1.00	0.734
Fatal and injury (FI)	—	0.243	0.111	1.61	1.00	0.179
Property damage only (PDO)	—	0.757	0.345	1.61	1.00	0.555

▪ Worksheet SP1I—Vehicle-Pedestrian Collisions for Urban and Suburban Roadway Segments

The predicted average crash frequency of multiple-vehicle nondriveway, single-vehicle, and multiple-vehicle driveway-related predicted crashes from Worksheets SP1C, SP1E, and SP1H are entered into Columns 2, 3, and 4, respectively. These values are summed in Column 5. Column 6 contains the pedestrian crash adjustment factor (see Table 12-8). Column 7 represents the calibration factor. The predicted average crash frequency of vehicle-pedestrian collisions (Column 8) is the product of Columns 5, 6, and 7. Since all vehicle-pedestrian crashes are assumed to involve some level of injury, there are no property-damage-only crashes.

Worksheet SP1I. Vehicle-Pedestrian Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Predicted N_{brmv}	Predicted N_{brsv}	Predicted N_{brdwy}	Predicted N_{br}	f_{pedr}		Predicted N_{pedr}
Crash Severity Level	(9) from Worksheet SP1C	(9) from Worksheet SP1E	(7) from Worksheet SP1H	(2)+(3)+(4)	from Table 12-8	Calibration Factor, C_r	(5)*(6)*(7)
Total	4.967	1.182	0.734	6.883	0.013	1.00	0.089
Fatal and injury (FI)	—	—	—	—	—	1.00	0.089

▪ Worksheet SP1J—Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

The predicted average crash frequency of multiple-vehicle nondriveway, single-vehicle, and multiple-vehicle driveway-related predicted crashes from Worksheets SP1C, SP1E, and SP1H are entered into Columns 2, 3, and 4, respectively. These values are summed in Column 5. Column 6 contains the bicycle crash adjustment factor (see Table 12-9). Column 7 represents the calibration factor. The predicted average crash frequency of vehicle-bicycle collisions (Column 8) is the product of Columns 5, 6, and 7. Since all vehicle-bicycle collisions are assumed to involve some level of injury, there are no property-damage-only crashes.

Worksheet SP1J. Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Predicted N_{brmv}	Predicted N_{brsv}	Predicted N_{brdwy}	Predicted N_{br}	f_{biker}		Predicted N_{biker}
Crash Severity Level	(9) from Worksheet SP1C	(9) from Worksheet SP1E	(7) from Worksheet SP1H	(2)+(3)+(4)	from Table 12-9	Calibration Factor, C_r	(5)*(6)*(7)
Total	4.967	1.182	0.734	6.883	0.007	1.00	0.048
Fatal and injury	—	—	—	—	—	1.00	0.048

▪ Worksheet SP1K—Crash Severity Distribution for Urban and Suburban Roadway Segments

Worksheet SP1K provides a summary of all collision types by severity level. Values from Worksheets SP1C, SP1E, SP1H, SP1I, and SP1J are presented and summed to provide the predicted average crash frequency for each severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 3)
- Total crashes (Column 4)

Worksheet SP1K. Crash Severity Distribution for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
	Fatal and Injury (FI)	Property Damage Only (PDO)	Total
Collision Type	(3) from Worksheets SP1D and SP1F; (7) from Worksheet SP1H; and (8) from Worksheets SP1I and SP1J	(5) from Worksheets SP1D and SP1F; and (7) from Worksheet SP1H	(6) from Worksheets SP1D and SP1F; (7) from Worksheet SP1H; and (8) from Worksheets SP1I and SP1J
MULTIPLE-VEHICLE			
Rear-end collisions (from Worksheet SP1D)	1.011	3.175	4.186
Head-on collisions (from Worksheet SP1D)	0.041	0.075	0.116
Angle collisions (from Worksheet SP1D)	0.083	0.075	0.158
Sideswipe, same direction (from Worksheet SP1D)	0.001	0.294	0.295
Sideswipe, opposite direction (from Worksheet SP1D)	0.020	0.075	0.095
Driveway-related collisions (from Worksheet SP1H)	0.179	0.555	0.734
Other multiple-vehicle collision (from Worksheet SP1D)	0.041	0.075	0.116
Subtotal	1.376	4.324	5.700
SINGLE-VEHICLE			
Collision with animal (from Worksheet SP1F)	0.000	0.001	0.001
Collision with fixed object (from Worksheet SP1F)	0.233	0.813	1.046
Collision with other object (from Worksheet SP1F)	0.000	0.001	0.001
Other single-vehicle collision (from Worksheet SP1F)	0.105	0.030	0.135
Collision with pedestrian (from Worksheet SP1I)	0.089	0.000	0.089
Collision with bicycle (from Worksheet SP1J)	0.048	0.000	0.048
Subtotal	0.475	0.845	1.320
Total	1.851	5.169	7.020

- **Worksheet SP1L—Summary Results for Urban and Suburban Roadway Segments**

Worksheet SP1L presents a summary of the results. Using the roadway segment length and the AADT, the worksheet presents the crash rate in miles per year (Column 4) and in million vehicle miles (Column 6).

Worksheet SP1L. Summary Results for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
	Predicted Average Crash Frequency, $N_{predicted\ rs}$ (crashes/year)	Roadway Segment Length, L (mi)	Crash Rate (crashes/mi/year)
Crash Severity Level	(Total) from Worksheet SP1K		(2)/(3)
Total	7.020	1.5	4.7
Fatal and injury (FI)	1.851	1.5	1.2
Property damage only (PDO)	5.169	1.5	3.4

Sample Problem 2

The Highway
A four-lane divided urban arterial roadway segment.

The Question
What is the predicted average crash frequency of the roadway segment for a particular year?

The Facts

- 0.75-mi length
- 23,000 veh/day
- On-street parking not permitted
- 8 driveways (1 major commercial, 4 minor commercial, 1 major residential, 1 minor residential, 1 minor industrial/institutional)
- 20 roadside fixed objects per mile
- 12-ft offset to roadside fixed objects
- 40-ft median
- Lighting present
- 30-mph posted speed

Assumptions
Collision type distributions used are the default values presented in Tables 12-4 and 12-6 and Equations 12-19 and 12-20.

The calibration factor is assumed to be 1.00.

Results
Using the predictive method steps as outlined below, the predicted average crash frequency for the roadway segment in Sample Problem 2 is determined to be 3.4 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8
To determine the predicted average crash frequency of the roadway segment in Sample Problem 2, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9—For the selected site, determine and apply the appropriate safety performance function (SPF) for the site's facility type and traffic control features.
For a four-lane divided urban arterial roadway segment, SPF values for multiple-vehicle nondriveway, single-vehicle, multiple-vehicle driveway-related, vehicle-pedestrian, and vehicle-bicycle collisions are determined. The calculations for total multiple-vehicle nondriveway, single-vehicle, and multiple-vehicle driveway-related collisions are presented below. Detailed steps for calculating SPFs for fatal-and-injury (FI) and property-damage-only (PDO) crashes are presented in Sample Problem 1. The calculations for vehicle-pedestrian and vehicle-bicycle collisions are shown in Step 10 since the CMF values are needed for these two models.

Multiple-Vehicle Nondriveway Collisions
The SPF for multiple-vehicle nondriveway collisions for the roadway segment is calculated from Equation 12-10 and Table 12-3 as follows:

$$N_{brmv} = exp\left(a + b \times \ln\left(AADT\right) + \ln\left(L\right)\right)$$
$$N_{brmv(\text{total})} = exp\left(-12.34 + 1.36 \times \ln\left(23,000\right) + \ln\left(0.75\right)\right)$$
$$= 2.804 \text{ crashes/year}$$

Single-Vehicle Crashes
The SFP for single-vehicle crashes for the roadway segments is calculated from Equation 12-13 and Table 12-5 as follows:

$$N_{brsv} = exp\left(a + b \times \ln\left(AADT\right) + \ln\left(L\right)\right)$$
$$N_{brsv(\text{total})} = exp\left(-5.05 + 0.47 \times \ln\left(23,000\right) + \ln\left(0.75\right)\right)$$
$$= 0.539 \text{ crashes/year}$$

Multiple-Vehicle Driveway-Related Collisions
The SPF for multiple-vehicle driveway-related collisions for the roadway segment is calculated from Equation 12-16 as follows:

$$N_{brdwy(\text{total})} = \sum_{\substack{\text{all} \\ \text{driveway} \\ \text{types}}} n_j \times N_j \times \left(\frac{AADT}{15,000}\right)^{(t)}$$

The number of driveways within the roadway segment, n_j, for Sample Problem 1 is one major commercial, four minor commercial, one major residential, one minor residential, and one minor industrial/institutional.

The number of driveway-related collisions, N_j, and the regression coefficient for AADT, t, for a four-lane divided arterial, are provided in Table 12-7.

$$N_{brdwy(\text{total})} = 1\times 0.033\times\left(\frac{23,000}{15,000}\right)^{(1.106)} + 4\times 0.011\times\left(\frac{23,000}{15,000}\right)^{(1.106)} + 1\times 0.018\times\left(\frac{23,000}{15,000}\right)^{(1.106)}$$
$$+1\times 0.003\times\left(\frac{23,000}{15,000}\right)^{(1.106)} + 1\times 0.005\times\left(\frac{23,000}{15,000}\right)^{(1.106)}$$
$$= 0.165 \text{ crashes/year}$$

The fatal-and-injury (FI) and property-damage-only (PDO) SPF values for multiple-vehicle nondriveway collisions, single-vehicle crashes and multiple-vehicle driveway-related collisions can be determined by using the same procedure presented in Sample Problem 1.

Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site specific geometric design and traffic control features.
Each CMF used in the calculation of the predicted average crash frequency of the roadway segment is calculated below:

On-Street Parking (CMF_{1r})
Since on-street parking is not permitted, $CMF_{1r} = 1.00$ (i.e., the base condition for CMF_{1r} is the absence of on-street parking).

Roadside Fixed Objects (CMF_{2r})
CMF_{2r} is calculated from Equation 12-33 as follows:

$$CMF_{2r} = f_{\text{offset}} \times D_{fo} \times p_{fo} + \left(1.0 - p_{fo}\right)$$

From Table 12-20, for a roadside fixed object with a 12-ft offset, the fixed-object offset factor, f_{offset}, is interpolated as 0.079.

From Table 12-21, for a four-lane divided arterial the proportion of total crashes, $p_{fo} = 0.036$.

$$CMF_{2r} = 0.079\times 20\times 0.036 + \left(1.0 - 0.036\right)$$
$$= 1.02$$

Median Width (CMF_{3r})
From Table 12-22, for a four-lane divided arterial with a 40-ft median, $CMF_{3r} = 0.97$.

Lighting (CMF_{4r})
CMF_{4r} can be calculated from Equation 12-34 as follows:

$$CMF_{4r} = 1.0 - \left(p_{nr} \times \left(1.0 - 0.72\times p_{inr} - 0.83\times p_{pnr}\right)\right)$$

For a four-lane divided arterial, $p_{inr} = 0.364$, $p_{pnr} = 0.636$, and $p_{nr} = 0.410$ (see Table 12-23).

$$CMF_{4r} = 1.0 - \left(0.410\times\left(1.0 - 0.72\times 0.364 - 0.83\times 0.636\right)\right)$$
$$= 0.91$$

Automated Speed Enforcement (CMF_{5r})

Since there is no automated speed enforcement in Sample Problem 2, $CMF_{5r} = 1.00$ (i.e., the base condition for CMF_{5r} is the absent of automated speed enforcement).

The combined CMF value for Sample Problem 2 is calculated below.

$$\begin{aligned} CMF_{comb} &= 1.02 \times 0.97 \times 0.91 \\ &= 0.90 \end{aligned}$$

Vehicle-Pedestrian and Vehicle-Bicycle Collisions

The predicted average crash frequency of an individual roadway segment (excluding vehicle-pedestrian and vehicle-bicycle collisions) for SPF base conditions, N_{br}, is calculated first in order to determine vehicle-pedestrian and vehicle-bicycle crashes. N_{br} is determined from Equation 12-3 as follows:

$$N_{br} = N_{spf\ rs} \times \left(CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{nr} \right)$$

From Equation 12-4, $N_{spf\ rs}$ can be calculated as follows:

$$\begin{aligned} N_{spf\ rs} &= N_{brmv} + N_{brsv} + N_{brdwy} \\ &= 2.804 + 0.539 + 0.165 \\ &= 3.508 \text{ crashes/year} \end{aligned}$$

The combined CMF value for Sample Problem 2 is 0.90.

$$\begin{aligned} N_{br} &= 3.508 \times (0.90) \\ &= 3.157 \text{ crashes/year} \end{aligned}$$

The SPF for vehicle-pedestrian collisions for the roadway segment is calculated from Equation 12-19 as follows:

$$N_{pedr} = N_{br} \times f_{pedr}$$

From Table 12-8, for a posted speed of 30 mph on four-lane divided arterials, the pedestrian crash adjustment factor $f_{pedr} = 0.067$.

$$\begin{aligned} N_{pedr} &= 3.157 \times 0.067 \\ &= 0.212 \text{ crashes/year} \end{aligned}$$

The SPF for vehicle-bicycle collisions is calculated from Equation 12-20 as follows:

$$N_{biker} = N_{br} \times f_{biker}$$

From Table 12-9, for a posted speed of 30 mph on four-lane divided arterials, the bicycle crash adjustment factor $f_{biker} = 0.013$.

$$\begin{aligned} N_{biker} &= 3.157 \times 0.013 \\ &= 0.041 \text{ crashes/year} \end{aligned}$$

Step 11—Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed in that a calibration factor, C_r, of 1.00 has been determined for local conditions. See Part C, Appendix A.1 for further discussion on calibration of the predicted models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 12-2 based on the results obtained in Steps 9 through 11 as follows:

$$\begin{aligned} N_{\text{predicted } rs} &= C_r \times \left(N_{br} + N_{pedr} + N_{biker} \right) \\ &= 1.00 \times \left(3.157 + 0.212 + 0.041 \right) \\ &= 3.410 \end{aligned}$$

WORKSHEETS

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for a roadway segment. To apply the predictive method steps to multiple segments, a series of 12 worksheets are provided for determining the predicted average crash frequency. The 12 worksheets include:

- *Worksheet SP2A (Corresponds to Worksheet 1A)*—General Information and Input Data for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2B (Corresponds to Worksheet 1B)*—Crash Modification Factors for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2C (Corresponds to Worksheet 1C)*—Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2D (Corresponds to Worksheet 1D)*—Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2E (Corresponds to Worksheet 1E)*—Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2F (Corresponds to Worksheet 1F)*—Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2G (Corresponds to Worksheet 1G)*—Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2H (Corresponds to Worksheet 1H)*—Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2I (Corresponds to Worksheet 1I)*—Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2J (Corresponds to Worksheet 1J)*—Vehicle-Bicycle Collisions for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2K (Corresponds to Worksheet 1K)*—Crash Severity Distribution for Urban and Suburban Arterial Roadway Segments
- *Worksheet SP2L (Corresponds to Worksheet 1L)*—Summary Results for Urban and Suburban Arterial Roadway Segments

Details of these sample problem worksheets are provided below. Blank versions of the corresponding worksheets are provided in Appendix 12A.

- **Worksheet SP2A—General Information and Input Data for Urban and Suburban Roadway Segments**

Worksheet SP2A is a summary of general information about the roadway segment, analysis, input data (i.e., "The Facts"), and assumptions for Sample Problem 2a

Worksheet SP2A. General Information and Input Data for Urban and Suburban Roadway Segments

General Information		Location Information	
Analyst		Roadway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	
Input Data		**Base Conditions**	**Site Conditions**
Road type (2U, 3T, 4U, 4D, 5T)		—	4D
Length of segment, L (mi)		—	0.75
AADT (veh/day)		—	23,000
Type of on-street parking (none/parallel/angle)		none	None
Proportion of curb length with on-street parking		—	N/A
Median width (ft)		15	40
Lighting (present/not present)		not present	present
Auto speed enforcement (present/not present)		not present	not present
Major commercial driveways (number)		—	1
Minor commercial driveways (number)		—	4
Major industrial/institutional driveways (number)		—	—
Minor industrial/institutional driveways (number)		—	1
Major residential driveways (number)		—	1
Minor residential driveways (number)		—	1
Other driveways (number)		—	—
Speed Category		—	Low (30mph)
Roadside fixed object density (fixed objects/mi)		not present	20
Offset to roadside fixed objects (ft)		not present	12
Calibration Factor, C_r		1.0	1.0

- **Worksheet SP2B—Crash Modification Factors for Urban and Suburban Roadway Segments**

In Step 10 of the predictive method, crash modification factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the CMF values. Once the value for each CMF has been determined, all of the CMFs are multiplied together in Column 6 of Worksheet SP2B which indicates the combined CMF value.

Worksheet SP2B. Crash Modification Factors for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
CMF for On-Street Parking	**CMF for Roadside Fixed Objects**	**CMF for Median Width**	**CMF for Lighting**	**CMF for Auto Speed Enforcement**	**Combined CMF**
CMF_{1r}	CMF_{2r}	CMF_{3r}	CMF_{4r}	CMF_{5r}	CMF_{comb}
from Equation 12-32	from Equation 12-33	from Table 12-22	from Equation 12-34	from Section 12.7.1	(1)*(2)*(3)*(4)*(5)
1.00	1.02	0.97	0.91	1.00	0.90

▪ Worksheet SP2C—Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

The SPF for multiple-vehicle nondriveway collisions along the roadway segment in Sample Problem 2 is calculated using Equation 12-10 and entered into Column 4 of Worksheet SP2C. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 2 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values(from Column 4) to assure that fatal-and-injury (FI) and property-damage-only (PDO) crashes sum to the total crashes as illustrated in Column 6. Column 7 represents the combined CMF (from Column 6 in Worksheet SP2B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle nondriveway crashes using the values in Column 6, the combined CMF in Column 7, and the calibration factor in Column 8.

Worksheet SP2C. Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients		**Overdispersion Parameter, *k***	**Initial N_{brmv}**		**Adjusted N_{brmv}**	**Combined CMFs**	**Calibration Factor**	**Predicted N_{brmv}**
Crash Severity Level	**from Table 12-3**			**from Equation 12-10**			**(6) from Worksheet SP2B**		
	a	**b**	**from Table 12-3**		**Proportion of Total Crashes**	**$(4)_{total}$*(5)**		C_r	**(6)*(7)*(8)**
Total	−12.34	1.36	1.32	2.804	1.000	2.804	0.90	1.00	2.524
Fatal and injury (FI)	−12.76	1.28	1.31	0.825	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.278	0.780	0.90	1.00	0.702
Property damage only (PDO)	−12.81	1.38	1.34	2.143	$(5)_{total}-(5)_{FI}$ 0.722	2.024	0.90	1.00	1.822

▪ Worksheet SP2D—Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

Worksheet SP2D presents the default proportions for collision type (from Table 12-4) by crash severity level as follows:

- Fatal-and-injury crashes (Column 2)

- Property-damage-only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for multiple-vehicle nondriveway crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for multiple-vehicle nondriveway crashes (from Column 9, Worksheet SP2C) into components by crash severity and collision type.

Worksheet SP2D. Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted** $N_{brmv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{brmv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{brmv\ (total)}$ **(crashes/year)**
Collision Type	**from Table 12-4**	**(9)**$_{FI}$ **from Worksheet SP2C**	**from Table 12-4**	**(9)**$_{PDO}$ **from Worksheet SP2C**	**(9)total from Worksheet SP2C**
Total	1.000	0.702 (2)*(3)$_{FI}$	1.000	1.822 (4)*(5)$_{PDO}$	2.524 (3)+(5)
Rear-end collision	0.832	0.584	0.662	1.206	1.790
Head-on collision	0.020	0.014	0.007	0.013	0.027
Angle collision	0.040	0.028	0.036	0.066	0.094
Sideswipe, same direction	0.050	0.035	0.223	0.406	0.441
Sideswipe, opposite direction	0.010	0.007	0.001	0.002	0.009
Other multiple-vehicle collision	0.048	0.034	0.071	0.129	0.163

- **Worksheet SP2E—Single-Vehicle Crashes by Severity Level for Urban and Suburban Roadway Segments**

The SPF for single-vehicle crashes along the roadway segment in Sample Problem 2 is calculated using Equation 12-13 and entered into Column 4 of Worksheet SP2E. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 2 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal-and-injury (FI) and property-damage-only (PDO) crashes sum to the total crashes as illustrated in Column 6. Column 7 represents the combined CMF (from Column 6 in Worksheet SP2B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle nondriveway crashes using the values in Column 6, the combined CMF in Column 7, and the calibration factor in Column 8.

Worksheet SP2E. Single-Vehicle Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients		Overdispersion Parameter, k	Initial N_{brsv}		Adjusted N_{brsv}	Combined CMFs	Calibration Factor	Predicted N_{brsv}
Crash Severity Level	from Table 12-5								
	a	b	from Table 12-5	from Equation 12-13	Proportion of Total Crashes	$(4)_{total}$*(5)	(6) from worksheet SP2B	C_r	(6)*(7)*(8)
Total	−5.05	0.47	0.86	0.539	1.000	0.539	0.90	1.00	0.485
Fatal and injury (FI)	−8.71	0.66	0.28	0.094	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ / 0.174	0.094	0.90	1.00	0.085
Property damage only (PDO)	−5.04	0.45	1.06	0.446	$(5)_{total}-(5)_{FI}$ / 0.826	0.445	0.90	1.00	0.401

▪ Worksheet SP2F—Single-Vehicle Crashes by Collision Type for Urban and Suburban Roadway Segments

Worksheet SP2F presents the default proportions for collision type (from Table 12-5) by crash severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for single-vehicle crashes by collision type is presented in 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and Columns 6 (Total).

These proportions may be used to separate the predicted average crash frequency for single-vehicle crashes (from Column 9, Worksheet SP2E) into components by crash severity and collision type.

Worksheet SP2F. Single-Vehicle Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type (FI)	Predicted $N_{brsv\ (FI)}$ (crashes/year)	Proportion of Collision Type (PDO)	Predicted $N_{brsv\ (PDO)}$ (crashes/year)	Predicted $N_{brsv\ (total)}$ (crashes/year)
Collision Type	from Table 12-6	$(9)_{FI}$ from Worksheet SP2E	from Table 12-6	$(9)_{PDO}$ from Worksheet SP2E	$(9)_{total}$ from Worksheet SP2E
Total	1.000	0.085 $(2)*(3)_{FI}$	1.000	0.401 $(4)*(5)_{PDO}$	0.485 (3)+(5)
Collision with animal	0.001	0.000	0.063	0.025	0.025
Collision with fixed object	0.500	0.043	0.813	0.326	0.369
Collision with other object	0.028	0.002	0.016	0.006	0.008

Other single-vehicle collision	0.471	0.040	0.108	0.043	0.083

▪ Worksheet SP2G—Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

Worksheet SP2G determines and presents the number of multiple-vehicle driveway-related collisions. The number of driveways along both sides of the road is entered in Column 2 by driveway type (Column 1). The associated number of crashes per driveway per year by driveway type as found in Table 12-7 is entered in Column 3. Column 4 contains the regression coefficient for AADT also found in Table 12-7. The initial average crash frequency of multiple-vehicle driveway-related crashes is calculated from Equation 12-16 and entered into Column 5. The overdispersion parameter from Table 12-7 is entered into Column 6; however, the overdispersion parameter is not needed for Sample Problem 2 (as the EB Method is not utilized).

Worksheet SP2G. Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
		Crashes per Driveway per Year, N_j	**Coefficient for Traffic Adjustment, t**	**Initial N_{brdwy}**	**Overdispersion Parameter, k**
Driveway Type	**Number of Driveways, n_j**	**from Table 12-7**	**from Table 12-7**	**Equation 12-16 n_j*N_j*(AADT/15,000)$_t$**	**from Table 12-7**
Major commercial	1	0.033	1.106	0.053	—
Minor commercial	4	0.011	1.106	0.071	
Major industrial/institutional	0	0.036	1.106	0.000	
Minor industrial/institutional	1	0.005	1.106	0.008	
Major residential	1	0.018	1.106	0.029	
Minor residential	1	0.003	1.106	0.005	
Other	0	0.005	1.106	0.000	
Total	—	—	—	0.166	1.39

▪ Worksheet SP2H—Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

The initial average crash frequency of multiple-vehicle driveway-related crashes from Column 5 of Worksheet SP2G is entered in Column 2. This value is multiplied by the proportion of crashes by severity (Column 3) found in Table 12-7, and the adjusted value is entered into Column 4. Column 5 represents the combined CMF (from Column 6 in Worksheet SP2B), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency of multiple-vehicle driveway-related crashes using the values in Column 4, the combined CMF in Column 5, and the calibration factor in Column 6.

Worksheet SP2H. Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Initial N_{brdwy}	Proportion of Total Crashes (f_{dwy})	Adjusted N_{brdwy}	Combined CMFs		Predicted N_{brdwy}
Crash Severity Level	$(5)_{total}$ from Worksheet SP2G	from Table 12-7	$(2)_{total}$ *(3)	(6) from Worksheet SP2B	Calibration Factor, C_r	(4)*(5)*(6)
Total	0.166	1.000	0.166	0.90	1.00	0.149
Fatal and injury (FI)	—	0.284	0.047	0.90	1.00	0.042
Property damage only (PDO)	—	0.716	0.119	0.90	1.00	0.107

▪ Worksheet SP2I—Vehicle-Pedestrian Collisions for Urban and Suburban Roadway Segments

The predicted average crash frequency of multiple-vehicle nondriveway, single-vehicle, and multiple-vehicle driveway-related predicted crashes from Worksheets SP2C, SP2E, and SP2H are entered into Columns 2, 3, and 4, respectively. These values are summed in Column 5. Column 6 contains the pedestrian crash adjustment factor (see Table 12-8). Column 7 represents the calibration factor. The predicted average crash frequency of vehicle-pedestrian collisions (Column 8) is the product of Columns 5, 6, and 7. Since all vehicle-pedestrian crashes are assumed to involve some level of injury, there are no property-damage-only crashes.

Worksheet SP2I. Vehicle-Pedestrian Collisions

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Predicted N_{brmv}	Predicted N_{brsv}	Predicted N_{brdwy}	Predicted N_{br}	f_{pedr}		Predicted N_{pedr}
Crash Severity Level	(9) from Worksheet SP2C	(9) from Worksheet SP2E	(7) from Worksheet SP2H	(2)+(3)+(4)	from Table 12-8	Calibration Factor, C_r	(5)*(6)*(7)
Total	2.524	0.485	0.149	3.158	0.067	1.000	0.212
Fatal and injury (FI)	—	—	—	—	—	1.00	0.212

▪ Worksheet SP2J—Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

The predicted average crash frequency of multiple-vehicle nondriveway, single-vehicle, and multiple-vehicle driveway-related predicted crashes from Worksheets SP2C, SP2E, and SP2H are entered into Columns 2, 3, and 4, respectively. These values are summed in Column 5. Column 6 contains the bicycle crash adjustment factor (see Table 12-9). Column 7 represents the calibration factor. The predicted average crash frequency of vehicle-bicycle collisions (Column 8) is the product of Columns 5, 6, and 7. Since all vehicle-bicycle collisions are assumed to involve some level of injury, there are no property-damage-only crashes.

Worksheet SP2J. Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Predicted N_{brmv}	Predicted N_{brsv}	Predicted N_{brdwy}	Predicted N_{br}	f_{biker}		Predicted N_{biker}
Crash Severity Level	(9) from Worksheet SP2C	(9) from Worksheet SP2E	(7) from Worksheet SP2H	(2)+(3)+(4)	from Table 12-9	Calibration Factor, C_r	(5)*(6)*(7)
Total	2.524	0.485	0.149	3.158	0.013	1.00	0.041
Fatal and injury	—	—	—	—	—	1.00	0.041

- **Worksheet SP2K—Crash Severity Distribution for Urban and Suburban Roadway Segments**

Worksheet SP2K provides a summary of all collision types by severity level. Values from Worksheets SP2C, SP2E, SP2H, SP2I, and SP2J are presented and summed to provide the predicted average crash frequency for each severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 3)
- Total crashes (Column 4)

Worksheet SP2K. Crash Severity Distribution for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
	Fatal and Injury (FI)	**Property Damage Only (PDO)**	**Total**
Collision Type	**(3) from Worksheet SP2D and SP2F; (7) from Worksheet SP2H; and (8) from Worksheet SP2I and SP2J**	**(5) from Worksheet SP2D and SP2F; and (7) from Worksheet SP2H**	**(6) from Worksheet SP2D and SP2F; (7) from Worksheet SP2H; and (8) from Worksheet SP2I and SP2J**
MULTIPLE-VEHICLE			
Rear-end collisions (from Worksheet SP2D)	0.584	1.206	1.790
Head-on collisions (from Worksheet SP2D)	0.014	0.013	0.027
Angle collisions (from Worksheet SP2D)	0.028	0.066	0.094
Sideswipe, same direction (from Worksheet SP2D)	0.035	0.406	0.441
Sideswipe, opposite direction (from Worksheet SP2D)	0.007	0.002	0.009
Driveway-related collisions (from Worksheet SP2H)	0.042	0.107	0.149
Other multiple-vehicle collision (from Worksheet SP2D)	0.034	0.129	0.163
Subtotal	0.744	1.929	2.673
SINGLE-VEHICLE			
Collision with animal (from Worksheet SP2F)	0.000	0.025	0.025
Collision with fixed object (from Worksheet SP2F)	0.043	0.326	0.369
Collision with other object (from Worksheet SP2F)	0.002	0.006	0.008
Other single-vehicle collision (from Worksheet SP2F)	0.040	0.043	0.083
Collision with pedestrian (from Worksheet SP2I)	0.212	0.000	0.212
Collision with bicycle (from Worksheet SP2J)	0.041	0.000	0.041
Subtotal	0.338	0.400	0.738
Total	1.082	2.329	3.411

▪ Worksheet SP2L—Summary Results for Urban and Suburban Roadway Segments

Worksheet SP2L presents a summary of the results. Using the roadway segment length and the AADT, the worksheet presents the crash rate in miles per year (Column 4) and in million vehicle miles (Column 6).

Worksheet SP2L. Summary Results for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
Crash Severity Level	Predicted Average Crash Frequency, $N_{predicted\ rs}$ (crashes/year) (Total) from Worksheet SP2K	Roadway Segment Length, L (mi)	Crash Rate (crashes/mi/year) (2)/(3)
Total	3.411	0.75	4.5
Fatal and injury (FI)	1.082	0.75	1.4
Property damage only (PDO)	2.329	0.75	3.1

Sample Problem 3

The Site/Facility
A three-leg stop-controlled intersection located on an urban arterial.

- **The Question**

What is the predicted crash frequency of the unsignalized intersection for a particular year?

- **The Facts**
- 1 left-turn lane on one major road approach
- No right-turn lanes on any approach
- AADT of major road is 14,000 veh/day
- AADT of minor road is 4,000 veh/day

- **Assumptions**

Collision type distributions used are the default values from Tables 12-11 and 12-13 and Equations 12-30 and 12-31.

The calibration factor is assumed to be 1.00.

- **Results**

Using the predictive method steps as outlined below, the predicted average crash frequency for the unsignalized intersection in Sample Problem 3 is determined to be 1.6 crashes per year (rounded to one decimal place).

- **Steps**

- **Step 1 through 8**

To determine the predicted average crash frequency of the roadway segment in Sample Problem 3, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9—For the selected site, determine and apply the appropriate safety performance function (SPF) for the site's facility type and traffic control features.
For a three-leg stop-controlled intersection, SPF values for multiple-vehicle, single-vehicle, vehicle-pedestrian, and vehicle-bicycle collisions are determined. The calculations for vehicle-pedestrian and vehicle-bicycle collisions are shown in Step 10 since the CMF values are needed for these two models.

- ***Multiple-Vehicle Crashes***

The SPF for multiple-vehicle collisions for a single three-leg stop-controlled intersection is calculated from Equation 12-21 and Table 12-10 as follows:

$$N_{bimv} = exp\left(a + b \times \ln\left(AADT_{maj}\right) + c \times \ln\left(AADT_{min}\right)\right)$$

$$N_{bimv(\text{total})} = exp\left(-13.63 + 1.11 \times \ln\left(14,000\right) + 0.41 \times \ln\left(4,000\right)\right)$$

$$= 1.892 \text{ crashes/year}$$

$$N_{bimv(FI)} = exp\left(-14.01 + 1.16 \times \ln\left(14,000\right) + 0.30 \times \ln\left(4,000\right)\right)$$

$$= 0.639 \text{ crashes/year}$$

$$N_{bimv(PDO)} = exp\left(-15.38 + 1.20 \times \ln\left(14,000\right) + 0.51 \times \ln\left(4,000\right)\right)$$

$$= 1.358 \text{ crashes/year}$$

These initial values for fatal-and-injury (FI) and property-damage-only (PDO) crashes are then adjusted using Equations 12-22 and 12-23 to assure that they sum to the value for total crashes as follows:

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

$$= 1.892 \times \left(\frac{0.639}{0.639 + 1.358} \right)$$

$$= 0.605 \text{ crashes/year}$$

$$N_{bimv(PDO)} = N_{bimv(\text{total})} - N_{bimv(FI)}$$

$$= 1.892 - 0.605$$

$$= 1.287 \text{ crashes/year}$$

- ***Single-Vehicle Crashes***

The SPF for single-vehicle crashes for a single three-leg stop-controlled intersection is calculated from Equation 12-24 and Table 12-12 as follows:

$$N_{bisv} = exp\left(a + b \times \ln\left(AADT_{maj}\right) + c \times \ln\left(AADT_{min}\right)\right)$$

$$N_{bisv(\text{total})} = exp\left(-6.81 + 0.16 \times \ln\left(14,000\right) + 0.51 \times \ln\left(4,000\right)\right)$$

$$= 0.349 \text{ crashes/year}$$

$$N_{bisv(PDO)} = exp\left(-8.36 + 0.25 \times \ln\left(14,000\right) + 0.55 \times \ln\left(4,000\right)\right)$$

$$= 0.244 \text{ crashes/year}$$

Since there are no models for fatal-and-injury crashes at a three-leg stop-controlled intersections, $N_{bisv(FI)}$ is calculated using Equation 12-27 (in place of Equation 12-25), and the initial value for $N_{bisv(PDO)}$ calculated above is then adjusted using Equation 12-26 to assure that fatal-and-injury and property-damage-only crashes sum to the value for total crashes as follows:

$$N_{bisv(FI)} = N_{bisv(\text{total})} \times f_{bisv}$$

For a three-leg stop-controlled intersection, the default proportion of fatal-and-injury crashes, $f_{bisv} = 0.31$ (see Section 12.6.2, Single-Vehicle Crashes)

$$\begin{aligned} N_{bisv(FI)} &= 0.349 \times 0.31 \\ &= 0.108 \text{ crashes/year} \\ N_{bisv(PDO)} &= N_{bisv(\text{total})} - N_{bisv(FI)} \\ &= 0.349 - 0.108 \\ &= 0.241 \text{ crashes/year} \end{aligned}$$

Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site specific geometric design and traffic control features.
Each CMF used in the calculation of the predicted average crash frequency of the intersection is calculated below:

Intersection Left-Turn Lanes (CMF_{1i})
From Table 12-24, for a three-leg stop-controlled intersection with one left-turn lane on the major road, $CMF_{1i} = 0.67$.

Intersection Left-Turn Signal Phasing (CMF_{2i})
For unsignalized intersections, $CMF_{2i} = 1.00$.

Intersection Right-Turn Lanes (CMF_{3i})
Since no right-turn lanes are present, CMF_{3i} is 1.00 (i.e., the base condition for CMF_{3i} is the absent of right-turn lanes on the intersection approaches).

Right-Turn-on-Red (CMF_{4i})
For unsignalized intersections, $CMF_{4i} = 1.00$.

Lighting (CMF_{5i})
Since there is no lighting at this intersection, CMF_{5i} is 1.00 (i.e., the base condition for CMF_{5i} is the absence of intersection lighting).

Red-Light Cameras (CMF_{6i})
For unsignalized intersections, CMF_{6i} is always 1.00.

The combined CMF value for Sample Problem 3 is 0.67.

- ***Vehicle-Pedestrian and Vehicle-Bicycle Collisions***

The predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle collisions) for SPF base conditions, N_{bi}, must be calculated in order to determine vehicle-pedestrian and vehicle-bicycle crashes. N_{bi} is determined from Equation 12-6 as follows:

$$N_{bi} = N_{spf\ int} \times \left(CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{6i} \right)$$

From Equation 12-7, $N_{spf\ int}$ can be calculated as follows:

$$\begin{aligned} N_{spf\ int} &= N_{bimv} + N_{bisv} \\ &= 1.892 + 0.349 \\ &= 2.241 \text{ crashes/year} \end{aligned}$$

The combined CMF value for Sample Problem 3 is 0.67.

$$N_{bi} = 2.241 \times (0.67)$$
$$= 1.501 \text{ crashes/year}$$

The SPF for vehicle-pedestrian collisions for a three-leg stop-controlled intersection is calculated from Equation 12-30 as follows:

$$N_{pedi} = N_{bi} \times f_{pedi}$$

From Table 12-16, for a three-leg stop-controlled intersection the pedestrian crash adjustment factor, $f_{pedi} = 0.211$.

$$N_{pedi} = 1.501 \times 0.021$$
$$= 0.032 \text{ crashes/year}$$

The SPF for vehicle-bicycle collisions is calculated from Equation 12-31 as follows:

$$N_{bikei} = N_{pedi} \times f_{bikei}$$

From Table 12-17, for a three-leg stop-controlled intersection, the bicycle crash adjustment factor $f_{bikei} = 0.016$.

$$N_{bikei} = 1.501 \times 0.016$$
$$= 0.024 \text{ crashes/year}$$

Step 11—Multiply the result obtained in Step 10 by the appropriate calibration factor.
It is assumed in Sample Problem 3 that a calibration factor, C_i, of 1.00 has been determined for local conditions. See Part C, Appendix A.1 for further discussion on calibration of the predicted models.

- ***Calculation of Predicted Average Crash Frequency***

The predicted average crash frequency is calculated using Equation 12-5 based on results obtained in Steps 9 through 11 as follows:

$$N_{\text{predicted } int} = C_i \times (N_{bi} + N_{pedi} + N_{bikei})$$
$$= 1.00 \times (1.501 + 0.032 + 0.024)$$
$$= 1.557 \text{ crashes/year}$$

WORKSHEETS

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for an intersection. To apply the predictive method steps to multiple intersections, a series of 12 worksheets are provided for determining the predicted average crash frequency at intersections. The 12 worksheets include:

- *Worksheet SP3A (Corresponds to Worksheet 2A)*—General Information and Input Data for Urban and Suburban Arterial Intersections
- *Worksheet SP3B (Corresponds to Worksheet 2B)*—Crash Modification Factors for Urban and Suburban Arterial Intersections

- *Worksheet SP3C (Corresponds to Worksheet 2C)*—Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections
- *Worksheet SP3D (Corresponds to Worksheet 2D)*—Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections
- *Worksheet SP3E (Corresponds to Worksheet 2E)*—Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Intersections
- *Worksheet SP3F (Corresponds to Worksheet 2F)*—Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Intersections
- *Worksheet SP3G (Corresponds to Worksheet 2G)*—Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections
- *Worksheet SP3J (Corresponds to Worksheet 2J)*—Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections
- *Worksheet SP3K (Corresponds to Worksheet 2K)*—Crash Severity Distribution for Urban and Suburban Arterial Intersections
- *Worksheet SP3L (Corresponds to Worksheet 2L)*—Summary Results for Urban and Suburban Arterial Intersections

Details of these sample problem worksheets are provided below. Blank versions of the corresponding worksheets are provided in Appendix 12A.

▪ Worksheet SP3A—General Information and Input Data for Urban and Suburban Arterial Intersections

Worksheet SP3A is a summary of general information about the intersection, analysis, input data (i.e., “The Facts”), and assumptions for Sample Problem 3.

Worksheet SP3A. General Information and Input Data for Urban and Suburban Arterial Intersections

General Information		Location Information	
Analyst		Roadway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	Base Conditions	Site Conditions
Intersection type (3ST, 3SG, 4ST, 4SG)	—	3ST
$AADT_{maj}$ (veh/day)	—	14,000
$AADT_{min}$ (veh/day)	—	4,000
Intersection lighting (present/not present)	not present	not present
Calibration factor, C_i	1.00	1.00
Data for unsignalized intersections only:	—	—
Number of major-road approaches with left-turn lanes (0, 1, 2)	0	1
Number of major-road approaches with right-turn lanes (0, 1, 2)	0	0
Data for signalized intersections only:	—	—
Number of approaches with left-turn lanes (0, 1, 2, 3, 4)	0	N/A
Number of approaches with right-turn lanes (0, 1, 2, 3, 4)	0	N/A
Number of approaches with left-turn signal phasing	—	N/A
Number of approaches with right-turn-on-red prohibited	0	N/A
Type of left-turn signal phasing	permissive	N/A
Intersection red light cameras (present/not present)	not present	N/A
Sum of all pedestrian crossing volumes (PedVol)	—	N/A
Maximum number of lanes crossed by a pedestrian (n_{lanesx})	—	N/A
Number of bus stops within 300 m (1,000 ft) of the intersection	0	N/A
Schools within 300 m (1,000 ft) of the intersection (present/not present)	not present	N/A
Number of alcohol sales establishments within 300 m (1,000 ft) of the intersection	0	N/A

▪ Worksheet SP3B—Crash Modification Factors for Urban and Suburban Arterial Intersections

In Step 10 of the predictive method, crash modification factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the CMF values. Once the value for each CMF has been determined, all of the CMFs are multiplied together in Column 7 of Worksheet SP3B which indicates the combined CMF value.

Worksheet SP3B. Crash Modification Factors for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
CMF for Left-Turn Lanes	**CMF for Left-Turn Signal Phasing**	**CMF for Right-Turn Lanes**	**CMF for Right-Turn-on-Red**	**CMF for Lighting**	**CMF for Red-Light Cameras**	**Combined CMF**
CMF_{1i}	CMF_{2i}	CMF_{3i}	CMF_{4i}	CMF_{5i}	CMF_{6i}	CMF_{comb}
from Table 12-24	from Table 12-25	from Table 12-26	from Equation 12-35	from Equation 12-36	from Equation 12-37	(1)*(2)*(3)*(4)*(5)*(6)
0.67	1.00	1.00	1.00	1.00	1.00	0.67

- **Worksheet SP3C—Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections**

The SPF for multiple-vehicle collisions at the intersection in Sample Problem 3 is calculated using Equation 12-22 and entered into Column 4 of Worksheet SP3C. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 3 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal-and-injury (FI) and property-damage-only (PDO) crashes sum to the total crashes as illustrated in Column 6. Column 7 represents the combined CMF (from Column 7 in Worksheet SP3B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle crashes using the values in Column 6, the combined CMF in Column 7, and the calibration factor in Column 8.

Worksheet SP3C. Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients			Overdispersion Parameter, k	Initial N_{bimv}		Adjusted N_{bimv}	Combined CMFs		Predicted N_{bimv}
	from Table 12-10									
Crash Severity Level	a	b	c	from Table 12-10	from Equation 12-22	Proportion of Total Crashes	$(4)_{total}$*(5)	(7) from Worksheet SP3B	Calibration Factor, C_i	(6)*(7)*(8)
Total	−13.36	1.11	0.41	0.80	1.892	1.000	1.892	0.67	1.00	1.268
Fatal and injury (FI)	−14.01	1.16	0.30	0.69	0.639	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.320	0.605	0.67	1.00	0.405
Property damage only (PDO)	−15.38	1.20	0.51	0.77	1.358	$(5)_{total}-(5)_{FI}$ 0.680	1.287	0.67	1.00	0.862

- **Worksheet SP3D—Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections**

Worksheet SP3D presents the default proportions for collision type (from Table 12-11) by crash severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for multiple-vehicle crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for multiple-vehicle crashes (from Column 9, Worksheet SP3C) into components by crash severity and collision type.

Worksheet SP3D. Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted** $N_{bimv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{bimv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{bimv\ (total)}$ **(crashes/year)**
Collision Type	**from Table 12-11**	**(9)**$_{FI}$ **from Worksheet SP3C**	**from Table 12-11**	**(9)**$_{PDO}$ **from Worksheet SP3C**	**(9)**$_{PDO}$ **from Worksheet SP3C**
Total	1.000	0.405 (2)*(3)$_{FI}$	1.000	0.862 (4)*(5)$_{PDO}$	1.268 (3)+(5)
Rear-end collision	0.421	0.171	0.440	0.379	0.550
Head-on collision	0.045	0.018	0.023	0.020	0.038
Angle collision	0.343	0.139	0.262	0.226	0.365
Sideswipe	0.126	0.051	0.040	0.034	0.085
Other multiple-vehicle collision	0.065	0.026	0.235	0.203	0.229

- **Worksheet SP3E—Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Intersections**

The SPF for single-vehicle crashes at the intersection in Sample Problem 3 is calculated using Equation 12-25 for total and property-damage-only (PDO) crashes and entered into Column 4 of Worksheet SP3E. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 3 (as the EB Method is not utilized). Since there are no models for fatal-and-injury crashes at a three-leg stop-controlled intersections, $N_{bisv(FI)}$ is calculated using Equation 12-27 (in place of Equation 12-25), and the value is entered into Column 4 and 6 since no further adjustment is required. Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal-and-injury (FI) and property-damage-only (PDO) crashes sum to the total crashes as illustrated in Column 6. Column 7 represents the combined CMF (from Column 7 in Worksheet SP3B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of single-vehicle crashes using the values in Column 6, the combined CMF in Column 7, and the calibration factor in Column 8.

Worksheet SP3E. Single-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients			Overdispersion Parameter, k	Initial N_{bisv}		Adjusted N_{bisv}	Combined CMFs		Predicted N_{bisv}
	from Table 12-12									
Crash Severity Level	a	b	c	from Table 12-12	from Equation 12-25; (*FI*) from Equation 12-25 or 12-27	Proportion of Total Crashes	$(4)_{total}$*(5)	(7) from Worksheet SP3B	Calibration Factor, C_i	(6)*(7)*(8)
Total	−6.81	0.16	0.51	1.14	0.349	1.000	0.349	0.67	1.00	0.234
Fatal and injury (FI)	N/A	N/A	N/A	N/A	0.108	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ / N/A	0.108	0.67	1.00	0.072
Property damage only (PDO)	−8.36	0.25	0.55	1.29	0.244	$(5)_{total}-(5)_{FI}$ / 0.693	0.242	0.67	1.00	0.162

▪ Worksheet SP3F—Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Intersections

Worksheet SP3F presents the default proportions for collision type (from Table 12-13) by crash severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for single-vehicle crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and 6 (total).

These proportions may be used to separate the predicted average crash frequency for single-vehicle crashes (from Column 9, Worksheet SP3E) into components by crash severity and collision type.

Worksheet SP3F. Single-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type (FI)	Predicted $N_{bisv\ (FI)}$ (crashes/year)	Proportion of Collision Type (PDO)	Predicted $N_{bisv\ (PDO)}$ (crashes/year)	Predicted $N_{bisv\ (total)}$ (crashes/year)
Collision Type	Table 12-13	$(9)_{FI}$ from Worksheet SP3E	Table 12-13	$(9)_{PDO}$ from Worksheet SP3E	$(9)_{PDO}$ from Worksheet SP3E
Total	1.000	0.072 $(2)*(3)_{FI}$	1.000	0.162 $(4)*(5)_{PDO}$	0.234 (3)+(5)
Collision with parked vehicle	0.001	0.000	0.003	0.000	0.000
Collision with animal	0.003	0.000	0.018	0.003	0.003
Collision with fixed object	0.762	0.055	0.834	0.135	0.190
Collision with other object	0.090	0.006	0.092	0.015	0.021
Other single-vehicle collision	0.039	0.003	0.023	0.004	0.007
Single-vehicle noncollision	0.105	0.008	0.030	0.005	0.013

▪ Worksheet SP3G—Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections

The predicted average crash frequency of multiple-vehicle predicted crashes and single-vehicle predicted crashes from Worksheets SP3C and SP3E are entered into Columns 2 and 3 respectively. These values are summed in Column 4. Column 5 contains the pedestrian crash adjustment factor (see Table 12-16). Column 6 presents the calibration factor. The predicted average crash frequency of vehicle-pedestrian collision (Column 7) is the product of Columns 4, 5, and 6. Since all vehicle-pedestrian crashes are assumed to involve some level of injury, there are no property-damage-only crashes.

Worksheet SP3G. Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Predicted N_{bimv}	Predicted N_{bisv}	Predicted N_{bi}	f_{pedi}		*Predicted* N_{pedi}
Crash Severity Level	(9) from Worksheet SP3C	(9) from Worksheet SP3E	(2)+(3)	from Table 12-16	Calibration Factor, C_i	(4)*(5)*(6)
Total	1.268	0.234	1.502	0.021	1.00	0.032
Fatal and injury (FI)	—	—	—	—	1.00	0.032

▪ Worksheet SP3J—Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

The predicted average crash frequency of multiple-vehicle predicted crashes and single-vehicle predicted crashes from Worksheets SP3C and SP3E are entered into Columns 2 and 3 respectively. These values are summed in Column 4. Column 5 contains the bicycle crash adjustment factor (see Table 12-17). Column 6 presents the calibration factor. The predicted average crash frequency of vehicle-bicycle collision (Column 7) is the product of

Columns 4, 5, and 6. Since all vehicle-bicycle crashes are assumed to involve some level of injury, there are no property-damage-only crashes.

Worksheet SP3J. Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Predicted N_{binv}	Predicted N_{bisv}	Predicted N_{bi}	f_{bikei}		Predicted N_{pedi}
Crash Severity Level	(9) from Worksheet SP3C	(9) from Worksheet SP3E	(2)+(3)	from Table 12-17	Calibration Factor, C_i	(4)*(5)*(6)
Total	1.268	0.234	1.502	0.016	1.000	0.024
Fatal and injury (FI)	—	—	—	—	1.000	0.024

▪ Worksheet SP3K—Crash Severity Distribution for Urban and Suburban Arterial Intersections

Worksheet SP3K provides a summary of all collision types by severity level. Values from Worksheets SP3D, SP3F, SP3G, and SP3J are presented and summed to provide the predicted average crash frequency for each severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 3)
- Total crashes (Column 4)

Worksheet SP3K. Crash Severity Distribution for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)
	Fatal and Injury (FI)	Property Damage Only (PDO)	Total
Collision Type	(3) from Worksheets SP3D and SP3F; (7) from SP3G and SP3J	(5) from Worksheets SP3D and SP3F	(6) from Worksheets SP3D and SP3F; (7) from SP3G and SP3J
MULTIPLE-VEHICLE COLLISIONS			
Rear-end collisions (from Worksheet SP3D)	0.171	0.379	0.550
Head-on collisions (from Worksheet SP3D)	0.018	0.020	0.038
Angle collisions (from Worksheet SP3D)	0.139	0.226	0.365
Sideswipe (from Worksheet SP3D)	0.051	0.034	0.085
Other multiple-vehicle collision (from Worksheet SP3D)	0.026	0.203	0.229
Subtotal	0.405	0.862	1.267
SINGLE-VEHICLE COLLISIONS			
Collision with parked vehicle (from Worksheet SP3F)	0.000	0.000	0.000
Collision with animal (from Worksheet SP3F)	0.000	0.003	0.003
Collision with fixed object (from Worksheet SP3F)	0.055	0.135	0.190
Collision with other object (from Worksheet SP3F)	0.006	0.015	0.021
Other single-vehicle collision (from Worksheet SP3F)	0.003	0.004	0.007
Single-vehicle noncollision (from Worksheet SP3F)	0.008	0.005	0.013
Collision with pedestrian (from Worksheet SP3G)	0.032	0.000	0.032
Collision with bicycle (from Worksheet SP3J)	0.024	0.000	0.024
Subtotal	0.128	0.162	0.290
Total	0.533	1.024	1.557

▪ Worksheet SP3L—Summary Results for Urban and Suburban Arterial Intersections

Worksheet SP3L presents a summary of the results.

Worksheet SP3L. Summary Results for Urban and Suburban Arterial Intersections

(1)	(2)
	Predicted Average Crash Frequency, $N_{predicted\ int}$ (crashes/year)
Crash Severity Level	(Total) from Worksheet SP3K
Total	1.557
Fatal and injury (FI)	0.533

Property damage only (PDO)	1.024

Sample Problem 4

The Intersection
A four-leg signalized intersection located on an urban arterial.

The Question
What is the predicted crash frequency of the signalized intersection for a particular year?

The Facts

- 1 left-turn lane on each of the two major road approaches
- 1 right-turn lane on each of the two major road approaches
- Protected/permissive left-turn signal phasing on major road
- AADT of major road is 15,000 veh/day
- AADT of minor road is 9,000 veh/day
- Lighting is present
- No approaches with prohibited right-turn-on-red
- Four-lane divided major road
- Two-lane undivided minor road
- Pedestrian volume is 1,500 peds/day
- The number of bus stops within 1,000 ft of intersection is 2
- A school is present within 1,000 ft of intersection
- The number of alcohol establishments within 1,000 ft of intersection is 6

Assumptions
Collision type distributions used are the default values from Tables 12-11 and 12-13 and Equations 12-28 and 12-31.

The calibration factor is assumed to be 1.00.

The maximum number of lanes crossed by a pedestrian is assumed to be four (crossing two through lanes, one left-turn lane, and one right-turn lane across one side of the divided major road).

Results
Using the predictive method steps as outlined below, the predicted average crash frequency for the signalized intersection in Sample Problem 4 is determined to be 3.4 crashes per year (rounded to one decimal place).

- ### Steps

- **Step 1 through 8**

To determine the predicted average crash frequency of the roadway segment in Sample Problem 4, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year and the EB Method is not applied.

- **Step 9—For the selected site, determine and apply the appropriate safety performance function (SPF) for the site's facility type and traffic control features.**

For a four-leg signalized intersection, SPF values for multiple-vehicle, single-vehicle, vehicle-pedestrian, and vehicle-bicycle collisions are determined. The calculations for total multiple- and single-vehicle collisions are presented below. Detailed steps for calculating SPFs for fatal-and-injury (FI) and property-damage-only (PDO) crashes are presented in Sample Problem 3 (for fatal-and-injury base crashes at a four-leg signalized intersection, Equation 12-25 in place of Equation 12-27 is used). The calculations for vehicle-pedestrian and vehicle-bicycle collisions are shown in Step 10 since the CMF values are needed for these two models.

- ***Multiple-Vehicle Collisions***

The SPF for multiple-vehicle collisions for a single four-leg signalized intersection is calculated from Equation 12-21 and Table 12-10 as follows:

$$N_{bimv} = exp\left(a + b \times \ln\left(AADT_{maj}\right) + c \times \ln\left(AADT_{min}\right)\right)$$

$$N_{bimv(\text{total})} = exp\left(-10.99 + 1.07 \times \ln\left(15{,}000\right) + c \times \ln\left(9{,}000\right)\right)$$

$$= 4.027 \text{ crashes/year}$$

- ***Single-Vehicle Crashes***

The SPF for single-vehicle crashes for a single four-leg signalized intersection is calculated from Equation 12-24 and Table 12-12 as follows:

$$N_{bisv} = exp\left(a + b \times \ln\left(AADT_{maj}\right) + c \times \ln\left(AADT_{min}\right)\right)$$

$$N_{bisv(\text{total})} = exp\left(-10.21 + 0.68 \times \ln\left(15{,}000\right) + 0.27 \times \ln\left(9{,}000\right)\right)$$

$$= 0.297 \text{ crashes/year}$$

- **Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site specific geometric design and traffic control features.**

Each CMF used in the calculation of the predicted average crash frequency of the intersection is calculated below. CMF_{1i} through CMF_{2i} are applied to multiple-vehicle collisions and single-vehicle crashes, while CMF_{1p} through CMF_{3p} are applied to vehicle-pedestrian collisions.

- ***Intersection Left-Turn Lanes (CMF_{1i})***

From Table 12-24, for a four-leg signalized intersection with one left-turn lane on each of two approaches, $CMF_{1i} = 0.81$.

- ***Intersection Left-Turn Signal Phasing (CMF_{2i})***

From Table 12-25, for a four-leg signalized intersection with protected/permissive left-turn signal phasing for two approaches, $CMF_{2i} = 0.98\left(0.99 * 0.99\right)$.

- ***Intersection Right-Turn Lanes (CMF_{3i})***

From Table 12-26, for a four-leg signalized intersection with one right-turn lane on each of two approaches, $CMF_{3i} = 0.92$.

- ***Right-Turn-on-Red (CMF_{4i})***

Since right-turn-on-red (RTOR) is not prohibited on any of the intersection legs, $CMF_{4i} = 1.00$ (i.e., the base condition for CMF_{4i} is permitting a RTOR at all approaches to a signalized intersection).

- ***Lighting (CMF_{5i})***

CMF_{5i} is calculated from Equation 12-36.

$$CMF_{5i} = 1 - 0.38 \times p_{ni}$$

From Table 12-27, the proportion of crashes that occur at night, $p_{ni} = 0.235$.

$$\begin{aligned} CMF_{5i} &= 1 - 0.38 \times 0.235 \\ &= 0.91 \end{aligned}$$

- ***Red-Light Cameras (CMF_{6i})***

Since no red light cameras are present at this intersection, $CMF_{6i} = 1.00$ (i.e., the base condition for CMF_{6i} is the absence of red light cameras).

The combined CMF value applied to multiple- and single-vehicle crashes in Sample Problem 4 is calculated below.

$$\begin{aligned} CMF_{comb} &= 0.81 \times 0.98 \times 0.92 \times 0.91 \\ &= 0.66 \end{aligned}$$

- ***Bus Stop (CMF_{1p})***

From Table 12-28, for two bus stops within 1,000 ft of the center of the intersection, $CMF_{1p} = 2.78$.

- ***Schools (CMF_{2p})***

From Table 12-29, for one school within 1,000 ft of the center of the intersection, $CMF_{2p} = 1.35$.

- ***Alcohol Sales Establishments (CMF_{3p})***

From Table 12-30, for six alcohol establishments within 1,000 ft of the center of the intersection, $CMF_{3p} = 1.12$.

- ***Vehicle-Pedestrian and Vehicle-Bicycle Collisions***

The SPF for vehicle-pedestrian collisions for a four-leg signalized intersection is calculated from Equation 12-28 as follows:

$$N_{pedi} = N_{pedbase} \times CMF_{1p} \times CMF_{2p} \times CMF_{3p}$$

Npedbase is calculated from Equation 12-29 using the coefficients from Table 12-14.

$$\begin{aligned} N_{pedbase} &= exp\left(a + b \times \ln\left(AADT_{\text{total}}\right) + c \times \ln\left(\frac{AADT_{min}}{AADT_{maj}}\right) + d \times \ln\left(PedVol\right) + e \times n_{lanesx} \right) \\ &= exp\left(-9.53 + 0.40 \times \ln\left(24{,}000\right) + 0.26 \times \ln\left(\frac{9{,}000}{15{,}000}\right) + 0.45 \times \ln\left(1{,}500\right) + 0.04 \times 4 \right) \\ &= 0.113 \text{ crashes/year} \end{aligned}$$

The CMF vehicle-pedestrian collision values calculated above are $CMF_{1p} = 2.78$, $CMF_{2p} = 1.35$, and $CMF_{3p} = 1.12$.

$$\begin{aligned} N_{pedi} &= 0.113 \times 2.78 \times 1.35 \times 1.12 \\ &= 0.475 \text{ crashes/year} \end{aligned}$$

The predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle collisions) for SPF base conditions, N_{bi}, must be calculated in order to determine vehicle-bicycle crashes. N_{bi} is determined from Equation 12-6 as follows:

$$N_{bi} = N_{spf\ int} \times \left(CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{6i}\right)$$

From Equation 12-7, $N_{spf\ int}$ can be calculated as follows:

$$\begin{aligned} N_{spf\ int} &= N_{bimv} + N_{bisv} \\ &= 4.027 + 0.297 \\ &= 4.324 \text{ crashes/year} \end{aligned}$$

The combined CMF value for Sample Problem 4 is 0.66.

$$\begin{aligned} N_{bi} &= 4.324 \times (0.66) \\ &= 2.854 \text{ crashes/year} \end{aligned}$$

The SPF for vehicle-bicycle collisions is calculated from Equation 12-31 as follows:

$$N_{bikei} = N_{bi} \times f_{bikei}$$

From Table 12-17, for a four-leg signalized intersection the bicycle crash adjustment factor, $f_{bikei} = 0.015$.

$$\begin{aligned} N_{bikei} &= 2.854 \times 0.015 \\ &= 0.043 \text{ crashes/year} \end{aligned}$$

- **Step 11—Multiply the result obtained in Step 10 by the appropriate calibration factor.**

It is assumed in Sample Problem 4 that a calibration factor, C_i, of 1.00 has been determined for local conditions. See Part C, Appendix A.1 for further discussion on calibration of the predicted models.

- ***Calculation of Predicted Average Crash Frequency***

The predicted average crash frequency is calculated from Equation 12-5 based on the results obtained in Steps 9 through 11 as follows:

$$\begin{aligned} N_{\text{predicted } int} &= C_i \times \left(N_{bi} + N_{pedi} + N_{bikei}\right) \\ &= 1.00 \times (2.854 + 0.475 + 0.043) \\ &= 3.372 \text{ crashes/year} \end{aligned}$$

WORKSHEETS

The step-by-step instructions abossve are provided to illustrate the predictive method for calculating the predicted average crash frequency for an intersection. To apply the predictive method steps to multiple intersections, a series of 12 worksheets are provided for determining the predicted average crash frequency at intersections. The 12 worksheets include:

- *Worksheet SP4A (Corresponds to Worksheet 2A)*—General Information and Input Data for Urban and Suburban Arterial Intersections

- *Worksheet SP4B (Corresponds to Worksheet 2B)*—Crash Modification Factors for Urban and Suburban Arterial Intersections
- *Worksheet SP4C (Corresponds to Worksheet 2C)*—Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections
- *Worksheet SP4D (Corresponds to Worksheet 2D)*—Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections
- *Worksheet SP4E (Corresponds to Worksheet 2E)*—Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Intersections
- *Worksheet SP4F (Corresponds to Worksheet 2F)*—Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Intersections
- *Worksheet SP4H (Corresponds to Worksheet 2H)*—Crash Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections
- *Worksheet SP4I (Corresponds to Worksheet 2I)*—Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections
- *Worksheet SP4J (Corresponds to Worksheet 2J)*—Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections
- *Worksheet SP4K (Corresponds to Worksheet 2K)*—Crash Severity Distribution for Urban and Suburban Arterial Intersections
- *Worksheet SP4L (Corresponds to Worksheet 2L)*—Summary Results for Urban and Suburban Arterial Intersections

Details of these sample problem worksheets are provided below. Blank versions of the corresponding worksheets are provided in Appendix 12A.

▪ Worksheet SP4A—General Information and Input Data for Urban and Suburban Arterial Intersections

Worksheet SP4A is a summary of general information about the intersection, analysis, input data (i.e., "The Facts"), and assumptions for Sample Problem 4.

Worksheet SP4A. General Information and Input Data for Urban and Suburban Arterial Intersections

General Information		Location Information	
Analyst		Roadway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	Base Conditions	Site Conditions
Intersection type (3ST, 3SG, 4ST, 4SG)	—	4SG
$AADT_{maj}$ (veh/day)	—	15,000
$AADT_{min}$ (veh/day)	—	9,000
Intersection lighting (present/not present)	not present	present

Calibration factor, C_i	1.00	1.00
Data for unsignalized intersections only:	—	—
Number of major-road approaches with left-turn lanes (0, 1, 2)	0	N/A
Number of major-road approaches with right-turn lanes (0, 1, 2)	0	N/A
Data for signalized intersections only:	—	—
Number of approaches with left-turn lanes (0, 1, 2, 3, 4)	0	2
Number of approaches with right-turn lanes (0, 1, 2, 3, 4)	0	2
Number of approaches with left-turn signal phasing	—	2
Number of approaches with right-turn-on-red prohibited	0	0
Type of left-turn signal phasing	permissive	protected/permissive
Intersection red-light cameras (present/not present)	not present	not present
Sum of all pedestrian crossing volumes (PedVol)	—	1,500
Maximum number of lanes crossed by a pedestrian (n_{lanesx})	—	4
Number of bus stops within 300 m (1,000 ft) of the intersection	0	2
Schools within 300 m (1,000 ft) of the intersection (present/not present)	not present	present
Number of alcohol sales establishments within 300 m (1,000 ft) of the intersection	0	6

- **Worksheet SP4B—Crash Modification Factors for Urban and Suburban Arterial Intersections**

In Step 10 of the predictive method, crash modification factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the CMF values. Once the value for each CMF has been determined, all of the CMFs are multiplied together in Column 7 of Worksheet SP4B which indicates the combined CMF value.

Worksheet SP4B. Crash Modification Factors for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
CMF for Left-Turn Lanes	**CMF for Left-Turn Signal Phasing**	**CMF for Right-Turn Lanes**	**CMF for Right-Turn-on-Red**	**CMF for Lighting**	**CMF for Red-Light Cameras**	**Combined CMF**
CMF_{1i}	CMF_{2i}	CMF_{3i}	CMF_{4i}	CMF_{5i}	CMF_{6i}	CMF_{comb}
from Table 12-24	from Table 12-25	from Table 12-26	from Equation 12-35	from Equation 12-36	from Equation 12-37	(1)*(2)*(3)*(4)*(5)*(6)
0.81	0.98	0.92	1.00	0.91	1.00	0.66

- **Worksheet SP4C—Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections**

The SPF for multiple-vehicle collisions at the intersection in Sample Problem 4 is calculated using Equation 12-22 and entered into Column 4 of Worksheet SP4C. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 4 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal-and-injury (FI) and property-damage-only (PDO) crashes sum to the total crashes as illustrated in Column 6. Column 7 represents the combined CMF (from Column 7 in Worksheet SP4B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle crashes using the values in Column 6, the combined CMF in Column 7, and the calibration factor in Column 8.

Worksheet SP4C. Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients			Overdispersion Parameter, k	Initial N_{bimv}		Adjusted N_{bimv}	Combined CMFs		Predicted N_{bimv}
	from Table 12-10									
Crash Severity Level	a	b	c	from Table 12-10	from Equation 12-22	Proportion of Total Crashes	$(4)_{total}$*(5)	(7) from Worksheet SP4B	Calibration Factor, C_i	(6)*(7)*(8)
Total	**−10.99**	**1.07**	**0.23**	**0.39**	**4.027**	**1.000**	**4.027**	**0.66**	**1.00**	**2.658**
Fatal and injury (FI)	−13.14	1.18	0.22	0.33	1.233	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.318	1.281	0.66	1.00	0.845
Property damage only (PDO)	−11.02	1.02	0.24	0.44	2.647	$(5)_{total}-(5)_{FI}$ 0.682	2.746	0.66	1.00	1.812

- **Worksheet SP4D—Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections**

Worksheet SP4D presents the default proportions for collision type (from Table 12-11) by crash severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for multiple-vehicle crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for multiple-vehicle crashes (from Column 9, Worksheet SP4C) into components by crash severity and collision type.

Worksheet SP4D. Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted** $N_{binmv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{binmv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{binmv\ (total)}$ **(crashes/year)**
Collision Type	**from Table 12-11**	**(9)**$_{FI}$ **from Worksheet SP4C**	**from Table 12-11**	**(9)**$_{PDO}$ **from Worksheet SP4C**	**(9)**$_{PDO}$ **from Worksheet SP4C**
Total	1.000	0.845 (2)*(3)$_{FI}$	1.000	1.812 (4)*(5)$_{PDO}$	2.658 (3)+(5)
Rear-end collision	0.450	0.380	0.483	0.875	1.255
Head-on collision	0.049	0.041	0.030	0.054	0.095
Angle collision	0.347	0.293	0.244	0.442	0.735
Sideswipe	0.099	0.084	0.032	0.058	0.142
Other multiple-vehicle collision	0.055	0.046	0.211	0.382	0.428

- **Worksheet SP4E—Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Intersections**

The SPF for single-vehicle crashes at the intersection in Sample Problem 4 is calculated using Equation 12-25 for total and property-damage-only (PDO) crashes and entered into Column 4 of Worksheet SP4E. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2, and 3; however, the overdispersion parameter is not needed for Sample Problem 4 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal-and-injury (FI) and property-damage-only (PDO) crashes sum to the total crashes as illustrated in Column 6. Column 7 represents the combined CMF (from Column 7 in Worksheet SP4B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of single-vehicle crashes using the values in Column 6, the combined CMF in Column 7, and the calibration factor in Column 8.

Worksheet SP4E. Single-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients			Overdispersion Parameter, k	Initial N_{bisv}		Adjusted N_{bisv}	Combined CMFs		Predicted N_{bisv}
	from Table 12-12									
Crash Severity Level	a	b	c	from Table 12-12	from Equation 12-25; (FI) from Equation 12-25 or 12-27	Proportion of Total Crashes	$(4)_{total}$*(5)	(7) from Worksheet SP4B	Calibration Factor, C_i	(6)*(7)*(8)
Total	−10.21	0.68	0.27	0.36	0.297	1.000	0.297	0.66	1.000	0.196
Fatal and injury (FI)	−9.25	0.43	0.29	0.09	0.084	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.287	0.085	0.66	1.000	0.056
Property damage only (PDO)	−11.34	0.78	0.25	0.44	0.209	$(5)_{total}-(5)_{FI}$ 0.713	0.212	0.66	1.000	0.140

- **Worksheet SP4F—Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Intersections**

Worksheet SP4F presents the default proportions for collision type (from Table 12-13) by crash severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for single-vehicle crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for single-vehicle crashes (from Column 9, Worksheet SP4E) into components by crash severity and collision type.

Worksheet SP4F. Single-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted** $N_{bisv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{bisv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{bisv\ (total)}$ **(crashes/year)**
Collision Type	**Table 12-13**	**(9)**$_{FI}$ **from Worksheet SP4E**	**Table 12-13**	**(9)**$_{PDO}$ **from Worksheet SP4E**	**(9)**$_{PDO}$ **from Worksheet SP4E**
Total	1.000	0.056 (2)*(3)$_{FI}$	1.000	0.140 (4)*(5)$_{PDO}$	0.196 (3)+(5)
Collision with parked vehicle	0.001	0.000	0.001	0.000	0.000
Collision with animal	0.002	0.000	0.002	0.000	0.000
Collision with fixed object	0.744	0.042	0.870	0.122	0.164
Collision with other object	0.072	0.004	0.070	0.010	0.014
Other single-vehicle collision	0.040	0.002	0.023	0.003	0.005
Single-vehicle noncollision	0.141	0.008	0.034	0.005	0.013

- **Worksheet SP4H—Crash Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections**

In Step 10 of the predictive method, crash modification factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the CMF values for vehicle-pedestrian collision. Once the value for each CMF has been determined, all of the CMFs are multiplied together in Column 4 of Worksheet SP4H which indicates the combined CMF value for vehicle-pedestrian collisions.

Worksheet SP4H. Crash Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

(1)	(2)	(3)	(4)
CMF for Bus Stops	**CMF for Schools**	**CMF for Alcohol Sales Establishments**	**Combined CMF**
CMF_{1p}	CMF_{2p}	CMF_{3p}	(1)*(2)*(3)
from Table 12-28	from Table 12-29	from Table 12-30	
2.78	1.35	1.12	4.20

▪ Worksheet SP4I—Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

The predicted number of vehicle-pedestrian collisions per year for base conditions at a signalized intersection, Npedbase, is calculated using Equation 12-30 and entered into Column 4 of Worksheet SP4I. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 4 (as the EB Method is not utilized). Column 5 represents the combined CMF for vehicle-pedestrian collisions (from Column 4 in Worksheet SP4H), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency of vehicle-pedestrian collisions using the values in Column 4, the combined CMF in Column 5, and the calibration factor in Column 6. Since all vehicle-pedestrian crashes are assumed to involve some level of injury, there are no property-damage-only crashes.

Worksheet SP4I. Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

(1)	(2)					(3)	(4)	(5)	(6)	(7)
	SPF Coefficients						$N_{pedbase}$	**Combined CMF**		**Predicted** N_{pedi}
	from Table 12-14									
Crash Severity Level	**a**	**b**	**c**	**d**	**e**	**Overdispersion Parameter, *k***	**from Equation 12-30**	**(4) from Worksheet SP4H**	**Calibration Factor, C_i**	**(8)*(9)*(10)**
Total	−9.53	0.40	0.26	0.45	0.04	0.24	0.113	4.20	1.00	0.475
Fatal and injury (FI)	—	—	—	—	—	—	—	—	1.00	0.475

▪ Worksheet SP4J—Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

The predicted average crash frequency of multiple-vehicle predicted crashes and single-vehicle predicted crashes from Worksheets SP4C and SP4E are entered into Columns 2 and 3 respectively. These values are summed in Column 4. Column 5 contains the bicycle crash adjustment factor (see Table 12-17). Column 6 presents the calibration factor. The predicted average crash frequency of vehicle-bicycle collision (Column 7) is the product of Columns 4, 5, and 6. Since all vehicle-bicycle crashes are assumed to involve some level of injury, there are no property-damage-only crashes.

Worksheet SP4J. Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Predicted N_{bimv}	Predicted N_{bisv}	Predicted N_{bi}	f_{bikei}		Predicted N_{pedi}
Crash Severity Level	(9) from Worksheet SP4C	(9) from Worksheet SP4E	(2)+(3)	from Table 12-17	Calibration Factor, C_i	(4)*(5)*(6)
Total	2.658	0.196	2.854	0.015	1.00	0.043
Fatal and injury (FI)	—	—	—	—	1.00	0.043

▪ Worksheet SP4K—Crash Severity Distribution for Urban and Suburban Arterial Intersections

Worksheet SP4K provides a summary of all collision types by severity level. Values from Worksheets SP4D, SP4F, SP4I, and SP4J are presented and summed to provide the predicted average crash frequency for each severity level as follows:

- Fatal-and-injury crashes (Column 2)
- Property-damage-only crashes (Column 3)
- Total crashes (Column 4)

Worksheet SP4K. Crash Severity Distribution for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)
	Fatal and Injury (FI)	**Property Damage Only (PDO)**	**Total**
Collision Type	**(3) from Worksheets SP4D and SP4F; (7) from SP4I and SP4J**	**(5) from Worksheets SP4D and SP4F**	**(6) from Worksheets SP4D and SP4F; (7) from SP4I and SP4J**
MULTIPLE-VEHICLE COLLISIONS			
Rear-end collisions (from Worksheet SP4D)	0.380	0.875	1.255
Head-on collisions (from Worksheet SP4D)	0.041	0.054	0.095
Angle collisions (from Worksheet SP4D)	0.293	0.442	0.735
Sideswipe (from Worksheet SP4D)	0.084	0.058	0.142
Other multiple-vehicle collision (from Worksheet SP4D)	0.046	0.382	0.428
Subtotal	0.844	1.811	2.655
SINGLE-VEHICLE COLLISIONS			
Collision with parked vehicle (from Worksheet SP4F)	0.000	0.000	0.000
Collision with animal (from Worksheet SP4F)	0.000	0.000	0.000
Collision with fixed object (from Worksheet SP4F)	0.042	0.122	0.164
Collision with other object (from Worksheet SP4F)	0.004	0.010	0.014
Other single-vehicle collision (from Worksheet SP4F)	0.002	0.003	0.005
Single-vehicle noncollision (from Worksheet SP4F)	0.008	0.005	0.013
Collision with pedestrian (from Worksheet SP4I)	0.475	0.000	0.475
Collision with bicycle (from Worksheet SP4J)	0.043	0.000	0.043
Subtotal	0.574	0.140	0.714
Total	1.418	1.951	3.369

- **Worksheet SP4L—Summary Results for Urban and Suburban Arterial Intersections**

Worksheet SP4L presents a summary of the results.

Worksheet SP4L. Summary Results for Urban and Suburban Arterial Intersections

(1)	(2)
	Predicted Average Crash Frequency, $N_{predicted\ int}$ (crashes/year)
Crash Severity Level	**(Total) from Worksheet SP4K**
Total	3.369
Fatal and injury (FI)	1.418
Property damage only (PDO)	1.951

Sample Problem 5

The Project
A project of interest consists of four sites located on an urban arterial: a three-lane TWLTL segment; a four-lane divided segment; a three-leg intersection with minor-road stop control; and a four-leg signalized intersection. (This project is a compilation of roadway segments and intersections from Sample Problems 1 through 4.)

- **The Question**

What is the expected crash frequency of the project for a particular year incorporating both the predicted crash frequencies from Sample Problems 1 through 4 and the observed crash frequencies using the site-specific EB Method?

- **The Facts**

- 2 roadway segments (3T segment, 4D segment)
- 2 intersections (3ST intersection, 4SG intersection)
- 34 observed crashes (3T segment: 7 multiple-vehicle nondriveway, 4 single-vehicle, 2 multiple-vehicle driveway related; 4D: 6 multiple-vehicle nondriveway, 3 single-vehicle, 1 multiple-vehicle driveway related; 3ST: 2 multiple-vehicle, 3 single-vehicle; 4SG 6 multiple-vehicle, 0 single-vehicle)

- **Outline of Solution**

To calculate the expected average crash frequency, site-specific observed crash frequencies are combined with predicted crash frequencies for the project using the site-specific EB Method (i.e., observed crashes are assigned to specific intersections or roadway segments) presented in Part C, Appendix A.2.4.

- **Results**

The expected average crash frequency for the project is 25.4 crashes per year (rounded to one decimal place).

WORKSHEETS

To apply the site-specific EB Method to multiple roadway segments and intersections on an urban or suburban arterial combined, three worksheets are provided for determining the expected average crash frequency. The three worksheets include:

- *Worksheet SP5A (Corresponds to Worksheet 3A)*—Predicted Crashes by Collision and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials.

- *Worksheet SP5B (Corresponds to Worksheet 3B)*—Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials.
- *Worksheet SP5C (Corresponds to Worksheet 3C)*—Site-Specific EB Method Summary Results for Urban and Suburban Arterials

Details of these sample problem worksheets are provided below. Blank versions of the corresponding worksheets are provided in Appendix 12A.

- **Worksheets SP5A—Predicted Crashes by Collision and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials.**

The predicted average crash frequencies by severity level and collision type determined in Sample Problems 1 through 4 are entered into Columns 2 through 4 of Worksheet SP5A. Column 5 presents the observed crash frequencies by site and collision type, and Column 6 presents the overdispersion parameters. The expected average crash frequency is calculated by applying the site-specific EB Method which considers both the predicted model estimate and observed crash frequencies for each roadway segment and intersection. Equation A-5 from Part C, Appendix A is used to calculate the weighted adjustment and entered into Column 7. The expected average crash frequency is calculated using Equation A-4 and entered into Column 8. Detailed calculation of Columns 7 and 8 are provided below.

Worksheet SP5A. Predicted Crashes by Collision and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Collision Type/Site Type	**Predicted Average Crash Frequency (crashes/year)**			**Observed Crashes, $N_{observed}$ (crashes/year)**	**Overdispersion Parameter, k**	**Weighted Adjustment, w**	**Expected Average Crash Frequency, $N_{expected\ (vehicle)}$**
	$N_{predicted\ (total)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			**Equation A-5**	**Equation A-4**
ROADWAY SEGMENTS							
Multiple-Vehicle Nondriveway							
Segment 1	4.967	1.196	3.771	7	0.66	0.234	6.524
Segment 2	2.524	0.702	1.822	6	1.32	0.231	5.197
Single-Vehicle							
Segment 1	1.182	0.338	0.844	4	1.37	0.382	2.924
Segment 2	0.485	0.085	0.401	3	0.86	0.706	1.224
Multiple-Vehicle Driveway-Related							
Segment 1	0.734	0.179	0.555	2	1.10	0.553	1.300
Segment 2	0.149	0.042	0.107	1	1.39	0.828	0.295
INTERSECTIONS							
Multiple-Vehicle							
Intersection 1	1.268	0.405	0.862	2	0.80	0.496	1.637
Intersection 2	2.658	0.845	1.812	6	0.39	0.491	4.359

Single-Vehicle							
Intersection 1	0.234	0.072	0.162	3	1.14	0.789	0.818
Intersection 2	0.196	0.056	0.140	0	0.36	0.934	0.183
Combined (Sum of Column)	14.397	3.920	10.476	34	—	—	24.461

Column 7—Weighted Adjustment

The weighted adjustment, *w*, to be placed on the predictive model estimate is calculated using Equation A-5 as follows:

$$w = \frac{1}{1 + k \times \left(\sum_{\substack{\text{all study} \\ \text{years}}} N_{\text{predicted}} \right)}$$

Multiple-Vehicle Nondriveway Collisions

Segment 1

$$w = \frac{1}{1 + 0.66 \times (4.967)} = 0.234$$

Segment 2

$$w = \frac{1}{1 + 1.32 \times (2.524)} = 0.231$$

Single-Vehicle Crashes

Segment 1

$$w = \frac{1}{1 + 1.37 \times (1.182)} = 0.382$$

Segment 2

$$w = \frac{1}{1 + 0.86 \times (0.485)} = 0.706$$

Multiple-Vehicle Driveway Related Collisions

Segment 1

$$w = \frac{1}{1 + 1.10 \times (0.734)} = 0.553$$

Segment 2

$$w = \frac{1}{1 + 1.39 \times (0.149)} = 0.828$$

Multiple-Vehicle Collisions

Intersection 1

$$w = \frac{1}{1 + 0.80 \times (1.268)} = 0.496$$

Intersection 2

$$w = \frac{1}{1 + 0.39 \times (2.658)} = 0.491$$

Single-Vehicle Crashes

Intersection 1

$$w = \frac{1}{1 + 1.149 \times (0.234)} = 0.789$$

Intersection 2

$$w = \frac{1}{1 + 0.36 \times (0.196)} = 0.934$$

Column 8—Expected Average Crash Frequency
The estimate of expected average crash frequency, N_{expected}, is calculated using Equation A-4 as follows:

$$N_{\text{expected}} = w \times N_{\text{predicted}} + (1 - w) \times N_{\text{observed}}$$

Multiple-Vehicle Nondriveway Collisions

Segment 1 $N_{\text{expected}} = 0.234 \times 4.967 + (1 - 0.234) \times 7 = 6.524$

Segment 2 $N_{\text{expected}} = 0.231 \times 2.524 + (1 - 0.231) \times 6 = 5.197$

Single-Vehicle Crashes

Segment 1 $N_{\text{expected}} = 0.382 \times 1.182 + (1 - 0.382) \times 4 = 2.924$

Segment 2 $N_{\text{expected}} = 0.706 \times 0.485 + (1 - 0.706) \times 3 = 1.224$

Multiple-Vehicle Driveway Related Collisions

Segment 1 $N_{\text{expected}} = 0.553 \times 0.734 + (1 - 0.553) \times 2 = 1.300$

Segment 2 $N_{\text{expected}} = 0.828 \times 0.149 + (1 - 0.828) \times 1 = 0.295$

Multiple-Vehicle Collisions

Intersection 1 $N_{\text{expected}} = 0.496 \times 1.268 + (1 - 0.496) \times 2 = 1.637$

Intersection 2 $N_{\text{expected}} = 0.491 \times 2.658 + (1 - 0.491) \times 6 = 4.359$

Single-Vehicle Crashes

Intersection 1 $N_{\text{expected}} = 0.789 \times 0.234 + (1 - 0.789) \times 3 = 0.818$

Intersection 2 $N_{\text{expected}} = 0.934 \times 0.196 + (1 - 0.934) \times 0 = 0.183$

▪ Worksheets SP5B—Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

Worksheet SP5B provides a summary of the vehicle-pedestrian and vehicle-bicycle crashes determined in Sample Problems 1 through 4.

Worksheet SP5B. Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

(1)	(2)	(3)
Site Type	N_{ped}	N_{bike}
ROADWAY SEGMENTS		
Segment 1	0.089	0.048
Segment 2	0.212	0.041
INTERSECTIONS		
Intersection 1	0.032	0.024
Intersection 2	0.475	0.043
Combined (Sum of Column)	0.808	0.156

▪ Worksheets SP5C—Site-Specific EB Method Summary Results for Urban and Suburban Arterials

Worksheet SP5C presents a summary of the results. Column 5 calculates the expected average crash frequency by severity level for vehicle crashes only by applying the proportion of predicted average crash frequency by severity level (Column 2) to the expected average crash frequency calculated using the site-specific EB Method. Column 6 calculates the total expected average crash frequency by severity level using the values in Column 3, 4, and 5.

Worksheet SP5C. Site-Specific EB Method Summary Results for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)
Crash Severity Level	$N_{predicted}$	N_{ped}	N_{bike}	$N_{expected\ (vehicle)}$	$N_{expected}$
Total	$(2)_{comb}$ Worksheet SP5A	$(2)_{comb}$ Worksheet SP5B	$(3)_{comb}$ Worksheet SP5B	$(13)_{comb}$ Worksheet SP5A	(3)+(4)+(5)
	14.397	0.808	0.156	24.461	25.4
Fatal and injury (FI)	$(3)_{comb}$ Worksheet SP5A	$(2)_{comb}$ Worksheet SP5B	$(3)_{comb}$ Worksheet SP5B	$(5)_{total}*(2)_{FI}/(2)total$	(3)+(4)+(5)
	3.920	0.808	0.156	6.660	7.6
Property damage only (PDO)	$(4)_{comb}$ Worksheet SP5A	—	—	$(5)_{total}*(2)_{PDO}/(2)_{total}$	(3)+(4)+(5)
	10.476	0.000	0.000	17.800	17.8

Sample Problem 6

The Project
A project of interest consists of four sites located on an urban arterial: a three-lane TWLTL segment; a four-lane divided segment; a three-leg intersection with minor-road stop control; and a four-leg signalized intersection. (This project is a compilation of roadway segments and intersections from Sample Problems 1 through 4.)

The Question
What is the expected average crash frequency of the project for a particular year incorporating both the predicted average crash frequencies from Sample Problems 1 through 4 and the observed crash frequencies using the **project-level EB Method?**

The Facts

- 2 roadway segments (3T segment, 4D segment)
- 2 intersection (3ST intersection, 4SG intersection)
- 34 observed crashes (but no information is available to attribute specific crashes to specific sites)

Outline of Solution
Observed crash frequencies for the project as a whole are combined with predicted average crash frequencies for the project as a whole using the project-level EB Method (i.e., observed crash data for individual roadway segments and intersections are not available, but observed crashes are assigned to a facility as a whole) presented in Part C, Appendix A.2.5.

Results
The expected average crash frequency for the project is 26.0 crashes per year (rounded to one decimal place).

WORKSHEETS

To apply the project-level EB Method to multiple roadway segments and intersections on an urban or suburban arterial combined, three worksheets are provided for determining the expected average crash frequency. The three worksheets include:

- *Worksheet SP6A (Corresponds to Worksheet 4A)*—Predicted Crashes by Collision and Site Type and Observed Crashes Using the Project-Level EB Method for Urban and Suburban Arterials
- *Worksheet SP6B (Corresponds to Worksheet 4B)*—Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials
- *Worksheet SP6C (Corresponds to Worksheet 4C)*—Project-EB Method Summary Results for Urban and Suburban Arterials

Details of these sample problem worksheets are provided below. Blank versions of the corresponding worksheets are provided in Appendix 12A.

▪ Worksheets SP6A—Predicted Crashes by Collision and Site Type and Observed Crashes Using the Project-Level EB Method for Urban and Suburban Arterials

The predicted average crash frequencies by severity level and collision type, excluding vehicle-pedestrian and vehicle-bicycle collisions, determined in Sample Problems 1 through 4 are entered in Columns 2 through 4 of Worksheet SP6A. Column 5 presents the total observed crash frequencies combined for all sites, and Column 6 presents the overdispersion parameters. The expected average crash frequency is calculated by applying the project-level EB Method which considers both the predicted model estimate for each roadway segment and intersection and the project observed crashes. Column 7 calculates N_{w0}, and Column 8 calculates N_{w1}. Equations A-10 through A-14 from Part C, Appendix A are used to calculate the expected average crash frequency of combined sites. The results obtained from each equation are presented in Columns 9 through 14. Part C, Appendix A.2.5 defines all the variables used in this worksheet. Detailed calculations of Columns 9 through 13 are provided below.

Worksheet SP6A. Predicted Crashes by Collision and Site Type and Observed Crashes Using the Project-Level EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	Predicted Crashes					$N_{predicted\ w0}$	$N_{predicted\ w1}$	w_0	N_0	w_1	N_1	$N_{expected/comb}$ (vehicle)
Collision Type/Site Type	$N_{predicted}$ (total)	$N_{predicted}$ (*FI*)	$N_{predicted\ (PDO)}$	Observed Crashes, $N_{observed}$ (crashes/year)	Overdispersion Parameter, *k*	Equation A-8 (6)*(2)²	Equation A-9 (sqrt((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS												
Multiple-Vehicle Nondriveway												
Segment 1	4.967	1.196	3.771	—	0.66	16.283	1.811	—	—	—	—	—
Segment 2	2.524	0.702	1.822	—	1.32	8.409	1.825	—	—	—	—	—
Single-Vehicle												
Segment 1	1.182	0.338	0.844	—	1.37	1.914	1.273	—	—	—	—	—
Segment 2	0.485	0.085	0.401	—	0.86	0.202	0.646	—	—	—	—	—
Multiple-Vehicle Driveway-Related												
Segment 1	0.734	0.179	0.555	—	1.10	0.593	0.899	—	—	—	—	—
Segment 2	0.149	0.042	0.107	—	1.39	0.031	0.455	—	—	—	—	—
INTERSECTIONS												
Multiple-Vehicle												
Intersection 1	1.268	0.405	0.862	—	0.80	1.286	1.007	—	—	—	—	—
Intersection 2	2.658	0.845	1.812	—	0.39	2.755	1.018	—	—	—	—	—
Single-Vehicle												
Intersection 1	0.234	0.072	0.162	—	1.14	0.062	0.516	—	—	—	—	—

Intersection 2	0.196	0.056	0.140	—	0.36	0.014	0.266	—	—	—	—	—
Combined (Sum of Column)	14.397	3.920	10.476	34	—	31.549	9.716	0.313	27.864	0.597	22.297	25.080

Note: $N_{\text{predicted } w0}$ = Predicted number of total crashes assuming that crash frequencies are statistically independent

$$N_{\text{predicted } w0} = \sum_{j=1}^{5} k_{rmj} N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj} N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj} N_{rdj}^2 + \sum_{j=1}^{4} k_{imj} N_{imj}^2 + \sum_{j=1}^{4} k_{isj} N_{isj}^2 \quad \text{(A-8)}$$

$N_{\text{predicted } w1}$ = Predicted number of total crashes assuming that crash frequencies are perfectly correlated

$$N_{\text{predicted } w1} = \sum_{j=1}^{5} \sqrt{k_{rmj} N_{rmj}} + \sum_{j=1}^{5} \sqrt{k_{rsj} N_{rsj}} + \sum_{j=1}^{5} \sqrt{k_{rdj} N_{rdj}} + \sum_{j=1}^{4} \sqrt{k_{imj} N_{imj}} + \sum_{j=1}^{4} \sqrt{k_{isj} N_{isj}} \quad \text{(A-9)}$$

Column 9—w_0

The weight placed on predicted crash frequency under the assumption that crashes frequencies for different roadway elements are statistically independent, w_0, is calculated using Equation A-10 as follows:

$$\begin{aligned} w_0 &= \frac{1}{1 + \frac{N_{\text{predicted } w0}}{N_{\text{predicted (total)}}}} \\ &= \frac{1}{1 + \frac{31.549}{14.397}} \\ &= 0.313 \end{aligned}$$

Column 10—N_0

The expected crash frequency based on the assumption that different roadway elements are statistically independent, N_0, is calculated using Equation A-11 as follows:

$$\begin{aligned} N_0 &= w_0 \times N_{\text{predicted(total)}} + (1 - w_0) \times N_{\text{observed(total)}} \\ &= 0.313 \times 14.397 + (1 - 0.313) \times 34 \\ &= 27.864 \end{aligned}$$

Column 11—w_1

The weight placed on predicted crash frequency under the assumption that crashes frequencies for different roadway elements are perfectly correlated, w_1, is calculated using Equation A-12 as follows:

$$\begin{aligned} w_1 &= \frac{1}{1 + \frac{N_{\text{predicted } w1}}{N_{\text{predicted (total)}}}} \\ &= \frac{1}{1 + \frac{9.716}{14.397}} \\ &= 0.597 \end{aligned}$$

Column 12—N_1

The expected crash frequency based on the assumption that different roadway elements are perfectly correlated, N_1, is calculated using Equation A-13 as follows:

$$\begin{aligned} N_1 &= w_1 \times N_{\text{predicted (total)}} + (1 - w_1) \times N_{\text{observed (total)}} \\ &= 0.597 \times 14.397 + (1 - 0.597) \times 34 \\ &= 22.297 \end{aligned}$$

Column 13—$N_{expected/comb}$

The expected average crash frequency based of combined sites, $N_{expected/comb}$, is calculated using Equation A-14 as follows:

$$N_{expected/comb} = \frac{N_0 + N_1}{2}$$
$$= \frac{27.864 + 22.297}{2}$$
$$= 25.080$$

▪ Worksheets SP6B—Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

Worksheet SP6B provides a summary of the vehicle-pedestrian and vehicle-bicycle crashes determined in Sample Problems 1 through 4.

Worksheet SP6B. Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

(1)	(2)	(3)
Site Type	N_{ped}	N_{bike}
ROADWAY SEGMENTS		
Segment 1	0.089	0.048
Segment 2	0.212	0.041
INTERSECTIONS		
Intersection 1	0.032	0.024
Intersection 2	0.475	0.043
Combined (Sum of Column)	0.808	0.156

▪ Worksheets SP6C—Project-Level EB Method Summary Results for Urban and Suburban Arterials

Worksheet SP6C presents a summary of the results. Column 5 calculates the expected average crash frequency by severity level for vehicle crashes only by applying the proportion of predicted average crash frequency by severity level (Column 2) to the expected average crash frequency calculated using the project-level EB Method. Column 6 calculates the total expected average crash frequency by severity level using the values in Column 3, 4, and 5.

Worksheet SP6C. Project-Level EB Method Summary Results for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)
Crash Severity Level	$N_{predicted}$	N_{ped}	N_{bike}	$N_{expected/comb\ (vehicle)}$	$N_{expected}$
Total	$(2)_{comb}$ Worksheet SP6A	$(2)_{comb}$ Worksheet SP6B	$(3)_{comb}$ Worksheet SP6B	$(13)_{comb}$ Worksheet SP6A	(3)+(4)+(5)
	14.397	0.808	0.156	25.080	26.0
Fatal and injury (FI)	$(3)_{comb}$ Worksheet SP6A	$(2)_{comb}$ Worksheet SP6B	$(3)_{comb}$ Worksheet SP6B	$(5)_{total}*(2)_{FI}/(2)_{total}$	(3)+(4)+(5)
	3.920	0.808	0.156	6.829	7.8
Property damage only (PDO)	$(4)_{comb}$ Worksheet SP6A	—	—	$(5)_{total}*(2)_{PDO}/(2)_{total}$	(3)+(4)+(5)
	10.476	0.000	0.000	18.250	18.3

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APPENDIX 12A. WORKSHEETS FOR PREDICTIVE METHOD FOR URBAN AND SUBURBAN ARTERIALS

Worksheet 1A. General Information and Input Data for Urban and Suburban Roadway Segments

General Information	**Location Information**	
Analyst	Roadway	
Agency or Company	Roadway Section	
Date Performed	Jurisdiction	
	Analysis Year	
Input Data	**Base Conditions**	**Site Conditions**
Road type (2U, 3T, 4U, 4D, 5T)	—	
Length of segment, *L* (mi)	—	
AADT (veh/day)	—	
Type of on-street parking (none/parallel/angle)	none	
Proportion of curb length with on-street parking	—	
Median width (ft)	15	
Lighting (present / not present)	not present	
Auto speed enforcement (present/not present)	not present	
Major commercial driveways (number)	—	
Minor commercial driveways (number)	—	
Major industrial/institutional driveways (number)	—	
Minor industrial/institutional driveways (number)	—	
Major residential driveways (number)	—	
Minor residential driveways (number)	—	
Other driveways (number)	—	
Speed Category	—	
Roadside fixed object density (fixed objects/mi)	not present	
Offset to roadside fixed objects (ft)	not present	
Calibration Factor, C_r	1.0	

Worksheet 1B. Crash Modification Factors for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
CMF for On-Street Parking	**CMF for Roadside Fixed Objects**	**CMF for Median Width**	**CMF for Lighting**	**CMF for Auto Speed Enforcement**	**Combined CMF**
CMF_{1r}	CMF_{2r}	CMF_{3r}	CMF_{4r}	CMF_{5r}	CMF_{comb}
from Equation 12-32	from Equation 12-33	from Table 12-22	from Equation 12-34	from Section 12.7.1	(1)*(2)*(3)*(4)*(5)

Worksheet 1C. Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients		**Overdispersion Parameter, *k***	**Initial N_{brmv}**		**Adjusted N_{brmv}**	**Combined CMFs**	**Calibration Factor**	**Predicted N_{brmv}**
Crash Severity Level	**from Table 12-3**								
	a	**b**	**from Table 12-3**	**from Equation 12-10**	**Proportion of Total Crashes**	**$(4)_{total}$*(5)**	**(6) from Worksheet 1B**	C_r	**(6)*(7)*(8)**
Total									
Fatal and injury (FI)					$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)					$(5)_{total}-(5)_{FI}$				

Worksheet 1D. Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	Predicted $N_{brmv\ (FI)}$ (crashes/year)	Proportion of Collision Type $_{(PDO)}$	Predicted $N_{brmv\ (PDO)}$ (crashes/year)	Predicted $N_{brmv\ (total)}$ (crashes/year)
Collision Type	from Table 12-4	$(9)_{FI}$ from Worksheet 1C	from Table 12-4	$(9)_{PDO}$ from Worksheet 1C	$(9)_{total}$ from Worksheet 1C
Total	1.000	$(2)*(3)_{FI}$	1.000	$(4)*(5)_{PDO}$	(3)+(5)
Rear-end collision					
Head-on collision					
Angle collision					
Sideswipe, same direction					
Sideswipe, opposite direction					
Other multiple-vehicle collision					

Worksheet 1E. Single-Vehicle Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients		Overdispersion Parameter, k	Initial N_{brsv}		Adjusted N_{brsv}	Combined CMFs	Calibration Factor	Predicted N_{brsv}
	from Table 12-5								
Crash Severity Level	a	b	from Table 12-5	from Equation 12-13	Proportion of Total Crashes	$(4)_{total}*(5)$	(6) from Worksheet 1B	C_r	(6)*(7)*(8)
Total									
Fatal and injury (FI)					$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)					$(5)_{total}-(5)_{FI}$				

Worksheet 1F. Single-Vehicle Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted** $N_{brsv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{brsv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{brsv\ (total)}$ **(crashes/year)**
Collision Type	**from Table 12-6**	**(9)**$_{FI}$ **from Worksheet 1E**	**from Table 12-6**	**(9)**$_{PDO}$ **from Worksheet 1E**	**(9)**$_{total}$ **from Worksheet 1E**
Total	1.000	(2)*(3)$_{FI}$	1.000	(4)*(5)$_{PDO}$	(3)+(5)
Collision with animal					
Collision with fixed object					
Collision with other object					
Other single-vehicle collision					

Worksheet 1G. Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
		Crashes per Driveway per Year, N_j	**Coefficient for Traffic Adjustment,** t	**Initial** N_{brdwy}	**Overdispersion Parameter,** k
Driveway Type	**Number of Driveways,** n_j	**from Table 12-7**	**from Table 12-7**	**Equation 12-16** n_j*N_j*(AADT/15,000)t	**from Table 12-7**
Major commercial					—
Minor commercial					
Major industrial/institutional					
Minor industrial/institutional					
Major residential					
Minor residential					
Other					
Total	—	—	—		

Worksheet 1H. Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Initial N_{brdwy}	**Proportion of Total Crashes (f_{dwy})**	**Adjusted N_{brdwy}**	**Combined CMFs**		**Predicted N_{brdwy}**
Crash Severity Level	**$(5)_{total}$ from Worksheet 1G**	**from Table 12-7**	**$(2)_{total}$ *(3)**	**(6) from Worksheet 1B**	**Calibration Factor, C_r**	**(4)*(5)*(6)**
Total						
Fatal and injury (FI)	—					
Property damage only (PDO)	—					

Worksheet 1I. Vehicle-Pedestrian Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Predicted N_{brmv}	**Predicted N_{brsv}**	**Predicted N_{brdwy}**	**Predicted N_{br}**	**f_{pedr}**		**Predicted N_{pedr}**
Crash Severity Level	**(9) from Worksheet 1C**	**(9) from Worksheet 1E**	**(7) from Worksheet 1H**	**(2)+(3)+(4)**	**from Table 12-8**	**Calibration Factor, C_r**	**(5)*(6)*(7)**
Total							
Fatal and injury (FI)	—	—	—	—	—		

Worksheet 1J. Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Predicted N_{brmv}	**Predicted N_{brsv}**	**Predicted N_{brdwy}**	**Predicted N_{br}**	**f_{biker}**		**Predicted N_{biker}**
Crash Severity Level	(9) from Worksheet 1C	(9) from Worksheet 1E	(7) from Worksheet 1H	(2)+(3)+(4)	from Table 12-9	**Calibration Factor, C_r**	(5)*(6)*(7)
Total							
Fatal and injury	—	—	—	—	—		

Worksheet 1K. Crash Severity Distribution for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
	Fatal and Injury (FI)	**Property Damage Only (PDO)**	**Total**
Collision Type	**(3) from Worksheets 1D and 1F; (7) from Worksheet 1H; and (8) from Worksheets 1I and 1J**	**(5) from Worksheets 1D and 1F; and (7) from Worksheet 1H**	**(6) from Worksheets 1D and 1F; (7) from Worksheet 1H; and (8) from Worksheets 1I and 1J**
MULTIPLE-VEHICLE			
Rear-end collisions (from Worksheet 1D)			
Head-on collisions (from Worksheet 1D)			
Angle collisions (from Worksheet 1D)			
Sideswipe, same direction (from Worksheet 1D)			
Sideswipe, opposite direction (from Worksheet 1D)			
Driveway-related collisions (from Worksheet 1H)			
Other multiple-vehicle collision (from Worksheet 1D)			
Subtotal			
SINGLE-VEHICLE			
Collision with animal (from Worksheet 1F)			
Collision with fixed object (from Worksheet 1F)			
Collision with other object (from Worksheet 1F)			
Other single-vehicle collision (from Worksheet 1F)			
Collision with pedestrian (from Worksheet 1I)			
Collision with bicycle (from Worksheet 1J)			
Subtotal			
Total			

Worksheet 1L. Summary Results for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
Crash Severity Level	Predicted Average Crash Frequency, $N_{predicted\ rs}$ (crashes/year) (total) from Worksheet 1K	Roadway Segment Length, L (mi)	Crash Rate (crashes/mi/year) (2)/(3)
Total			
Fatal and injury (FI)			
Property damage only (PDO)			

Worksheet 2A. General Information and Input Data for Urban and Suburban Arterial Intersections

General Information		Location Information	
Analyst		Roadway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	Base Conditions	Site Conditions
Intersection type (3ST, 3SG, 4ST, 4SG)	—	
$AADT_{maj}$ (veh/day)	—	
$AADT_{min}$ (veh/day)	—	
Intersection lighting (present/not present)	not present	
Calibration factor, C_i	1.00	
Data for unsignalized intersections only:	—	
Number of major-road approaches with left-turn lanes (0, 1, 2)	0	
Number of major-road approaches with right-turn lanes (0, 1, 2)	0	
Data for signalized intersections only:	—	
Number of approaches with left-turn lanes (0, 1, 2, 3, 4)	0	
Number of approaches with right-turn lanes (0, 1, 2, 3, 4)	0	
Number of approaches with left-turn signal phasing	—	
Number of approaches with right-turn-on-red prohibited	0	
Type of left-turn signal phasing	permissive	
Intersection red-light cameras (present/not present)	not present	
Sum of all pedestrian crossing volumes (PedVol)	—	
Maximum number of lanes crossed by a pedestrian (n_{lanesx})	—	
Number of bus stops within 300 m (1,000 ft) of the intersection	0	

Schools within 300 m (1,000 ft) of the intersection (present/not present)	not present	
Number of alcohol sales establishments within 300 m (1,000 ft) of the intersection	0	

Worksheet 2B. Crash Modification Factors for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
CMF for Left-Turn Lanes	**CMF for Left-Turn Signal Phasing**	**CMF for Right-Turn Lanes**	**CMF for Right-Turn-on-Red**	**CMF for Lighting**	**CMF for Red-Light Cameras**	**Combined CMF**
CMF_{1i}	CMF_{2i}	CMF_{3i}	CMF_{4i}	CMF_{5i}	CMF_{6i}	CMF_{comb}
from Table 12-24	from Table 12-25	from Table 12-26	from Equation 12-35	from Equation 12-36	from Equation 12-37	(1)*(2)*(3)*(4)*(5)*(6)

Worksheet 2C. Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients			Overdispersion Parameter, k	Initial N_{bimv}		Adjusted N_{bimv}	Combined CMFs		Predicted N_{bimv}
	from Table 12-10									
Crash Severity Level	a	b	c	from Table 12-10	from Equation 12-22	Proportion of Total Crashes	$(4)_{total}$*(5)	(7) from Worksheet 2B	Calibration Factor, C_i	(6)*(7)*(8)
Total										
Fatal and injury (FI)						$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)						$(5)_{total}-(5)_{FI}$				

Worksheet 2D. Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted** $N_{bimv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{bimv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{bimv\ (total)}$ **(crashes/year)**
Collision Type	**from Table 12-11**	**(9)**$_{FI}$ **from Worksheet 2C**	**from Table 12-11**	**(9)**$_{PDO}$ **from Worksheet 2C**	**(9)**$_{PDO}$ **from Worksheet 2C**
Total	1.000	(2)*(3)$_{FI}$	1.000	(4)*(5)$_{PDO}$	(3)+(5)
Rear-end collision					
Head-on collision					
Angle collision					
Sideswipe					
Other multiple-vehicle collision					

Worksheet 2E. Single-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
	SPF Coefficients			Overdispersion Parameter, k	Initial N_{bisv}		Adjusted N_{bisv}	Combined CMFs		Predicted N_{bisv}
	from Table 12-12									
Crash Severity Level	a	b	c	from Table 12-12	from Equation 12-25; (FI) from Equation 12-25 or 12-27	Proportion of Total Crashes	$(4)_{total}$*(5)	(7) from Worksheet 2B	Calibration Factor, C_i	(6)*(7)*(8)
Total										
Fatal and injury (FI)						$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)						$(5)_{total}-(5)_{FI}$				

Worksheet 2F. Single-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted** $N_{bisv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{bisv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{bisv\ (total)}$ **(crashes/year)**
Collision Type	**Table 12-13**	**(9)**$_{FI}$ **from Worksheet 2E**	**Table 12-13**	**(9)**$_{PDO}$ **from Worksheet 2E**	**(9)**$_{PDO}$ **from Worksheet 2E**
Total	1.000	$(2)*(3)_{FI}$	1.000	$(4)*(5)_{PDO}$	(3)+(5)
Collision with parked vehicle					
Collision with animal					
Collision with fixed object					
Collision with other object					
Other single-vehicle collision					
Single-vehicle noncollision					

Worksheet 2G. Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Predicted N_{bimv}	**Predicted** N_{bisv}	**Predicted** N_{bi}	f_{pedi}		**Predicted** N_{pedi}
Crash Severity Level	**(9) from Worksheet 2C**	**(9) from Worksheet 2E**	**(2)+(3)**	**from Table 12-16**	**Calibration Factor,** C_i	**(4)*(5)*(6)**
Total						
Fatal and injury (FI)	—	—	—	—		

Worksheet 2H. Crash Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

(1)	(2)	(3)	(4)
CMF for Bus Stops	**CMF for Schools**	**CMF for Alcohol Sales Establishments**	**Combined CMF**
CMF_{1p}	CMF_{2p}	CMF_{3p}	
from Table 12-28	**from Table 12-29**	**from Table 12-30**	**(1)*(2)*(3)**

Worksheet 2I. Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

(1)	(2)					(3)	(4)	(5)	(6)	(7)
	SPF Coefficients						$N_{pedbase}$	Combined CMF		Predicted N_{pedi}
	from Table 12-14									
Crash Severity Level	a	b	c	d	e	Overdispersion Parameter, k	from Equation 12-30	(4) from Worksheet 2H	Calibration Factor, C_i	(8)*(9)*(10)
Total										
Fatal and injury (FI)	—	—	—	—	—	—	—	—		

Worksheet 2J. Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Predicted N_{bimv}	Predicted N_{bisv}	Predicted N_{bi}	f_{bikei}		Predicted N_{pedi}
Crash Severity Level	(9) from Worksheet 2C	(9) from Worksheet 2E	(2)+(3)	from Table 12-17	Calibration Factor, C_i	(4)*(5)*(6)
Total						
Fatal and injury (FI)	—	—	—	—		

Worksheet 2K. Crash Severity Distribution for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)
	Fatal and Injury (FI)	**Property Damage Only (PDO)**	**Total**
Collision Type	**(3) from Worksheets 2D and 2F; (7) from Worksheets 2G or 2I and 2J**	**(5) from Worksheets 2D and 2F**	**(6) from Worksheets 2D and 2F; (7) from Worksheets 2G or 2I and 2J**
MULTIPLE-VEHICLE COLLISIONS			
Rear-end collisions (from Worksheet 2D)			
Head-on collisions (from Worksheet 2D)			
Angle collisions (from Worksheet 2D)			
Sideswipe (from Worksheet 2D)			
Other multiple-vehicle collision (from Worksheet 2D)			
Subtotal			
SINGLE-VEHICLE COLLISIONS			
Collision with parked vehicle (from Worksheet 2F)			
Collision with animal (from Worksheet 2F)			
Collision with fixed object (from Worksheet 2F)			
Collision with other object (from Worksheet 2F)			
Other single-vehicle collision (from Worksheet 2F)			
Single-vehicle noncollision (from Worksheet 2F)			
Collision with pedestrian (from Worksheet 2G or 2I)			
Collision with bicycle (from Worksheet 2J)			
Subtotal			
Total			

Worksheet 2L. Summary Results for Urban and Suburban Arterial Intersections

(1)	(2)
	Predicted Average Crash Frequency, $N_{predicted\ int}$ (crashes/year)
Crash Severity Level	**(Total) from Worksheet 2K**
Total	
Fatal and injury (FI)	
Property damage only (PDO)	

Worksheet 3A. Predicted Crashes by Collision and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Collision Type/Site Type	**Predicted Average Crash Frequency (crashes/year)**			**Observed Crashes, $N_{observed}$ (crashes/year)**	**Overdispersion Parameter, k**	**Weighted Adjustment, w**	**Expected Average Crash Frequency, $N_{expected}$ (vehicle)**
	$N_{predicted\ (total)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			**Equation A-5**	**Equation A-4**
ROADWAY SEGMENTS							
Multiple-Vehicle Nondriveway							
Segment 1							
Segment 2							
Segment 3							
Segment 4							
Single-Vehicle							
Segment 1							
Segment 2							
Segment 3							
Segment 4							
Multiple-Vehicle Driveway-Related							
Segment 1							
Segment 2							
Segment 3							
Segment 4							
INTERSECTIONS							
Multiple-Vehicle							
Intersection 1							
Intersection 2							
Intersection 3							
Intersection 4							
Single-Vehicle							
Intersection 1							
Intersection 2							
Intersection 3							
Intersection 4							

Combined (Sum of Column)					—	—	

Worksheet 3B. Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

(1)	(2)	(3)
Site Type	N_{ped}	N_{bike}
ROADWAY SEGMENTS		
Segment 1		
Segment 2		
Segment 3		
Segment 4		
INTERSECTIONS		
Intersection 1		
Intersection 2		
Intersection 3		
Intersection 4		
Combined (Sum of Column)		

Worksheet 3C. Site-Specific EB Method Summary Results for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)
Crash Severity Level	$N_{predicted}$	N_{ped}	N_{bike}	$N_{expected\ (vehicle)}$	$N_{expected}$
Total	$(2)_{comb}$ Worksheet 3A	$(2)_{comb}$ Worksheet 3B	$(3)_{comb}$ Worksheet 3B	$(13)_{comb}$ Worksheet 3A	(3)+(4)+(5)
Fatal and injury (FI)	$(3)_{comb}$ Worksheet 3A	$(2)_{comb}$ Worksheet 3B	$(3)_{comb}$ Worksheet 3B	$(5)_{total}*(2)_{FI}/(2)_{total}$	(3)+(4)+(5)
Property damage only (PDO)	$(4)_{comb}$ Worksheet 3A	— 0.000	— 0.000	$(5)_{total}*(2)_{PDO}/(2)_{total}$	(3)+(4)+(5)

Worksheet 4A. Predicted Crashes by Collision and Site Type and Observed Crashes Using the Project-Level EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	Predicted Crashes					$N_{predicted\ wo}$	$N_{predicted\ w1}$	w_o	N_o	w_1	N_1	$N_{expected/comb\ (vehicle)}$
Collision Type/Site Type	$N_{predicted\ (total)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$	Observed Crashes, $N_{observed}$ (crashes/year)	Overdispersion Parameter, k	Equation A-8 (6)*(2)²	Equation A-9 (*sqrt*((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS												
Multiple-Vehicle Nondriveway												
Segment 1				—				—	—	—	—	—
Segment 2				—				—	—	—	—	—
Segment 3				—				—	—	—	—	—
Segment 4				—				—	—	—	—	—
Single-Vehicle												
Segment 1				—				—	—	—	—	—
Segment 2				—				—	—	—	—	—
Segment 3				—				—	—	—	—	—
Segment 4				—				—	—	—	—	—
Multiple-Vehicle Driveway-Related												
Segment 1				—				—	—	—	—	—
Segment 2				—				—	—	—	—	—
Segment 3				—				—	—	—	—	—
Segment 4				—				—	—	—	—	—
INTERSECTIONS												
Multiple-Vehicle												

Intersection 1				—				—	—	—	—	—
Intersection 2				—				—	—	—	—	—
Intersection 3				—				—	—	—	—	—
Intersection 4				—				—	—	—	—	—
Single-Vehicle												
Intersection 1				—				—	—	—		—
Intersection 2				—				—	—	—		—
Intersection 3				—				—	—	—		—
Intersection 4				—				—	—	—		—
Combined (Sum of Column)												

Worksheet 4B. Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

(1)	(2)	(3)
Site Type	N_{ped}	N_{bike}
ROADWAY SEGMENTS		
Segment 1		
Segment 2		
Segment 3		
Segment 4		
INTERSECTIONS		
Intersection 1		
Intersection 2		
Intersection 3		
Intersection 4		
Combined (Sum of Column)		

Worksheet 4C. Project-Level EB Method Summary Results for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)
Crash Severity Level	$N_{predicted}$	N_{ped}	N_{bike}	$N_{expected/comb\ (vehicle)}$	$N_{expected}$
Total	$(2)_{comb}$ Worksheet 4A	$(2)_{comb}$ Worksheet 4B	$(3)_{comb}$ Worksheet 4B	$(13)_{comb}$ Worksheet 4A	(3)+(4)+(5)
Fatal and injury (FI)	$(3)_{comb}$ Worksheet 4A	$(2)_{comb}$ Worksheet 4B	$(3)_{comb}$ Worksheet 4B	$(5)_{total}*(2)_{FI}/(2)_{total}$	(3)+(4)+(5)
Property damage only (PDO)	$(4)_{comb}$ Worksheet 4A	—	—	$(5)_{total}*(2)_{PDO}/(2)_{total}$	(3)+(4)+(5)
		0.000	0.000		

Roundabout Sample Problems

SAMPLE PROBLEM 1

The Site/Facility
A four-leg roundabout located in a rural area at a ramp terminal.

The Question
What is the predicted crash frequency of the roundabout for a particular year?

The Facts
- 1 circulating lane
- Entering AADT on leg 1 of 2,000 veh/day
- Leg 2 is outbound only (entrance ramp to a freeway)
- Entering AADT on leg 3 of 1,500 veh/day
- Entering AADT on leg 4 of 1,000 veh/day
- Total Entering AADT = 4,500 veh/day
- Bypass lane present (only) on Leg 1
- Leg 1 has one unsignalized intersection access within 250 ft. of the yield line.

Assumptions
The calibration factor is assumed to be 1.00.

Access related crashes are included in the data.

Results
Using the predictive method steps as outlined below, the predicted average crash frequency for the roundabout in Sample Problem 1 is determined to be 0.0446 FI and 0.8100 PDO crashes per year for a total of 0.8546 crashes per year.

Steps

Step 1 through 8
To determine the predicted average crash frequency of the roundabout in Sample Problem 1, only Steps 9 through 11 are conducted. No other steps are necessary because only one roundabout is analyzed for one year, and the EB Method is not applied.

Step 9—For the selected site, determine and apply the appropriate safety performance function (SPF) for the site's facility type and traffic control features.
For a four-leg roundabout, SPF values for fatal-injury (FI) and PDO crashes are determined.

The SPF for a single roundabout with one circulating lane is calculated from Equation 12-[Insert #] and Table 12-[Insert #] as follows:

$$N_{SPF,m} = exp\left[\mathrm{a} + \mathrm{b} \times LN(EntAADT \quad /1000) + \mathrm{c} \times I_{rural}\right]$$

$$N_{SPF,(FI)} = exp\left[-3.503 + 0.915 \times LN(4500 \quad /1000) + 0.206\right]$$

$$= \quad 0.1465 \text{ FI crashes/year}$$

$$N_{SPF,(PDO)} = exp\left[-1.475 + 0.702 \times LN(4500 \quad /1000) + 0.168\right]$$

$$= 0.7779 \text{ PDO crashes/year}$$

Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site-specific geometric design and traffic control features.

Each CMF used in the calculation of the predicted average crash frequency of the intersection is calculated below. These are the CMFs that pertain to rural, single lane roundabouts.

CMF_{outbd}—Outbound-Only Leg

The outbound-only leg CMF from Table 12-?? is 0.426 and is applied only for FI crashes.

$CMF_{bypass,j}$—Right-Turn Bypass Lane

The right-turn bypass lane CMF (CMF_{bypass}) is a leg-specific CMF in this case applicable only to Leg 1. It has a value of 0.335 (from Table [Insert #]) that is applied only for FI crashes.

$CMF_{ap,j}$—Access Point Frequency

The access point frequency CMF is a leg-specific CMF, in this case applicable only to Leg 1 and only applied because access-related crashes are included in the data. It is described using the following equation.

$$CMF_{ap,j} = exp\left[\mathrm{a} \times n_{ap,j}\right]$$

where

-
- $n_{ap,j}$ = number of driveways or unsignalized access points on leg j (j = 1 to m) (i.e., within 250 ft of yield line).

From Table xx, the value of a is 0.0659 for FI crashes and 0.0885 for PDO crashes giving CMFs of 1.068 and 1.092 for FI and PDO crashes, respectively.

CMF_{leg}—Leg-level features

The leg-specific CMF for Leg 1 is the product of the CMFs for right turn by-pass lane and access point frequency. The value is 0.358 for FI crashes and 1.092 for PDO crashes.

The leg-specific level CMF is 1.0 for the other legs.

A weighted average CMF for leg-level features is obtained by weighting the leg specific CMFs by the respective entering AADTs.

Thus, CMF = [0.357*2000+1*1500+1*1000]/4500 = 0.7142 for FI crashes

and, equivalently, 1.0409 for PDO crashes.

For FI crashes, the CMF-adjusted SPF prediction is obtained by multiplying the base prediction by the CMF for leg-level features and the CMF for presence of an outbound only leg.

For FI crashes, the CMF-adjusted SPF prediction is 0.1465* 0.7142*0.426 = 0.0446 crashes/year

For PDO crashes, the CMF-adjusted SPF prediction is obtained by multiplying the base prediction by the CMF for leg level features.

For the CMF-adjusted SPF prediction is 0.7779*1.0409 = 0.8100 crashes/year

The results for FI and PDO crashes may be summed to obtain 0.8546 total crashes/year

Step 11—Multiply the result obtained in Step 10 by the appropriate calibration factor.
It is assumed in Sample Problem 1 that a calibration factor, C_i, of 1.00 has been determined for local conditions. See Part C, Appendix A.1 for further discussion on calibration of the predicted models.

Calculation of Predicted Average Crash Frequency
The predicted average crash frequency is calculated using Equation 12-[Insert #] based on results obtained in Steps 9 through 11 as follows:

$$\begin{aligned} N_{\text{predicted int}} &= C_i \times (0.8546) \\ &= 1.00 \times 0.8546 \\ &= 0.8546 \text{ crashes/year} \end{aligned}$$

SAMPLE PROBLEM 2

The Site/Facility
Leg 1 of the four-leg roundabout in Sample Problem 2 located in a rural area.

The Question
What is the predicted crash frequency for a particular year for the following crash types?

- Entering-circulating
- Exiting-circulating
- Rear-end approach crashes on the leg

The Facts
- 1 circulating lane
- 1 entering lane
- 2 exiting lanes
- Entering AADT on leg of 2,000 veh/day
- Bypass lane is present

- Exiting AADT on leg of 2,000 veh/day
- Circulating AADT of 1,800 veh/day
- Approach AADT (total of entering and exiting AADT) of 4,000 veh/day
- Leg has one unsignalized intersection access within 250 ft. of the yield line.
- Inscribed circle diameter (ICD) of 155 ft
- Intersection angle of 90 degrees
- Circulating width of 35 ft
- 4 luminaires are present

Assumptions

The calibration factor is assumed to be 1.00.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the leg in Sample Problem 2 is determined to be 0.0486 entering-circulating crashes, 0.0191 exiting-circulating crashes and 0.0403 rear-end approach crashes per year.

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roundabout leg in Sample Problem 2, only Steps 9 through 11 are conducted. No other steps are necessary because only one roundabout is analyzed for one year, and the EB Method is not applied.

Step 9—For the selected site, determine and apply the appropriate safety performance function (SPF) for the site's facility type and traffic control features.

For a roundabout leg, SPF values for entering-circulating crashes, exiting-circulating crashes and rear-end approach crashes are determined from Equations 12-[Insert #] to 12-[Insert #] and Table 12-[Insert #].

For entering-circulating crashes

Equation 12-?

$$N_{SPF,i} = exp\left[\mathrm{a} + \mathrm{b} \times LN(EnteringAADT \quad /1000) + \mathrm{c} \times LN(CirculatingAADT/1000)\right]$$

$$N_{SPF,1} = exp\left[-2.584 + 0.6091 \times LN(2000 \quad /1000) + 0.3020 \times LN(1800/1000)\right]$$

= 0.1374 entering-circulating crashes/year.

For exiting-circulating crashes

Equation 12-?

$$N_{SPF,i} = exp\left[\mathrm{a} + \mathrm{b} \times LN(ExitingAADT \quad /1000) + \mathrm{c} \times LN(CirculatingAADT/1000)\right]$$

$$N_{SPF,1} = exp\left[-6.582 + 2.0150 \times LN(2000 \quad /1000) + 0.5511 \times LN(1800/1000)\right]$$

•

= 0.0077 exiting-circulating crashes/year.

For rear-end approach crashes

Equation 12-?

$$N_{SPF,i} = exp\left[\mathrm{a} + \mathrm{b} \times LN(ApproachAADT \quad \mathrm{c} \times LN(CirculatingAADT/1000)\right]$$

$$N_{SPF,1} = exp\left[-4.781 + 1.0978 \times LN(4000 \quad /1000) + 0.3034 \times LN(1800/1000)\right]$$

= 0.0459 rear-end approach crashes/year.

Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site-specific geometric design and traffic control features.
Each CMF used in the calculation of the predicted average crash frequency of the leg is calculated below. These are the CMFs that pertain to the specific crash types of interest in this sample problem.

For entering-circulating crashes for a leg with one entering lane and one circulating lane, the following CMFs from Table 12-XX are applicable:

Inscribed Circle Diameter (ICD): The CMF is 0.9932 for each 1 ft. increase in ICD from a base SPF value of 150 ft.

Bypass lane present: The CMF is 0.3685 for presence of a bypass lane (from a base value of no lanes present)

The CMF-adjusted SPF prediction is obtained by multiplying the base prediction by the CMF for circulating width, and the CMF for ICD and the CMF for presence of by-pass lane.

For entering-circulating crashes, the CMF-adjusted SPF prediction is $0.1374*[0.9932^{(155-150)}]*0.3685 = 0.0486$ crashes/year

For exiting-circulating crashes for a leg with two exiting lanes and one circulating lane, the following CMF from Table 12-xx is applicable:

Circulating Width: The CMF is 1.198 for each 1 ft. increase in width from a base SPF value of 30 ft.

The CMF-adjusted SPF prediction is obtained by multiplying the base prediction by the CMF for circulating width.

For exiting-circulating crashes, the CMF-adjusted SPF prediction is $0.0077* [1.198^{(35-30)}] = 0.0191$ crashes/year

For rear-end approach crashes for a roundabout leg, the following CMFs from Table 12-xx are applicable:

Number of accesses within 250 ft: The CMF is 1.094 for each increase in access frequency from a base SPF value of 1 access.

Number of luminaires: The CMF is 0.937 for each increase in luminaire frequency from a base SPF value of 2 luminaires.

The CMF-adjusted SPF prediction is obtained by multiplying the base prediction by the CMF for number of accesses, and the CMF for number of luminaires.

For rear-end approach crashes, the CMF-adjusted SPF prediction is $0.0459 * [0.9379^{(4-2)}] * [1.094^{(1-1)}] = 0.0403$ crashes/year

Step 11—Multiply the results obtained in Step 10 by the appropriate calibration factor.
It is assumed in Sample Problem 2 that a calibration factor, C_i, of 1.00 has been determined for local conditions. See Part C, Appendix A.1 for further discussion on calibration of the predicted models. Thus the predicted average crash frequencies are the same as those obtained in Step 10.

Revisions to Part C, Appendix A from HSM, First Edition

This appendix presents two specialized procedures intended for use with the predictive method presented in Chapters 10, 11, and 12. These include the procedure for calibrating the predictive models presented in the Part C chapters to local conditions and the Empirical Bayes (EB) Method for combining observed crash frequencies with the estimate provided by the predictive models in Part C. Both of these procedures are an integral part of the predictive method in Chapters 10, 11, and 12, and are presented in this Appendix only to avoid repetition across the chapters.

A.1. CALIBRATION OF THE PART C PREDICTIVE MODELS

The Part C predictive method in Chapters 10, 11, and 12 include predictive models which consist of safety performance functions (SPFs), crash modification factors (CMFs) and calibration factors and have been developed for specific roadway segment and intersection types. The SPF functions are the basis of the predictive models and were developed in HSM-related research from the most complete and consistent available data sets. However, the general level of crash frequencies may vary substantially from one jurisdiction to another for a variety of reasons including climate, driver populations, animal populations, crash reporting thresholds, and crash reporting system procedures. Therefore, for the Part C predictive models to provide results that are meaningful and accurate for each jurisdiction, it is important that the SPFs be calibrated for application in each jurisdiction. A procedure for determining the calibration factors for the Part C predictive models is presented below in Appendix A.1.1.

Some HSM users may prefer to develop SPFs with data from their own jurisdiction for use in the Part C predictive models rather than calibrating the Part C SPFs. Calibration of the Part C SPFs will provide satisfactory results. However, SPFs developed directly with data for a specific jurisdiction may provide more reliable estimates for that jurisdiction than calibration of Part C SPFs. Therefore, jurisdictions that have the capability, and wish to develop their own models, are encouraged to do so. Guidance on development of jurisdiction-specific SPFs that are suitable for use in the Part C predictive method is presented in Appendix A.1.2.

Most of the regression coefficients and distribution values used in the Part C predictive models in Chapters 10, 11, and 12 have been determined through research and, therefore, modification by users is not recommended. However, a few specific quantities, such as the distribution of crashes by collision type or the proportion of crashes occurring during nighttime conditions, are known to vary substantially from jurisdiction to jurisdiction. Where appropriate local data are available, users are encouraged to replace these default values with locally derived values. The values in the predictive models that may be updated by users to fit local conditions are explicitly identified in Chapters 10, 11, and 12. Unless explicitly identified, values in the predictive models should not be modified by the user. A procedure for deriving jurisdiction-specific values to replace these selected parameters is presented below in Appendix A.1.3.

A.1.1. Calibration of Predictive Models

The purpose of the Part C calibration procedure is to adjust the predictive models which were developed with data from one jurisdiction for application in another jurisdiction. Calibration provides a method to account for differences between jurisdictions in factors such as climate, driver populations, animal populations, crash reporting thresholds, and crash reporting system procedures.

The calibration procedure is used to derive the values of the calibration factors for roadway segments and for intersections that are used in the Part C predictive models. The calibration factor for roadway segments, C_r, is used in Equations 10-2, 11-2, 11-3, and 12-2. The calibration factor for intersections, C_i, is used in Equations 10-3, 11-4,

and 12-5. The calibration factors, C_r and C_i, are based on the ratio of the total observed crash frequencies for a selected set of sites to the total expected average crash frequency estimated for the same sites, during the same time period, using the applicable Part C predictive method. Thus, the nominal value of the calibration factor, when the observed and predicted crash frequencies happen to be equal, is 1.00. When there are more crashes observed than are predicted by the Part C predictive method, the computed calibration factor will be greater than 1.00. When there are fewer crashes observed than are predicted by the Part C predictive method, the computed calibration factor will be less than 1.00.

It is recommended that new values of the calibration factors be derived at least every two to three years, and some HSM users may prefer to develop calibration factors on an annual basis. The calibration factor for the most recent available period is to be used for all assessment of proposed future projects. If available, calibration factors for the specific time periods included in the evaluation periods before and after a project or treatment implementation are to be used in effectiveness evaluations that use the procedures presented in Chapter 9.

If the procedures in Appendix A.1.3 are used to calibrate any default values in the Part C predictive models to local conditions, the locally-calibrated values should be used in the calibration process described below.

The calibration procedure involves five steps:

- *Step 1*—Identify facility types for which the applicable Part C predictive model is to be calibrated.
- *Step 2*—Select sites for calibration of the predictive model for each facility type.
- *Step 3*—Obtain data for each facility type applicable to a specific calibration period.
- *Step 4*—Apply the applicable Part C predictive model to predict total crash frequency for each site during the calibration period as a whole.
- *Step 5*—Compute calibration factors for use in Part C predictive model.

Each of these steps is described below.

A.1.1.1. Step 1—Identify Facility Types for Which the Applicable Part C SPFs are to be Calibrated.

Calibration is performed separately for each facility type addressed in each Part C chapter. Table A-1 identifies all of the facility types included in the Part C chapters for which calibration factors need to be derived. The Part C SPFs for each of these facility types are to be calibrated before use, but HSM users may choose not to calibrate the SPFs for particular facility types if they do not plan to apply the Part C SPFs for those facility types.

Table A-1. SPFs in the Part C Predictive Models that Need Calibration

Facility, Segment, or Intersection Type	Calibration Factor to be Derived	
	Symbol	Equation Number(s)
ROADWAY SEGMENTS		
Rural Two-Lane, Two-Way Roads		
Two-lane undivided segments	C_r	10-2
Rural Multilane Highways		
Undivided segments	C_r	11-2
Divided segments	C_r	11-3
Urban and Suburban Arterials		
Two-lane undivided segments	C_r	12-2
Three-lane segments with center two-way left-turn lane	C_r	12-2
Four-lane undivided segments	C_r	12-2
Four-lane divided segments	C_r	12-2
Five-lane segments with center two-way left-turn lane	C_r	12-2
INTERSECTIONS		
Rural Two-Lane, Two-Way Roads		
Three-leg intersections with minor-road stop control	C_i	10-3
Four-leg intersections with minor-road stop control	C_i	10-3
Four-leg signalized intersections	C_i	10-3
Rural Multilane Highways		
Three-leg intersections with minor-road stop control	C_i	11-4
Four-leg intersections with minor-road stop control	C_i	11-4
Four-leg signalized intersections	C_i	11-4
Urban and Suburban Arterials		
Three-leg intersections with minor-road stop control	C_i	12-5
Three-leg signalized intersections	C_i	12-5
Four-leg intersections with minor-road stop control	C_i	12-5
Four-leg signalized intersections	C_i	12-5
Roundabout Intersections		
Three-leg single circulating lane	C_i	12-?
Three-leg two circulating lanes	C_i	12-?
Four-leg single circulating lane	C_i	12-?
Four-leg two circulating lanes	C_i	12-?
Roundabout Approaches.	???	???

A.1.1.2. Step 2—Select Sites for Calibration of the SPF for Each Facility Type.

For each facility type, the desirable minimum sample size for the calibration data set is 30 to 50 sites, with each site long enough to adequately represent physical and safety conditions for the facility. Calibration sites should be selected without regard to the number of crashes on individual sites; in other words, calibration sites should not be selected to intentionally limit the calibration data set to include only sites with either high or low crash frequencies. Where practical, this may be accomplished by selecting calibration sites randomly from a larger set of candidate sites. Following site selection, the entire group of calibration sites should represent a total of at least 100 crashes per year. These calibration sites will be either roadway segments or intersections, as appropriate to the facility type being addressed. If the required data discussed in Step 3 are readily available for a larger number of sites, that larger number of sites should be used for calibration. If a jurisdiction has fewer than 30 sites for a particular facility type, then it is desirable to use all of those available sites for calibration. For large jurisdictions, such as entire states, with a variety of topographical and climate conditions, it may be desirable to assemble a separate set of sites and develop separate calibration factors for each specific terrain type or geographical region. For example, a state with distinct plains and mountains regions, or with distinct dry and wet regions, might choose to develop separate calibration factors for those regions. On the other hand, a state that is relatively uniform in terrain and climate might choose to perform a single calibration for the entire state. Where separate calibration factors are developed by terrain type or region, this needs to be done consistently for all applicable facility types in those regions.

It is desirable that the calibration sites for each facility type be reasonably representative of the range of site characteristics to which the predictive model will be applied. However, no formal stratification by traffic volume or other site characteristics is needed in selecting the calibration sites, so the sites can be selected in a manner to make the data collection needed for Step 3 as efficient as practical. There is no need to develop a new data set if an existing data set with sites suitable for calibration is already available. If no existing data set is available so that a calibration data set consisting entirely of new data needs to be developed, or if some new sites need to be chosen to supplement an existing data set, it is desirable to choose the new calibration sites by random selection from among all sites of the applicable facility type.

Step 2 only needs to be performed the first time that calibration is performed for a given facility type. For calibration in subsequent years, the same sites may be used again.

A.1.1.3. Step 3—Obtain Data for Each Facility Type Applicable to a Specific Calibration Period.

Once the calibration sites have been selected, the next step is to assemble the calibration data set if a suitable data set is not already available. For each site in the calibration data set, the calibration data set should include:

- Total observed crash frequency for a period of one or more years in duration.
- All site characteristics data needed to apply the applicable Part C predictive model.

Observed crashes for all severity levels should be included in calibration. The duration of crash frequency data should correspond to the period for which the resulting calibration factor, C_r or C_i, will be applied in the Part C predictive models. Thus, if an annual calibration factor is being developed, the duration of the calibration period should include just that one year. If the resulting calibration factor will be employed for two or three years, the duration of the calibration period should include only those years. Since crash frequency is likely to change over time, calibration periods longer than three years are not recommended. All calibration periods should have durations that are multiples of 12 months to avoid seasonal effects. For ease of application, it is recommended that the calibration periods consist of one, two, or three full calendar years. It is recommended to use the same calibration period for all sites, but exceptions may be made where necessary.

The observed crash data used for calibration should include all crashes related to each roadway segment or intersection selected for the calibration data set. Crashes should be assigned to specific roadway segments or intersections based on the guidelines presented below in Appendix A.2.3.

Table A-2 identifies the site characteristics data that are needed to apply the Part C predictive models for each facility type. The table classifies each data element as either required or desirable for the calibration procedure. Data

for each of the required elements are needed for calibration. If data for some required elements are not readily available, it may be possible to select sites in Step 2 for which these data are available. For example, in calibrating the predictive models for roadway segments on rural two-lane, two-way roads, if data on the radii of horizontal curves are not readily available, the calibration data set could be limited to tangent roadways. Decisions of this type should be made, as needed, to keep the effort required to assemble the calibration data set within reasonable bounds. For the data elements identified in Table A-2 as desirable, but not required, it is recommended that actual data be used if available, but assumptions are suggested in the table for application where data are not available.

A.2. Table A-2. Data Needs for Calibration of Part C Predictive Models by Facility Type

Chapter	Data Element	Data Need: Required	Data Need: Desirable	Default Assumption
ROADWAY SEGMENTS				
10—Rural Two-Lane, Two-Way Roads	Segment length	X		Need actual data
	Annual average daily traffic (AADT)	X		Need actual data
	Lengths of horizontal curves and tangents	X		Need actual data
	Radii of horizontal curves	X		Need actual data
	Presence of spiral transition for horizontal curves		X	Base default on agency design policy
	Superelevation variance for horizontal curves		X	No superelevation variance
	Percent grade		X	Base default on terrain[a]
	Lane width	X		Need actual data
	Shoulder type	X		Need actual data
	Shoulder width	X		Need actual data
	Presence of lighting		X	Assume no lighting
	Driveway density		X	Assume 5 driveways per mile
	Presence of passing lane		X	Assume not present
	Presence of short four-lane section		X	Assume not present
	Presence of center two-way left-turn lane	X		Need actual data
	Presence of centerline rumble strip		X	Base default on agency design policy
	Roadside hazard rating		X	Assume roadside hazard rating = 3
	Use of automated speed enforcement		X	Base default on current practice
11—Rural Multilane Highways	*For all rural multilane highways*:			
	Segment length	X		Need actual data
	Annual average daily traffic (AADT)	X		Need actual data
	Lane width	X		Need actual data
	Shoulder width	X		Need actual data
	Presence of lighting		X	Assume no lighting
	Use of automated speed enforcement		X	Base default on current practice
	For undivided highways only:			
	Sideslope	X		Need actual data
	For divided highways only:			
	Median width	X		Need actual data
12—Urban and	Segment length	X		Need actual data

Suburban Arterials	Number of through traffic lanes	X		Need actual data
	Presence of median	X		Need actual data
	Presence of center two-way left-turn lane	X		Need actual data
	Annual average daily traffic (AADT)	X		Need actual data
	Number of driveways by land-use type	X		Need actual data[b]
	Posted speed limit	X		Need actual data
	Presence of on-street parking	X		Need actual data
	Type of on-street parking	X		Need actual data
	Roadside fixed object density		X	database default on fixed-object offset and density categories[c]
	Presence of lighting		X	Base default on agency practice
	Presence of automated speed enforcement		X	Base default on agency practice
INTERSECTIONS				
10—Rural Two-Lane, Two-Way Roads	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Annual average daily traffic (AADT) for major road	X		Need actual data
	Annual average daily traffic (AADT) for minor road	X		Need actual data or best estimate
	Intersection skew angle		X	Assume no skew[d]
	Number of approaches with left-turn lanes	X		Need actual data
	Number of approaches with right-turn lanes	X		Need actual data
	Presence of lighting	X		Need actual data
11—Rural Multilane Highways	*For all rural multilane highways:*			
	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Annual average daily traffic (AADT) for major road	X		Need actual data
	Annual average daily traffic (AADT) for minor road	X		Need actual data or best estimate
	Presence of lighting	X		Need actual data
	Intersection skew angle		X	Assume no skew[d]
	Number of approaches with left-turn lanes	X		Need actual data
	Number of approaches with right-turn lanes	X		Need actual data
12—Urban and Suburban Arterials	*For all signalized or stop-controlled intersections on arterials:*			
	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Average annual daily traffic (AADT) for	X		Need actual data

	major road		
	Average annual daily traffic (AADT) for minor road	X	Need actual data or best estimate
	Number of approaches with left-turn lanes	X	Need actual data
	Number of approaches with right-turn lanes	X	Need actual data
	Presence of lighting	X	Need actual data
	For signalized intersections only:		
	Presence of left-turn phasing	X	Need actual data
	Type of left-turn phasing	X	Prefer actual data, but agency practice may be used as a default
	Use of right-turn-on-red signal operation	X	Need actual data
	Use of red-light cameras	X	Need actual data
	Pedestrian volume	X	Estimate with Table 12-15
	Maximum number of lanes crossed by pedestrians on any approach	X	Estimate from number of lanes and presence of median on major road
	Presence of bus stops within 1,000 ft	X	Assume not present
	Presence of schools within 1,000 ft	X	Assume not present
	Presence of alcohol sales establishments within 1,000 ft	X	Assume not present
	For all roundabout intersections:		
	Average annual daily traffic (AADT) entering roundabout	X	Need actual data
	Area Type (urban or rural)	X	Need actual data
	Inscribed circle diameter in ft	X	Need actual data
	Presence of outbound only leg	X	Need actual data
	Presence of right-turn bypass lanes	X	Need actual data
	Number of driveways or unsignalized access points within 250 ft of each leg	X	Need actual data
	Entry width in ft on each leg	X	Need actual data
	Circulating width in ft at each entry	X	Need actual data
	For all roundabout approaches:		
	Approaching sverage annual daily traffic (AADT) entering roundabout	X	Need actual data
	Circulating sverage annual daily traffic (AADT) entering roundabout	X	Need actual data
	Exiting sverage annual daily traffic (AADT) entering roundabout	X	Need actual data
	Inscribed circle diameter in ft	X	Need actual data
	Presence of right-turn bypass lanes	X	Need actual data
	Number of driveways or unsignalized access	X	Need actual data

	points within 250 ft of each leg		
	Circulating width in ft at each entry	X	Need actual data
	Posted speed on approaches in m.p.h.	X	Need actual data
	Number of entering lanes	X	Need actual data
	Number of circulating lanes	X	Need actual data
	Number of exiting lanes	X	Need actual data
	Number of luminaires within 250 ft. of entry point on each arm	X	Need actual data

[a] Suggested default values for calibration purposes: $CMF = 1.00$ for level terrain; $CMF = 1.06$ for rolling terrain; $CMF = 1.14$ for mountainous terrain
[b] Use actual data for number of driveways, but simplified land-use categories may be used (e.g., commercial and residential only).
[c] CMFs may be estimated based on two categories of fixed-object offset (O_{fo})—either 5 or 20 ft—and three categories of fixed-object density (D_{fo})—0, 50, or 100 objects per mile.
[d] If measurements of intersection skew angles are not available, the calibration should preferably be performed for intersections with no skew.

A.1.1.4. Step 4—Apply the Applicable Part C Predictive Method to Predict Total Crash Frequency for Each Site During the Calibration Period as a Whole

The site characteristics data assembled in Step 3 should be used to apply the applicable predictive method from Chapter 10, 11, or 12 to each site in the calibration data set. For this application, the predictive method should be applied without using the EB Method and, of course, without employing a calibration factor (i.e., a calibration factor of 1.00 is assumed). Using the predictive models, the expected average crash frequency is obtained for either one, two, or three years, depending on the duration of the calibration period selected.

A.1.1.5. Step 5—Compute Calibration Factors for Use in Part C Predictive Models

The final step is to compute the calibration factor as:

$$C_r(\text{ or } C_i) = \frac{\sum_{\text{all sites}} \text{observed crashes}}{\sum_{\text{all sites}} \text{predicted crashes}} \tag{A-1}$$

The computation is performed separately for each facility type. The computed calibration factor is rounded to two decimal places for application in the appropriate Part C predictive model.

Example Calibration Factor Calculation

The SPF for four-leg signalized intersections on rural two-lane, two-way roads from Equation 10-10 is:

$$N_{spf\ int} = e^{\left[-5.13+0.60\times\ln\left(AADT_{maj}\right)+0.20\times\ln\left(AADT_{min}\right)\right]}$$

Where:

$N_{spf\ int}$ = predicted number of total intersection-related crashes per year for base conditions;

$AADT_{maj}$ = average annual daily entering traffic volumes (vehicles/day) on the major road; and

$AADT_{min}$ = average annual daily entering traffic volumes (vehicles/day) on the minor road.

The base conditions are:

- No left-turn lanes on any approach
- No right-turn lanes on any approach

The CMF values from Chapter 10 are:

- CMF for one approach with a left-turn lane = 0.82
- CMF for one approach with a right-turn lane = 0.96
- CMF for two approaches with right-turn lanes = 0.92
- No lighting present (so lighting $\text{CMF} = 1.00$ for all cases)

Typical data for eight intersections is shown in an example calculation shown below. Note that for an actual calibration, the recommended minimum sample size would be 30 to 50 sites that experience at least 100 crashes per year. Thus, the number of sites used here is smaller than recommended, and is intended solely to illustrate the calculations.

For the first intersection in the example the predicted crash frequency for base conditions is:

$$N_{bibase} = e^{\left(-5.13+0.60\times\ln(4000)+0.20\times\ln(2000)\right)} = 3.922 \text{ crashes/year}$$

The intersection has a left-turn lane on the major road, for which CMF_{1i} is 0.82, and a right-turn lane on one approach, a feature for which CMF_{2i} is 0.96. There are three years of data, during which four crashes were observed (shown in Column 10 of Table Ex-1). The predicted average crash frequency from the Chapter 10 for this intersection without calibration is from Equation 10-2:

$$\begin{aligned} N_{bi} &= (N_{bibase})\times(CMF_{1i})\times(CMF_{2i})\times(\text{number of years of data}) \\ &= 3.922\times0.82\times0.96\times3 = 9.262 \text{ crashes in three years, shown in Column 9.} \end{aligned}$$

Similar calculations were done for each intersection in the table shown below. The sum of the observed crash frequencies in Column 10 (43) is divided by the sum of the predicted average crash frequencies in Column 9 (87.928) to obtain the calibration factor, *Ci*, equal to 0.489. It is recommended that calibration factors be rounded to two decimal places, so calibration factor equal to 0.49 should be used in the Chapter 10 predictive model for four-leg signalized intersections.

Table Ex-1. Example of Calibration Factor Computation

1	2	3	4	5	6	7	8	9	10
$AADT_{maj}$	$AADT_{min}$	SPF Prediction	Intersection Approaches with Left-Turn Lanes	CMF_{1i}	Intersection Approaches with Right-Turn Lane	CMF_{2i}	Years of Data	Predicted Average Crash Frequency	Observed Crash Frequency
4000	2000	3.922	1	0.82	1	0.96	3	9.262	4
3000	1500	3.116	0	1.00	2	0.92	2	5.733	5
5000	3400	4.986	0	1.00	2	0.92	3	13.761	10
6500	3000	5.692	0	1.00	2	0.92	3	15.709	5
3600	2300	3.786	1	0.82	1	0.96	3	8.941	2
4600	4500	5.016	0	1.00	2	0.92	3	13.844	8
5700	3300	5.362	1	0.82	1	0.96	3	12.662	5
6800	1500	5.091	1	0.82	1	0.96	2	8.015	4
							Sum	87.928	43
							Calibration Factor (C_i)		0.489

A.1.2. Development of Jurisdiction-Specific Safety Performance Functions for Use in the Part C Predictive Method

Satisfactory results from the Part C predictive method can be obtained by calibrating the predictive model for each facility type, as explained in Appendix A.1.1. However, some users may prefer to develop jurisdiction-specific SPFs using their agency's own data, and this is likely to enhance the reliability of the Part C predictive method. While there is no requirement that this be done, HSM users are welcome to use local data to develop their own SPFs, or if they wish, replace some SPFs with jurisdiction-specific models and retain other SPFs from the Part C chapters. Within the first two to three years after a jurisdiction-specific SPF is developed, calibration of the jurisdiction-specific SPF using the procedure presented in Appendix A.1.1 may not be necessary, particularly if other default values in the Part C models are replaced with locally-derived values, as explained in Appendix A.1.3.

If jurisdiction-specific SPFs are used in the Part C predictive method, they need to be developed with methods that are statistically valid and developed in such a manner that they fit into the applicable Part C predictive method. The following guidelines for development of jurisdiction-specific SPFs that are acceptable for use in Part C include:

- In preparing the crash data to be used for development of jurisdiction-specific SPFs, crashes are assigned to roadway segments and intersections following the definitions explained in Appendix A.2.3 and illustrated in Figure A-1.
- The jurisdiction-specific SPF should be developed with a statistical technique such as negative binomial regression that accounts for the overdispersion typically found in crash data and quantifies an overdispersion parameter so that the model's predictions can be combined with observed crash frequency data using the EB Method.
- The jurisdiction-specific SPF should use the same base conditions as the corresponding SPF in Part C or should be capable of being converted to those base conditions.
- The jurisdiction-specific SPF should include the effects of the following traffic volumes: average annual daily traffic volume for roadway segment and major- and minor-road average annual daily traffic volumes for intersections.

- The jurisdiction-specific SPF for any roadway segment facility type should have a functional form in which predicted average crash frequency is directly proportional to segment length.

These guidelines are not intended to stifle creativity and innovation in model development. However, a model that does not account for overdispersed data or that cannot be integrated with the rest of the Part C predictive method will not be useful.

Two types of data sets may be used for SPF development. First, SPFs may be developed using only data that represent the base conditions, which are defined for each SPF in Chapters 10, 11, and 12. Second, it is also acceptable to develop models using data for a broader set of conditions than the base conditions. In this approach, all variables that are part of the applicable base-condition definition, but have non-base-condition values, should be included in an initial model. Then, the initial model should be made applicable to the base conditions by substituting values that correspond to those base conditions into the model. Several examples of this process are presented in Appendix 10A.

A.1.3. Replacement of Selected Default Values in the Part C Predictive Models to Local Conditions

The Part C predictive models use many default values that have been derived from crash data in HSM-related research. For example, the urban intersection predictive model in Chapter 12 uses pedestrian factors that are based on the proportion of pedestrian crashes compared to total crashes. Replacing these default values with locally derived values will improve the reliability of the Part C predictive models. Table A-3 identifies the specific tables in Part C that may be replaced with locally derived values. In addition to these tables, there is one equation—Equation 10-18—which uses constant values given in the accompanying text in Chapter 10. These constant values may be replaced with locally derived values.

Providing locally-derived values for the data elements identified in Table A-3 is optional. Satisfactory results can be obtained with the Part C predictive models, as they stand, when the predictive model for each facility type is calibrated with the procedure given in Appendix A.1.1. But, more reliable results may be obtained by updating the data elements listed in Table A-3. It is acceptable to replace some, but not all of these data elements, if data to replace all of them are not available. Each element that is updated with locally-derived values should provide a small improvement in the reliability of that specific predictive model. To preserve the integrity of the Part C predictive method, the quantitative values in the predictive models, (other than those listed in Table A-3 and those discussed in Appendices A.1.1 and A.2.2), should not be modified. Any replacement values derived with the procedures presented in this section should be incorporated in the predictive models before the calibration described in Appendix A.1.1 is performed.

A.3. Table A-3. Default Crash Distributions Used in Part C Predictive Models Which May Be Calibrated by Users to Local Conditions

		Type of Roadway Element		
Chapter	**Table or Equation Number**	**Roadway Segments**	**Intersections**	**Data Element or Distribution That May Be Calibrated to Local Conditions**
10—Rural Two-Lane, Two-Way Roads	Table 10-3	X		Crash severity by facility type for roadway segments
	Table 10-4	X		Collision type by facility type for roadway segments
	Table 10-5		X	Crash severity by facility type for intersections
	Table 10-6		X	Collision type by facility type for intersections
	Equation 10-18	X		Driveway-related crashes as a proportion of total crashes (p_{dwy})
	Table 10-12	X		Nighttime crashes as a proportion of total crashes by severity level

	Table 10-15		X	Nighttime crashes as a proportion of total crashes by severity level and by intersection type
11—Rural Multilane Highways	Table 11-4	X		Crash severity and collision type for undivided segments
	Table 11-6	X		Crash severity and collision type for divided segments
	Table 11-9		X	Crash severity and collision type by intersection type
	Table 11-15	X		Nighttime crashes as a proportion of total crashes by severity level and by roadway segment type for undivided roadway segments
	Table 11-19	X		Nighttime crashes as a proportion of total crashes by severity level and by roadway segment type for divided roadway segments
	Table 11-24		X	Nighttime crashes as a proportion of total crashes by severity level and by intersection type
12—Urban and Suburban Arterials	Table 12-4	X		Crash severity and collision type for multiple-vehicle nondriveway collisions by roadway segment type
	Table 12-6	X		Crash severity and collision type for single-vehicle crashes by roadway segment type
	Table 12-7	X		Crash severity for driveway-related collisions by roadway segment type[a]
	Table 12-8	X		Pedestrian crash adjustment factor by roadway segment type
	Table 12-9	X		Bicycle crash adjustment factor by roadway segment type
	Table 12-11		X	Crash severity and collision type for multiple-vehicle collisions by intersection type
	Table 12-13		X	Crash severity and collision type for single-vehicle crashes by intersection type
	Table 12-16		X	Pedestrian crash adjustment factor by intersection type for stop-controlled intersections
	Table 12-17		X	Bicycle crash adjustment factor by intersection type
	Table 12-23	X		Nighttime crashes as a proportion of total crashes by severity level and by roadway segment type
	Table 12-27		X	Nighttime crashes as a proportion of total crashes by severity level and by intersection type
	Table 12-?		X	Fatal+Injury crash type distribution for single circulating lane roundabouts
	Table 12-?		X	Fatal+Injury crash type distribution for two circulating lane roundabouts
	Table 12-?		X	PDO crash type distribution for single circulating lane roundabouts
	Table 12-?		X	PDO crash type distribution for two circulating lane roundabouts

[a] The only portion of Table 12-7 that should be modified by the user are the crash severity proportions.
Note: No quantitative values in the Part C predictive models, other than those listed here and those discussed in Appendices A.1.1 and A.1.2, should be modified by HSM users.

Procedures for developing replacement values for each data element identified in Table A-3 are presented below. Most of the data elements to be replaced are proportions of crash severity levels and/or crash types that are part of a specific distribution. Each replacement value for a given facility type should be derived from data for a set of sites that, as a group, includes at least 100 crashes and preferably more. The duration of the study period for a given set of sites may be as long as necessary to include at least 100 crashes. In the following discussion, the term "sufficient data" refers to a data set including a sufficient number of sites to meet this criterion for total crashes. In a few cases,

explicitly identified below, the definition of sufficient data will be expressed in terms of a crash category other than total crashes. In assembling data for developing replacements for default values, crashes are to be assigned to specific roadway segments or intersections following the definitions explained in Appendix A.2.3 and illustrated in Figure A-1.

A.1.3.1. Replacement of Default Values for Rural Two-Lane, Two-Way Roads

Five specific sets of default values for rural two-lane, two-way roads may be updated with locally-derived replacement values by HSM users. Procedures to develop each of these replacement values are presented below.

Crash Severity by Facility Type

Tables 10-3 and 10-5 present the distribution of crashes by five crash severity levels for roadway segments and intersections, respectively, on rural two-lane, two-way roads. If sufficient data, including these five severity levels (fatal, incapacitating injury, nonincapacitating injury, possible injury, and property damage only), are available for a given facility type, the values in Tables 10-3 and 10-5 for that facility type may be updated. If sufficient data are available only for the three standard crash severity levels (fatal, injury, and property damage only), the existing values in Tables 10-3 and 10-5 may be used to allocate the injury crashes to specific injury severity levels (incapacitating injury, nonincapacitating injury, and possible injury).

Collision Type by Facility Type

Table 10-4 presents the distribution of crashes by collision type for seven specific types of single-vehicle crashes and six specific types of multiple-vehicle crashes for roadway segments, and Table 10-6 presents the distribution of crashes by collision type for three intersection types on rural two-lane, two-way roads. If sufficient data are available for a given facility type, the values in Tables 10-4 and 10-6 for that facility type may be updated.

Driveway-Related Crashes as a Proportion of Total Crashes for Roadway Segments

Equation 10-18 includes a factor, p_{dwy}, which represents the proportion of total crashes represented by driveway-related crashes. A value for p_{dwy} based on research is presented in the accompanying text. This value may be replaced with a locally-derived value, if data are available for a set for sites that, as a group, have experienced at least 100 driveway-related crashes.

Nighttime Crashes as a Proportion of Total Crashes for Roadway Segments

Table 10-12 presents the proportions of total nighttime crashes by severity level and the proportion of total crashes that occur at night for roadway segments on rural two-lane, two-way roads. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime crashes.

Nighttime Crashes as a Proportion of Total Crashes for Intersections

Table 10-15 presents the proportion of total crashes that occur at night for intersections on rural two-lane, two-way roads. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime crashes.

A.1.3.2. Replacement of Default Values for Rural Multilane Highways

Five specific sets of default values for rural multilane highways may be updated with locally-derived replacement values by HSM users. Procedures to develop each of these replacement values are presented below.

Crash Severity and Collision Type for Undivided Roadway Segments

Table 11-4 presents the combined distribution of crashes for four crash severity levels and six collision types. If sufficient data are available for undivided roadway segments, the values in Table 11-4 for this facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 crashes in the time period for which data are available.

Crash Severity and Collision Type for Divided Roadway Segments

Table 11-6 presents the combined distribution of crashes for four crash severity levels and six collision types. If sufficient data are available for divided roadway segments, the values in Table 11-6 for this facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires sites that have experienced at least 200 crashes in the time period for which data are available.

Crash Severity and Collision Type by Intersection Type

Table 11-9 presents the combined distribution of crashes at intersections for four crash severity levels and six collision types. If sufficient data are available for a given intersection type, the values in Table 11-9 for that intersection type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 crashes in the time period for which data are available.

Nighttime Crashes as a Proportion of Total Crashes for Roadway Segments

Tables 11-15 and 11-19 present the proportions of total nighttime crashes by severity level and the proportion of total crashes that occur at night for undivided and divided roadway segments, respectively, on rural multilane highways. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime crashes.

Nighttime Crashes as a Proportion of Total Crashes for Intersections

Table 11-24 presents the proportion of total crashes that occur at night for intersections on rural multilane highways. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime crashes.

A.1.3.3. Replacement of Default Values for Urban and Suburban Arterials

Eleven specific sets of default values for urban and suburban arterial highways may be updated with locally-derived replacement values by HSM users. Procedures to develop each of these replacement values are presented below.

Crash Severity and Collision Type for Multiple-Vehicle Nondriveway Crashes by Roadway Segment Type

Table 12-4 presents the combined distribution of crashes for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Table 12-4 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 crashes in the time period for which data are available.

Crash Severity and Collision Type for Single-Vehicle Crashes by Roadway Segment Type

Table 12-6 presents the combined distribution of crashes for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Table 12-6 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 crashes in the time period for which data are available.

Crash Severity for Driveway-Related Collision by Roadway Segment Type

Table 12-7 includes data on the proportions of driveway-related crashes for two crash severity levels (fatal-and-injury and property-damage-only crashes) by facility type for roadway segments. If sufficient data are available for a given facility type, these specific severity-related values in Table 12-7 for that facility type may be updated. The rest of Table 12-7, other than the last two rows of data which are related to crash severity, should not be modified.

Pedestrian Crash Adjustment Factor by Roadway Segment Type

Table 12-8 presents a pedestrian crash adjustment factor for specific roadway segment facility types and for two speed categories: low speed (traffic speeds or posted speed limits of 30 mph or less) and intermediate or high speed (traffic speeds or posted speed limits greater than 30 mph). For a given facility type and speed category, the pedestrian crash adjustment factor is computed as:

$$f_{pedr} = \frac{K_{ped}}{K_{non}} \tag{A-2}$$

APPENDIX A. Where:

f_{pedr} = pedestrian crash adjustment factor;

K_{ped} = observed vehicle-pedestrian crash frequency; and

K_{non} = observed frequency for all crashes not including vehicle-pedestrian and vehicle-bicycle crash.

The pedestrian crash adjustment factor for a given facility type should be determined with a set of sites of that speed type that, as a group, includes at least 20 vehicle-pedestrian collisions.

Bicycle Crash Adjustment Factor by Roadway Segment Type

Table 12-9 presents a bicycle crash adjustment factor for specific roadway segment facility types and for two speed categories: low speed (traffic speeds or posted speed limits of 30 mph or less) and intermediate or high speed (traffic speeds or posted speed limits greater than 30 mph). For a given facility type and speed category, the bicycle crash adjustment factor is computed as:

$$f_{biker} = \frac{K_{bike}}{K_{non}} \tag{A-3}$$

APPENDIX A. Where:

f_{biker} = bicycle crash adjustment factor;

K_{bike} = observed vehicle-bicycle crash frequency; and

K_{non} = observed frequency for all crashes not including vehicle-pedestrian and vehicle-bicycle crashes.

The bicycle crash adjustment factor for a given facility type should be determined with a set of sites of that speed type that, as a group, includes at least 20 vehicle-bicycle collisions.

Crash Severity and Collision Type for Multiple-Vehicle Crashes by Intersection Type

Table 12-11 presents the combined distribution of crashes for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Table 12-11 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 crashes in the time period for which data are available.

Crash Severity and Collision Type for Single-Vehicle Crashes by Intersection Type

Table 12-13 presents the combined distribution of crashes for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Table 12-13 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 crashes in the time period for which data are available. The default values for f_{bisv} in Equation 12-27 should be replaced with locally available data.

Pedestrian Crash Adjustment Factor by Intersection Type

Table 12-16 presents a pedestrian crash adjustment factor for two specific types of intersections with stop control on the minor road. For a given facility type and speed category, the pedestrian crash adjustment factor is computed using Equation A-2. The pedestrian crash adjustment factor for a given facility type is determined with a set of sites that, as a group, have experienced at least 20 vehicle-pedestrian collisions.

Bicycle Crash Adjustment Factor by Intersection Type

Table 12-17 presents a bicycle crash adjustment factor for four specific intersection facility types. For a given facility type, the bicycle crash adjustment factor is computed using Equation A-3. The bicycle crash adjustment factor for a given facility type is determined with a set of sites that, as a group, have experienced at least 20 vehicle-bicycle collisions.

Nighttime Crashes as a Proportion of Total Crashes for Roadway Segments

Table 12-23 presents the proportions of total nighttime crashes by severity level for specific facility types for roadway segments and the proportion of total crashes that occur at night. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime crashes.

Nighttime Crashes as a Proportion of Total Crashes for Intersections

Table 12-27 presents the proportions of total nighttime crashes by severity level for specific facility types for intersections and the proportion of total crashes that occur at night. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime crashes.

Fatal+Injury Crashes as a proportion of Total Crashes for Single-Lane Roundabouts

Table 12-? presents the proportions of fatal+injury crashes by crash type at single-lane roundabouts. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 crashes.

Fatal+Injury Crashes as a proportion of Total Crashes for Two-Lane Roundabouts

Table 12-? presents the proportions of fatal+injury crashes by crash type at two-lane roundabouts. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 crashes.

PDO Crashes as a proportion of Total Crashes for Single-Lane Roundabouts

Table 12-? presents the proportions of PDO crashes by crash type at single-lane roundabouts. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 crashes.

PDO Crashes as a proportion of Total Crashes for Two-Lane Roundabouts

Table 12-? presents the proportions of PDO crashes by crash type at two-lane roundabouts. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 crashes.

A.2. USE OF THE EMPIRICAL BAYES METHOD TO COMBINE PREDICTED AVERAGE CRASH FREQUENCY AND OBSERVED CRASH FREQUENCY

Application of the EB Method provides a method to combine the estimate using a Part C predictive model and observed crash frequencies to obtain a more reliable estimate of expected average crash frequency. The EB Method is a key tool to compensate for the potential bias due to regression-to-the-mean. Crash frequencies vary naturally from one time period to the next. When a site has a higher than average frequency for a particular time period, the site is likely to have lower crash frequency in subsequent time periods. Statistical methods can help to assure that this natural decrease in crash frequency following a high observed value is not mistaken for the effect of a project or for a true shift in the long-term expected crash frequency.

There are several statistical methods that can be employed to compensate for regression-to-the-mean. The EB Method is used in the HSM because it is best suited to the context of the HSM. The Part C predictive models include negative binomial regression models that were developed before the publication of the HSM by researchers who had no data on the specific sites to which HSM users would later apply those predictive models. The HSM users are generally engineers and planners, without formal statistical training, who would not generally be capable of developing custom models for each set of the sites they wish to apply the HSM to and, even if there were, would have no wish to spend the time and effort needed for model development each time they apply the HSM. The EB Method provides the most suitable tool for compensating for regression-to-the-mean that works in this context.

Each of the Part C chapters presents a four-step process for applying the EB Method. The EB Method assumes that the appropriate Part C predictive model (see Section 10.3.1 for rural two-lane, two-way roads, Section 11.3.1 for rural multilane highways, or Section 12.3.1 for urban and suburban arterials) has been applied to determine the predicted crash frequency for the sites that make up a particular project or facility for a particular past time period of interest. The steps in applying the EB Method are:

- Determine whether the EB Method is applicable, as explained in Appendix A.2.1.
- Determine whether observed crash frequency data are available for the project or facility for the time period for which the predictive model was applied and, if so, obtain those crash frequency data, as explained in Appendix A.2.2. Assign each crash instance to individual roadway segments and intersections, as explained in Appendix A.2.3.
- Apply the EB Method to estimate the expected crash frequency by combining the predicted and observed crash frequencies for the time period of interest. The site-specific EB Method, applicable when observed crash frequency data are available for the individual roadway segments and intersections that make up a project or facility, is presented in Appendix A.2.4. The project-level EB Method, applicable when observed crash frequency data are available only for the project or facility as a whole, is presented in Appendix A.2.5.
- Adjust the estimated value of expected crash frequency to a future time period, if appropriate, as explained in Appendix A.2.6.

Consideration of observed crash history data in the Part C predictive method increases the reliability of the estimate of the expected crash frequencies. When at least two years of observed crash history data are available for the facility or project being evaluated, and when the facility or project meets certain criteria discussed below, the observed crash data should be used. When considering observed crash history data, the procedure must consider both the existing geometric design and traffic control for the facility or project (i.e., the conditions that existed during the before period while the observed crash history was accumulated) and the proposed geometric design and traffic control for the project (i.e., the conditions that will exist during the after period, the period for which crash predictions are being made). In estimating the expected crash frequency for an existing arterial facility in a future time period where no improvement project is planned, only the traffic volumes should differ between the before and after periods. For an arterial on which an improvement project is planned, traffic volumes, geometric design features, and traffic control features may all change between the before and after periods. The EB Method presented below provides a method to combine predicted and observed crash frequencies.

A.2.1. Determine whether the EB Method is Applicable

The applicability of the EB Method to a particular project or facility depends on the type of analysis being performed and the type of future project work that is anticipated. If the analysis is being performed to assess the expected average crash frequency of a specific highway facility, but is not part of the analysis of a planned future project, then the EB Method should be applied. If a future project is being planned, then the nature of that future project should be considered in deciding whether to apply the EB Method.

The EB Method should be applied for the analyses involving the following future project types:

- Sites at which the roadway geometrics and traffic control are not being changed (e.g., the "do-nothing" alternative);
- Projects in which the roadway cross section is modified but the basic number of through lanes remains the same (This would include, for example, projects for which lanes or shoulders were widened or the roadside was improved, but the roadway remained a rural two-lane highway);
- Projects in which minor changes in alignment are made, such as flattening individual horizontal curves while leaving most of the alignment intact;
- Projects in which a passing lane or a short four-lane section is added to a rural two-lane, two-way road to increase passing opportunities; and
- Any combination of the above improvements.

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

- Projects in which a new alignment is developed for a substantial proportion of the project length; and
- Intersections at which the basic number of intersection legs or type of traffic control is changed as part of a project.

The reason that the EB Method is not used for these project types is that the observed crash data for a previous time period is not necessarily indicative of the crash experience that is likely to occur in the future after such a major geometric improvement. Since, for these project types, the observed crash frequency for the existing design is not relevant to estimation of the future crash frequencies for the site, the EB Method is not needed and should not be applied. If the EB Method is applied to individual roadway segments and intersections, and some roadway segments and intersections within the project limits will not be affected by the major geometric improvement, it is acceptable to apply the EB Method to those unaffected segments and intersections.

If the EB Method is not applicable, do not proceed to the remaining steps. Instead, follow the procedure described in the Applications section of the applicable Part C chapter.

A.2.2. Determine whether Observed Crash Frequency Data are Available for the Project or Facility and, if so, Obtain those Data

If the EB Method is applicable, it should be determined whether observed crash frequency data are available for the project or facility of interest directly from the jurisdiction's crash record system or indirectly from another source. At least two years of observed crash frequency data are desirable to apply the EB Method. The best results in applying the EB Method will be obtained if observed crash frequency data are available for each individual roadway segment and intersection that makes up the project of interest. The EB Method applicable to this situation is presented in Appendix A.2.4. Criteria for assigning crashes to individual roadway segments and intersections are presented in Appendix A.2.3. If observed crash frequency data are not available for individual roadway segments and intersections, the EB Method can still be applied if observed crash frequency data are available for the project or facility as a whole. The EB Method applicable to this situation is presented in Appendix A.2.5.

If appropriate crash frequency data are not available, do not proceed to the remaining steps. Instead, follow the procedure described in the Applications section of the applicable Part C chapter.

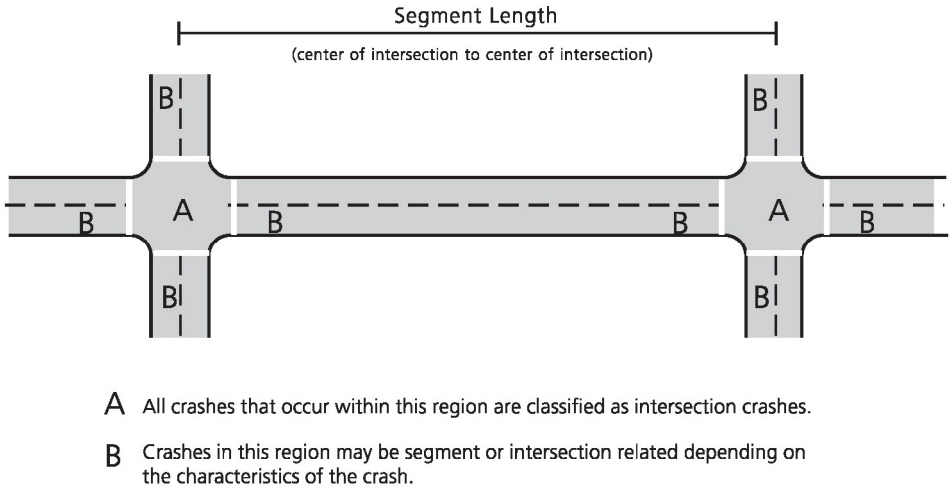
A.2.3. Assign Crashes to Individual Roadway Segments and Intersections for Use in the EB Method

The Part C predictive method has been developed to estimate crash frequencies separately for intersections and roadways segments. In the site-specific EB Method presented in Appendix A.2.4, observed crashes are combined with the predictive model estimate of crash frequency to provide a more reliable estimate of the expected average

crash frequency of a particular site. In Step 6 of the predictive method, if the site-specific EB Method is applicable, observed crashes are assigned to each individual site identified within the facility of interest. Because the predictive models estimate crashes separately for intersections and roadway segments, which may physically overall in some cases, observed crashes are differentiated and assigned as either intersection related crashes or roadway segment related crashes.

Intersection crashes include crashes that occur at an intersection (i.e., within the curb limits) and crashes that occur on the intersection legs and are intersection-related. All crashes that are not classified as intersection or intersection-related crashes are considered to be roadway segment crashes. Figure A-1 illustrates the method used to assign crashes to roadway segments or intersections. As shown:

- All crashes that occur within the curbline limits of an intersection (Region A in the figure) are assigned to that intersection.
- Crashes that occur outside the curbline limits of an intersection (Region B in the figure) are assigned to either the roadway segment on which they occur or an intersection, depending on their characteristics. Crashes that are classified on the crash report as intersection-related or have characteristics consistent with an intersection-related crash are assigned to the intersection to which they are related; such crashes would include rear-end collisions related to queues on an intersection approach. Crashes that occur between intersections and are not related to an intersection, such as collisions related to turning maneuvers at driveways, are assigned to the roadway segment on which they occur.



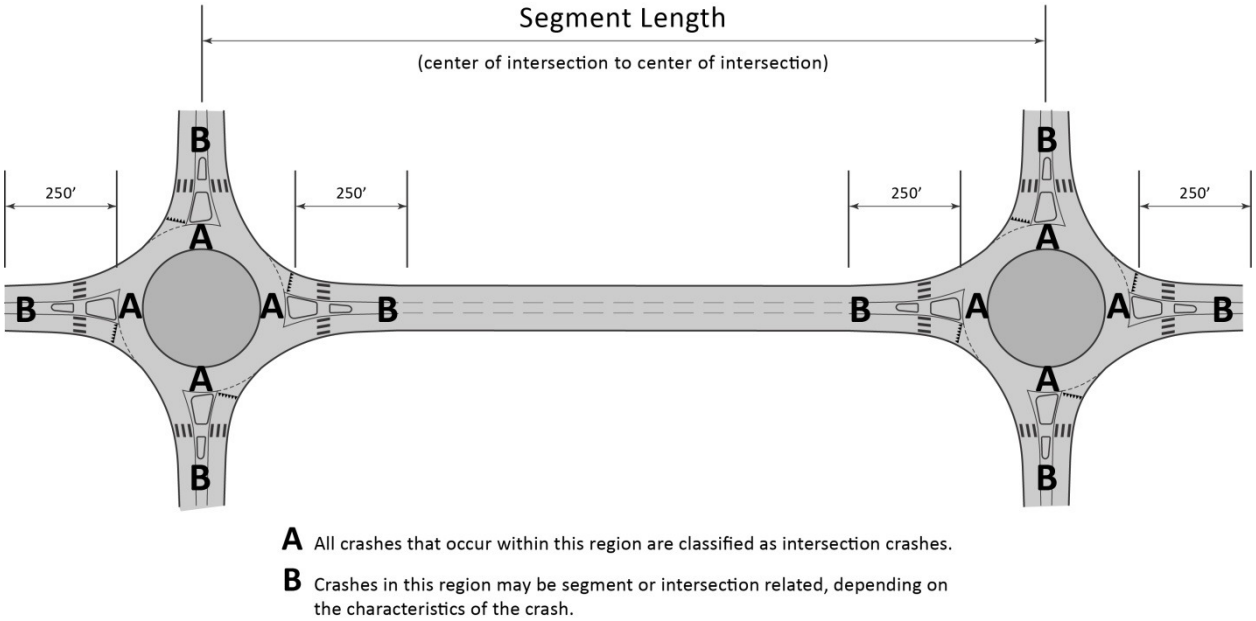


Figure A-1. Definition of Roadway Segments and Intersections

In some jurisdictions, crash reports include a field that allows the reporting officer to designate the crash as intersection-related. When this field is available on the crash reports, crashes should be assigned to the intersection or the segment based on the way the officer marked the field on the report. In jurisdictions where there is not a field on the crash report that allows the officer to designate crashes as intersection-related, the characteristics of the crash may be considered to make a judgment as to whether the crash should be assigned to the intersection or the segment. Other fields on the report, such as collision type, number of vehicles involved, contributing circumstances, weather condition, pavement condition, traffic control malfunction, and sequence of events can provide helpful information in making this determination.

If the officer's narrative and crash diagram are available to the user, they can also assist in making the determination. The following crash characteristics may indicate that the crash was related to the intersection:

- Rear-end collision in which both vehicles were going straight approaching an intersection or in which one vehicle was going straight and struck a stopped vehicle
- Collision in which the report indicates a signal malfunction or improper traffic control at the intersection

The following crash characteristics may indicate that the crash was not related to the intersection and should be assigned to the segment on which it occurred:

- Collision related to a driveway or involving a turning movement not at an intersection
- Single-vehicle run-off-the-road or fixed object collision in which pavement surface condition was marked as wet or icy and identified as a contributing factor

These examples are provided as guidance when an "intersection-related" field is not available on the crash report; they are not strict rules for assigning crashes. Information on the crash report should be considered to help make the determination, which will rely on judgment. The information needed for classifying crashes is whether each crash is, or is not, related to an intersection. The consideration of crash type data is presented here only as an example of one approach to making this determination.

Using these guidelines, the roadway segment predictive models estimate the average frequency of crashes that would occur on the roadway if no intersection were present. The intersection predictive models estimate the average frequency of additional crashes that occur because of the presence of an intersection.

A.2.4. Apply the Site-Specific EB Method

Equations A-4 and A-5 are used directly to estimate the expected crash frequency for a specific site by combining the predictive model estimate with observed crash frequency. The value of $N_{expected}$ from Equation A-4 represents the expected crash frequency for the same time period represented by the predicted and observed crash frequencies. $N_{predicted}$, $N_{observed,}$ and $N_{expected}$ all represent either total crashes or a specific severity level or collision type of interest. The expected average crash frequency considering both the predictive model estimate and observed crash frequencies for an individual roadway segment or intersection is computed as:

$$N_{expected} = w \times N_{predicted} + (1-w) \times N_{observed} \tag{A-4}$$

$$w = \frac{1}{1 + k \times (\sum_{\substack{\text{all study} \\ \text{years}}} N_{predicted})} \tag{A-5}$$

Where:

$N_{expected}$ = estimate of expected average crashes frequency for the study period;

$N_{predicted}$ = predictive model estimate of average crash frequency predicted for the study period under the given conditions;

$N_{observed}$ = observed crash frequency at the site over the study period;

w = weighted adjustment to be placed on the predictive model estimate; and

k = overdispersion parameter of the associated SPF used to estimate $N_{predicted}$.

When observed crash data by severity level is not available, the estimate of expected average crash frequency for fatal-and-injury and property-damage-only crashes is calculated by applying the proportion of predicted average crash frequency by severity level ($N_{predicted(FI)} / N_{predicted(total)}$ and $N_{predicted(PDO)} / N_{predicted(total)}$) to the total expected average crash frequency from Equation A-4.

Equation A-5 shows an inverse relationship between the overdispersion parameter, k, and the weight, w. This implies that when a model with little overdispersion is available; more reliance will be placed on the predictive model estimate, $N_{predicted}$, and less reliance on the observed crash frequency, $N_{observed}$. The opposite is also the case; when a model with substantial overdispersion is available, less reliance will be placed on the predictive model estimate, $N_{predicted}$, and more reliance on the observed crash frequency, $N_{observed}$.

It is important to note in Equation A-5 that, as $N_{predicted}$ increases, there is less weight placed on $N_{predicted}$ and more on $N_{observed}$. This might seem counterintuitive at first. However, this implies that for longer sites and for longer study periods, there are more opportunities for crashes to occur. Thus, the observed crash history is likely to be more meaningful and the model prediction less important. So, as $N_{predicted}$ increases, the EB Method places more weight on the number of crashes that actually occur, $N_{observed}$. When few crashes are predicted, the observed crash frequency, $N_{observed}$, is not likely to be meaningful, in statistical terms, so greater reliance is placed on the predicted crash frequency, $N_{predicted}$.

The values of the overdispersion parameters, *k*, for the safety performance functions used in the predictive models are presented with each SPF in Sections 10.6, 11.6, and 12.6.

Since application of the EB Method requires use of an overdispersion parameter, it cannot be applied to portions of the prediction method where no overdispersion parameter is available. For example, vehicle-pedestrian and vehicle-bicycle collisions are estimated in portions of Chapter 12 from adjustment factors rather than from models and should, therefore, be excluded from the computations with the EB Method. Chapter 12 uses multiple models with different overdispersion parameters in safety predictions for any specific roadway segment or intersection. Where observed crash data are aggregated so that the corresponding value of predicted crash frequency is determined as the sum of the results from multiple predictive models with differing overdispersion parameters, the project-level EB Method presented in Appendix A.2.5 should be applied rather than the site-specific method presented here.

Chapters 10, 11, and 12 each present worksheets that can be used to apply the site-specific EB Method as presented in this section.

Appendix A.2.6 explains how to update N_{expected} to a future time period, such as the time period when a proposed future project will be implemented. This procedure is only applicable if the conditions of the proposed project will not be substantially different from the roadway conditions during which the observed crash data was collected.

A.2.5. Apply the Project-Level EB Method

HSM users may not always have location specific information for observed crash data for the individual roadway segments and intersections that make up a facility or project of interest. Alternative procedures are available where observed crash frequency data are aggregated across several sites (e.g., for an entire facility or project). This requires a more complex EB Method for two reasons. First, the overdispersion parameter, *k*, in the denominator of Equation A-5 is not uniquely defined, because estimate of crash frequency from two or more predictive models with different overdispersion parameters are combined. Second, it cannot be assumed, as is normally done, that the expected average crash frequency for different site types are statistically correlated with one another. Rather, an estimate of expected average crash frequency should be computed based on the assumption that the various roadway segments and intersections are statistically independent ($r = 0$) and on the alternative assumption that they are perfectly correlated ($r = 1$). The expected average crash frequency is then estimated as the average of the estimates for $r = 0$ and $r = 1$.

The following equations implement this approach, summing the first three terms, which represent the three roadway-segment-related crash types, over the five types of roadway segments considered in the (2U, 3T, 4U, 4D, 5T) and the last two terms, which represent the two intersection-related crash types, over the four types of intersections (3ST, 3SG, 4ST, 4SG):

$$N_{\text{predicted (total)}} = \sum_{j=1}^{5} N_{\text{predicted } rmj} + \sum_{j=1}^{5} N_{\text{predicted } rsj} + \sum_{j=1}^{5} N_{\text{predicted } rdj} + \sum_{j=1}^{4} N_{\text{predicted } imj} + \sum_{j=1}^{4} N_{\text{predicted } isj} \tag{A-6}$$

$$N_{\text{observed (total)}} = \sum_{j=1}^{5} N_{\text{observed } rmj} + \sum_{j=1}^{5} N_{\text{observed } rsj} + \sum_{j=1}^{5} N_{\text{observed } rdj} + \sum_{j=1}^{4} N_{\text{observed } imj} + \sum_{j=1}^{4} N_{\text{observed } isj} \tag{A-7}$$

$$N_{\text{predicted } w0} = \sum_{j=1}^{5} k_{rmj}\ N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj}\ N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj}\ N_{rdj}^2 + \sum_{j=1}^{4} k_{imj}\ N_{imj}^2 + \sum_{j=1}^{4} k_{isj}\ N_{isj}^2 \tag{A-8}$$

$$N_{\text{predicted},w1} = \left[\begin{array}{l} \sum_{j=1}^{5} \sqrt{k_{rmj}\ N_{rmj}^2} + \sum_{j=1}^{5} \sqrt{k_{rsj}\ N_{rsj}^2} \\ + \sum_{j=1}^{5} \sqrt{k_{rdj}\ N_{rdj}^2} + \sum_{j=1}^{4} \sqrt{k_{imj}\ N_{imj}^2} \\ + \sum_{j=1}^{4} \sqrt{k_{isj}\ N_{isj}^2} \end{array} \right]^2 \tag{A-9}$$

$$w_0 = \frac{1}{1 + \frac{N_{\text{predicted } w0}}{N_{\text{predicted (total)}}}} \tag{A-10}$$

$$N_0 = w_0 N_{\text{predicted(total)}} + (1 - w_0) N_{\text{observed(total)}} \tag{A-11}$$

$$w_1 = \frac{1}{1 + \frac{N_{\text{predicted } w1}}{N_{\text{predicted (total)}}}} \tag{A-12}$$

$$N_1 = w_1 N_{\text{predicted(total)}} + (1 - w_1) N_{\text{observed(total)}} \tag{A-13}$$

$$N_{\text{expected}/comb} = \frac{N_0 + N_1}{2} \tag{A-14}$$

Where:

$N_{\text{predicted (total)}}$ = predicted number of total crashes for the facility or project of interest during the same period for which crashes were observed;

$N_{\text{predicted } rmj}$ = Predicted number of multiple-vehicle nondriveway collisions for roadway segments of type j, $j = 1...,5$, during the same period for which crashes were observed;

$N_{\text{predicted } rsj}$ = Predicted number of single-vehicle collisions for roadway segments of type j, during the same period for which crashes were observed;

$N_{\text{predicted } rdj}$ = Predicted number of multiple-vehicle driveway-related collisions for roadway segments of type j, during the same period for which crashes were observed;

$N_{\text{predicted } imj}$ = Predicted number of multiple-vehicle collisions for intersections of type j, $j = 1...,4$, during the same period for which crashes were observed;

$N_{\text{predicted } isj}$ = Predicted number of single-vehicle collisions for intersections of type j, during the same period for which crashes were observed;

$N_{\text{observed (total)}}$ = Observed number of total crashes for the facility or project of interest;

$N_{\text{observed } rmj}$ = Observed number of multiple-vehicle nondriveway collisions for roadway segments of type j;

$N_{\text{observed } rsj}$ = Observed number of single-vehicle collisions for roadway segments of type j;

$N_{\text{observed } rdj}$ = Observed number of driveway-related collisions for roadway segments of type j;

$N_{\text{observed } imj}$ = Observed number of multiple-vehicle collisions for intersections of type j;

$N_{\text{observed } isj}$ = Observed number of single-vehicle collisions for intersections of type j;

$N_{\text{predicted } w0}$ = Predicted number of total crashes during the same period for which crashes were observed under the assumption that crash frequencies for different roadway elements are statistically independent ($\rho = 0$);

k_{rmj} = Overdispersion parameter for multiple-vehicle nondriveway collisions for roadway segments of type j;

k_{rsj} = Overdispersion parameter for single-vehicle collisions for roadway segments of type j;

k_{rdj} = Overdispersion parameter for driveway-related collisions for roadway segments of type j;

k_{imj} = Overdispersion parameter for multiple-vehicle collisions for intersections of type j;

k_{isj} = Overdispersion parameter for single-vehicle collisions for intersections of type j;

$N_{\text{predicted } w1}$ = Predicted number of total crashes under the assumption that crash frequencies for different roadway elements are perfectly correlated ($\rho = 1$);

w_0 = weight placed on predicted crash frequency under the assumption that crash frequencies for different roadway elements are statistically independent ($r = 0$);

w_1 = weight placed on predicted crash frequency under the assumption that crash frequencies for different roadway elements are perfectly correlated ($r = 1$);

N_0 = expected crash frequency based on the assumption that different roadway elements are statistically independent ($r = 0$);

N_1 = expected crash frequency based on the assumption that different roadway elements are perfectly correlated ($r = 1$); and

$N_{\text{expected}/comb}$ = expected average crash frequency of combined sites including two or more roadway segments or intersections.

All of the crash terms for roadway segments and intersections presented in Equations A-6 through A-9 are used for analysis of urban and suburban arterials (Chapter 12). The predictive models for rural two-lane, two-way roads and multilane highways (Chapters 10 and 11) are based on the site type and not on the collision type. Therefore, only one of the predicted crash terms for roadway segments ($N_{\text{predicted } rmj}$, $N_{\text{predicted } rsj}$, $N_{\text{predicted } rdj}$), one of the predicted crash terms for intersections ($N_{\text{predicted } imj}$, $N_{\text{predicted } isj}$), one of the observed crash terms for roadway segments ($N_{\text{observed } rmj}$, $N_{\text{observed } rsj}$, $N_{\text{observed } rdj}$), and one of the observed crash terms for intersections ($N_{\text{observed } imj}$, $N_{\text{observed } isj}$) is used. For rural two-lane, two-way roads and multilane highways, it is recommended that the multiple-vehicle collision terms (with subscripts *rmj* and *imj*) be used to represent total crashes; the remaining unneeded terms can be set to zero.

Chapters 10, 11, and 12 each present worksheets that can be used to apply the project-level EB Method as presented in this section.

The value of $N_{\text{expected}/comb}$ from Equation A-14 represents the expected average crash frequency for the same time period represented by the predicted and observed crash frequencies. The estimate of expected average crash frequency of combined sites for fatal-and-injury and property-damage-only crashes is calculated by multiplying the proportion of predicted average crash frequency by severity level ($N_{\text{predicted}(FI)}/N_{\text{predicted(total)}}$ and $N_{\text{predicted}(PDO)}/N_{\text{predicted(total)}}$) to the total expected average crash frequency of combined sites from Equation A-14.

Appendix .2.6 explains how to update $N_{expected/comb}$ to a future time period, such as the time period when a proposed future project will be implemented.

A.2.6. Adjust the Estimated Value of Expected Average Crash frequency to a Future Time Period, If Appropriate

The value of the expected average crash frequency ($N_{expected}$) from Equation A-4 or $N_{expected/comb}$ from Equation A-14 represents the expected average crash frequency for a given roadway segment or intersection (or project, for $N_{expected/comb}$) during the before period. To obtain an estimate of expected average crash frequency in a future period (the after period), the estimate is corrected for (1) any difference in the duration of the before and after periods; (2) any growth or decline in AADTs between the before and after periods; and (3) any changes in geometric design or traffic control features between the before and after periods that affect the values of the CMFs for the roadway segment or intersection. The expected average crash frequency for a roadway segment or intersection in the after period can be estimated as:

$$N_f = N_p \left(\frac{N_{bf}}{N_{bp}}\right)\left(\frac{CMF_{1f}}{CMF_{1P}}\right)\left(\frac{CMF_{2f}}{CMF_{2P}}\right)\cdots\left(\frac{CMF_{nf}}{CMF_{np}}\right) \tag{A-15}$$

Where:

N_f = expected average crash frequency during the future time period for which crashes are being forecast for the segment or intersection in question (i.e., the after period);

N_p = expected average crash frequency for the past time period for which observed crash history data were available (i.e., the before period);

N_{bf} = number of crashes forecast by the SPF using the future AADT data, the specified nominal values for geometric parameters, and—in the case of a roadway segment—the actual length of the segment;

N_{bp} = number of crashes forecast by the SPF using the past AADT data, the specified nominal values for geometric parameters, and—in the case of a roadway segment—the actual length of the segment;

CMF_{nf} = value of the *n*th CMF for the geometric conditions planned for the future (i.e., proposed) design; and

CMF_{np} = value of the *n*th CMF for the geometric conditions for the past (i.e., existing) design.

Because of the form of the SPFs for roadway segments, if the length of the roadway segments are not changed, the ratio N_{bf}/N_{bp} is the same as the ratio of the traffic volumes, $AADT_f/AADT_p$. However, for intersections, the ratio N_{bf}/N_{bp} is evaluated explicitly with the SPFs because the intersection SPFs incorporate separate major- and minor-road AADT terms with differing coefficients. In applying Equation A-15, the values of N_{bp}, N_{bf}, CMF_{np}, and CMF_{nf} should be based on the average AADTs during the entire before or after period, respectively.

In projects that involve roadway realignment, if only a small portion of the roadway is realigned, the ratio N_{bf}/N_{bp} should be determined so that its value reflects the change in roadway length. In projects that involve extensive roadway realignment, the EB Method may not be applicable (see discussion in Appendix A.2.1).

Equation A-15 is applied to total average crash frequency. The expected future average crash frequencies by severity level should also be determined by multiplying the expected average crash frequency from the before period for each severity level by the ratio N_f/N_p.

In the case of minor changes in roadway alignment (i.e., flattening a horizontal curve), the length of an analysis segment may change from the past to the future time period, and this would be reflected in the values of N_{bp} and N_{bf}.

Equation A-15 can also be applied in cases for which only facility- or project-level data are available for observed crash frequencies. In this situation, $N_{\text{expected}/comb}$ should be used instead of N_{expected} in the equation.