HSM
Highway Safety Manual
AASHTO

HIGHWAY SAFETY MANUAL USER GUIDE

National Cooperative Highway Research Program 17-50
Lead States Initiative for Implementing the Highway Safety Manual

Highway Safety Manual

User Guide

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16. Abstract

The *Highway Safety Manual User Guide* is a user friendly document that helps safety analysts begin to use the *Highway Safety Manual* (HSM). The *Highway Safety Manual User Guide* is a companion document to the HSM and is used as a reference document. It is not a substitution for the HSM or a design guideline for safety projects. It is designed and written primarily for analysts with basic knowledge of the HSM and basic to moderate knowledge of highway safety analysis procedures, but it also contains insights that are useful to all practitioners.

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Dear Reader,

Thank you for taking time to learn and understand the American Association of State Highway and Transportation Officials (AASTHO) *Highway Safety Manual* (HSM) and how it can help you in your daily work as a transportation professional. The HSM provides tools to conduct quantitative safety analyses, allowing safety to be evaluated quantitatively alongside other transportation performance measures such as traffic operations, environmental impacts, and construction costs by the use of analytical tools for predicting the impact of transportation project and program decisions on road safety.

Dedicated agencies working on National Cooperative Highway Research Program (NCHRP) 17-50: *Lead State Initiative for Implementing the Highway Safety Manual* identified a need to encourage broader use of the HSM. They determined that a user guide focused on **simple, straightforward use of the manual** would introduce more professionals to the benefits of the HSM and ease practitioners in the use and application of the HSM. The *Highway Safety Manual User Guide* is a companion document to the AASTHO HSM and requires an HSM or HSM tools to complete the calculations identified in this guide. While the *Highway Safety Manual User Guide* details calculations so that the user understands the process, tools are available to automate more cumbersome or lengthy computations.

Currently, the AASTHO HSM 1st Edition focuses on several aspects of transportation safety: the roadway safety management process, predictive methods, and crash modification factors. Most safety professionals are already applying some or all of the roadway safety management process, which includes approaches for network screening, diagnosis, countermeasure selection, economic appraisal, project prioritization, and evaluation. Many current users of the HSM are using the predictive methods, which predict the number of crashes for rural two-lane facilities, rural multilane facilities, and urban and suburban arterials. Additional facility types are being added including freeways, interchanges, and roundabouts. Over 300 crash modification factors (CMFs) are included in the HSM, and additional CMFs are being developed and shared on the CMF Clearinghouse website, www.cmfclearinghouse.com.

A variety of guides and resources are available to assist all levels of agencies to incorporate the HSM principles into practice. This *Highway Safety Manual User Guide* focuses on getting the analyst started and on the right track for use of the AASTHO HSM 1st Edition. Additional information and resources are available at the AASTHO HSM website, www.highwaysafetymanual.org.

Sincerely,
NCHRP 17-50

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Disclaimer

This is an uncorrected draft as submitted by the research agency. The opinions and conclusions expressed or implied in the document are those of the research agency. They are not necessarily those of the TRB, the National Academies, or the program sponsors.

The *Highway Safety Manual User Guide* is not a legal standard of care as to the information contained herein. Instead, the *Highway Safety Manual User Guide* is a companion document to the AASHTO *Highway Safety Manual* (HSM) and is to be used as reference document. As a resource, the *Highway Safety Manual User Guide* does not supersede any publications, guidelines, manuals, and policies by AASHTO, FHWA, TRB, or other federal and state agencies. The user should check agency-specific approaches before applying the HSM and the *Highway Safety Manual User Guide* to estimate the crash frequency and crash severity for designated highway facilities.

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Abbreviations and Acronyms

2U	two-lane undivided arterials
3SG	signalized three-leg intersections
3ST	three-leg intersection with stop control
3T	three-lane arterials
4D	divided four-lane roadway segments
4SG	four-leg signalized intersection
4ST	four-leg intersection with stop control
4U	undivided four-lane roadway segment
5T	five-lane arterials
AASHTO	American Association of State Highway and Transportation Officials
AASHTO Redbook	*A Manual of User Benefit Analysis for Highway and Bus-Transit Improvements*
AADT	average annual daily traffic
$AADT_{major}$	average annual daily traffic on the major route
$AADT_{minor}$	average annual daily traffic for the minor route
BCR	benefit-cost ratio
C_i	intersection calibration factor
C_r	segment calibration factor
CMF	crash modification factor
DOT	Department of Transportation
EB	Empirical Bayes(ian)
EEACF	excess expected average crash frequency
FAQ	frequently asked question
FHWA	Federal Highway Administration
FI	fatal and injury
GIS	geographic information system
HFG	Human Factors Guide
HOV	high-occupancy vehicle
HSIP	Highway Safety Improvement Program
HSM	*Highway Safety Manual*
ID	identification number
IHSDM	Interactive Highway Safety Design Model

Int	intersection
k	overdispersion parameter
KABCO	Five-level injury severity scale. K = fatal injury; A = incapacitating injury; B = non-incapacitating evident injury; C = possible injury; O = property damage only
mph	miles per hour
MSE	multiple of standard error
NCHRP	National Cooperative Highway Research Program
NPV	net present value
N	number
PDO	property damage only
PV	present value
RHR	roadside hazard rating
RTM	regression- to-the-mean
RTOR	right-turn-on-red
SE	standard error
Seg	segment
SPF	safety performance function
SR	State Route
TRB	Transportation Research Board
TWLTL	two-way left-turn lane
vpd	vehicles per day
w	weighting factor

Introduction

1.1 Foreword

The American Association of State and Highway Transportation Officials (AASHTO) *Highway Safety Manual* (HSM), 1st Edition (published in 2010) represents the culmination of 10 years of research and development by an international ream of safety experts, academics, and practitioners. The HSM is a powerful tool that can be used to quantify the effects of changes to the roadway environment on safety. The HSM is a potentially transformative document for Departments of Transportation (DOTs) and other agencies responsible for the planning, design, construction, and operation of their highway systems. Under current practices, agency actions are based on results from proven, science-based tools to measure or estimate effects of traffic operations, on a myriad of environmental factors, and on the many aspects of capital and life-cycle costs. However, no proven and accepted tools or methods exist for understanding explicit safety effects. With publication of the HSM, DOTs and other agencies for the first time have access to a proven and vetted science-based means of characterizing the explicit safety effects (such as crash frequency and severity) of the decisions or actions of an agency.

The HSM can be used to identify sites with the most potential for crash frequency or severity reduction; identify contributing factors to crashes and mitigation measures; and estimate the potential crash frequency and severity on highway networks, among other uses. The HSM can also be used to measure, estimate, and evaluate roadways in terms of crash frequency and crash severity for corridor studies, traffic studies, environmental impact studies, design analysis, corridor planning studies, and more.

The HSM contains the most current and accepted knowledge and practices, and it covers the safety fundamentals, the roadway safety management process, predictive methods, and crash modification factors. The predictive methods focus on roadway segments and intersections for three facility types: rural, two-lane, two-way roads; rural multilane highways; and urban and suburban arterials. Research continues to advance the science of safety and predictive methods for additional facility types will be added as they become available. There is flexibility in the use of the HSM, as there are areas where the analyst has to make a judgment based on several factors including data availability, interpretation, and others. The AASHTO HSM website contains additional information including the Errata to the HSM.

1.2 Using the *Highway Safety Manual User Guide*

The *Highway Safety Manual User Guide* is a user-friendly document that helps safety analysts use the HSM. The *Highway Safety Manual User Guide* is a companion document to the HSM and is used as a reference document. It is not a substitution for the HSM or a design guideline for safety projects. It is designed and written primarily for analysts with basic knowledge of the HSM and basic to moderate knowledge of highway safety analysis procedures, but it also contains insights that are useful to all practitioners.

The *Highway Safety Manual User Guide* has three major sections: the HSM overview, integrating the HSM into the project development process, and frequently asked questions. The overview includes the theoretical background of the HSM. The section on integrating the HSM into the project development process includes well-designed examples with step-by-step procedures for HSM application. Readers are also encouraged to refer to the HSM as well as the following resources:

AASHTO HSM website: http://www.highwaysafetymanual.org/Pages/default.aspx

FHWA Office of Safety HSM website: http://safety.fhwa.dot.gov/hsm

Highway Safety Manual Overview

The HSM provides analytical tools and techniques for quantifying the potential effects on crashes as a result of decisions made in planning, design, operations, and maintenance. The information provided in the manual will assist agencies in their efforts to integrate safety into their decision-making processes. HSM users should have a safety knowledge base that includes familiarity with general highway safety principles, basic statistical procedures, and interpretation of results, along with suitable competence to exercise sound traffic safety and operational engineering judgment.

The HSM can be used for the following actions:

- Identify sites with the most potential for crash frequency or severity reduction
- Identify factors contributing to crashes and associated potential mitigation measures
- Conduct economic appraisals of safety countermeasures and project prioritization
- Evaluate the crash reduction benefits of implemented treatments
- Calculate the effect of various design alternatives on crash frequency and severity
- Estimate potential crash frequency and severity on highway networks
- Estimate the potential effect on crash frequency and severity of planning, design, operations, and policy decisions

The HSM can be used to consider safety in planning, design, construction/implementation, operations, and maintenance activities. The project development process was developed as a means to discuss the stages of a project from planning to post-construction operations and maintenance activities. The HSM is organized into four parts: HSM Part A – Introduction, Human Factors, and Fundamentals; HSM Part B – Roadway Safety Management Process; HSM Part C – Predictive Methods; and Part D – Crash Modification Factors.

2.1 HSM Part A: Introduction, Human Factors, and Fundamentals

HSM Part A has three chapters: HSM Chapter 1 - Introduction and Overview, HSM Chapter 2 – Human Factors, and HSM Chapter 3 – Fundamentals.

HSM Chapter 1 – Introduction and Overview describes the purpose and scope of the HSM, describes the basics of highway safety, and explains the relationship of the HSM to planning, design, operations, and maintenance activities. This chapter summarizes the different elements included in the manual, provides a general description of the purpose and scope of the HSM, and explains the relationship of the HSM to the project development process.

HSM Chapter 2 – Human Factors describes the core elements of human factors that affect the interaction of drivers and roadways, and provides an introduction to human factors to support the application of information presented in HSM Parts B, C, and D. Good understanding of this interaction allows highway agencies to plan and construct highways in a manner that minimizes human error and crashes. The NCHRP Report 600A: *Human Factors Guidelines for Road Systems* provides more detailed information and insights about driver's characteristics allowing analysts to more effectively bring into consideration the road users' capabilities and limitations into better roadway design and operational decisions.

HSM Chapter 3 – Fundamentals describes a variety of analysis approaches and methodologies as well as the background information needed to apply the predictive method, crash modification factors (CMFs), and evaluation methods provided in Parts B, C, and D of the HSM.

2.2 HSM Part B: Roadway Safety Management Process

HSM Part B discusses the process of monitoring and reducing crash frequency on existing roadway networks. The roadway safety management process consists of six steps: network screening (HSM Chapter 4), diagnosis (HSM Chapter 5), safety countermeasure selection (HSM Chapter 6), economic appraisal (HSM Chapter 7), project prioritization (HSM Chapter 8), and safety effectiveness evaluation (HSM Chapter 9).

HSM Part B allows users to:

- Identify and rank sites based on the potential for reducing average crash frequency
- Identify crash patterns with crash data, historical site data, and field conditions
- Identify the crash contributing factors at a site
- Select possible appropriate safety countermeasures to reduce the average crash frequency
- Evaluate the benefits and costs of the possible safety countermeasures
- Identify individual projects that are cost-effective or economically justified
- Identify improvement projects at specific sites and across multiple sites
- Evaluate effectiveness of a safety countermeasure in reducing crash frequency or severity

The roadway safety management process can be applied in different stages of the project development process, as shown in Table 1.

TABLE 1
Application of HSM Part B on Different Stages of Project Development Process

HSM Chapter	System Planning	Project Planning	Preliminary Design	Final Design	Construction/ Implementation	Operation	Maintenance
Chapter 4 – Network Screening	✓						
Chapter 5 – Diagnosis	✓	✓				✓	✓
Chapter 6 – Select Countermeasures	✓	✓	✓	✓	✓	✓	✓
Chapter 7 – Economic Appraisal	✓	✓	✓	✓	✓	✓	✓
Chapter 8 – Prioritize Projects	✓						
Chapter 9 – Safety Effectiveness Evaluation						✓	✓

Key concepts discussed in HSM Part B include:

- **Performance measure** is used to evaluate the potential to reduce crash frequency at a site.
- A **collision diagram** is a two-dimensional plan view representation to simplify the visualization of crash patterns that have occurred at a site within a given time period.
- A **countermeasure** is a roadway strategy intended to decrease crash frequency or severity, or both, at a site.
- The **Haddon Matrix** is used to identify crash contributing factors before, during, and after a crash from the perspective of human, vehicle, and roadway.
- **Regression-to-the-mean (RTM) or selection bias** refers to the bias created by the natural fluctuation of crash frequencies, which may lead one to draw incorrect conclusions about countermeasure effectiveness or sites with potential for improvement.
- The **net present value** (NPV) method is used to express the difference between discounted costs and discounted benefits of an individual improvement project in a single amount. The monetary costs and benefits are converted to a present value using a discount rate.
- A **benefit-cost ratio** (BCR) is the ratio of the present-value benefits of a project to the implementation costs of the project.

The following sections summarize the theoretical framework together with some important concepts and procedures for applying HSM Part B in the roadway safety management process. Refer to the relevant chapters in the HSM for more detailed information about roadway safety management.

2.2.1 HSM Chapter 4: Network Screening

HSM Chapter 4 provides a process for reviewing a transportation network to identify and rank sites based on the potential for reducing average crash frequency and/or crash severity. The network screening process is comprised of five steps: establish the focus of network screening, identify the network and reference population, select the performance measures, select screening method, and screen and evaluate the results.

The intended purpose of network screening can be either to identify sites with potential to reduce the average crash frequency or severity or focus on reducing a particular crash type, severity, frequency, or contributing factor. The selected network elements can then be identified and organized into different reference populations based on the roadway site characteristics (such as intersections, roadway segments). HSM Part B Section 4.2.2 (HSM p. 4-3) lists some potential characteristics that can be used to establish reference populations for intersections and roadway segments.

The third step in the network screening process is to select one or more performance measures to evaluate the potential for reducing the number of crashes or crash severity at a site. The performance measures can be selected based on data availability, RTM, or other statistical bias, and how the performance threshold is established (Figure 1). Figure 1 presents different performance measures in relative order of complexity, from least to most complex. For example, crash rate near the top of the list. Crash rate is often used because the data are readily available, but the results are not statistically stable. Excess Expected Average Crash Frequency with Empirical Bayes (EB) adjustments is more reliable but requires more data than for analysis based on crash rate.

Each of the performance metrics are described in HSM Part B Section 4.2.3 (HSM p. 4-6) along with the strengths and limitations of different performance measures. Refer to HSM Part B Section 4.4.2 for more details on data needs and calculation procedures for intersection performance measures.

More data; Account for RTM bias

Performance Measure	Accounts for RTM Bias	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 Severity Index	No	Yes
Critical Rate	Considers data variance; does not account for RTM bias	Yes
Excess Predicted Average Crash Frequency using Method of Moments	Considers data variance; does not account for RTM bias	Yes
Level of Service of Safety	Considers data variance; 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; does not account for RTM bias	Yes
Excess Proportions of Specific Crash Types	Considers data variance; does not account for RTM bias	Yes
Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency at the site
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency at the site
Excess Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency per year at the site

Greater Reliability

Source: *Highway Safety Manual*, 1st Edition

Figure 1: Stability of Performance Measures

The selected performance measure can be applied to roadway segments, intersections, and facilities using different screening methods. Generally, roadway segments can be screened using either a sliding-window or peak-searching method, while intersections can be screened using only a simple ranking method. Facilities that are a combination of intersections and roadway segments can be screened with a combination of screening methods. Only those screening methods that are consistent with the performance measures can be selected. Users can refer to HSM Part B Table 4-3 (p. 4-19) to determine the consistent screening method for the selected performance measure.

Finally, the performance measure and the screening method can be applied to one or more of the roadway segments, intersections, or facilities. A list of sites ordered according to the selected performance measure can be generated for the next step to identify locations for further review.

2.2.2 HSM Chapter 5: Diagnosis

The second step of the roadway safety management process, known as diagnosis, is to identify the contributing factors to the crashes; crash patterns; crash types; weather; potential road or roadside, vehicle, or human factors that may be relevant for the sites under investigation. Diagnosis is completed by reviewing existing crash data, assessing supporting documentation about the site conditions, and conducting an onsite field review.

It is recommended to use 3 to 5 years of crash data to evaluate crash locations, crash type, and crash severity to identify patterns. The crash data can be displayed using geographic information system (GIS) tools, linear graphs, bar charts, pie charts, or tabular summaries to better interpret and understand the data. Tools such as collision diagrams, condition diagrams, and crash mapping are described in HSM Part B Section 5.2.2 (HSM p. 5-4).

In addition to the safety data review, supporting documentation of site geometrics, traffic operations, site conditions, and uses should be evaluated. Documented information and personal testimony from local transportation enforcement and emergency services professionals may be useful for identifying potential crash contributing factors or to verify information gained from earlier data evaluations and analysis. HSM Part B Section 5.3 (HSM p. 5-8) lists examples of possible supporting documentation to be used during a site safety assessment, and HSM Appendix 5B (HSM p. 5-24) provides a list of questions and data to consider when reviewing past site documentation.

A site review is helpful to understand the area and potential issues better. Information gathered onsite might include geometric and traffic control information, as well as observation of traffic. A comprehensive field assessment involves travel through the site from all possible directions and modes, visiting the site during different times of the day and under different lighting/weather conditions. HSM Appendix 5C provides guidance on how to prepare for assessing field conditions. HSM Appendix 5D provides examples of field review checklists for different types of roadway environments.

After the field assessment, crash data review, and supporting documentation review are completed, the information can be compiled and used to identify trends or crash patterns. If trends or patterns are identified, safety countermeasures can be selected to mitigate or address the contributing factor(s) for crash occurrence.

2.2.3 HSM Chapter 6: Select Countermeasures

The contributing factors to observed crash patterns or types need to be identified before selecting appropriate safety countermeasures to address them. Multiple factors may be contributing to each identified crash pattern or types of crashes. To minimize the probability that a major contributing factor is overlooked, a broad range of possible contributing factors should be identified. Engineering judgment and statistical assessment are commonly applied to identify those factors that are expected to be the greatest contributors to each particular crash type or type after considering a broad range of contributing factors.

The Haddon Matrix (which divides the crash contributing factors into human, vehicle, and roadway categories) can be used to identify contributing factors for observed crash types or patterns. Potential contributing factors before, during, and after a crash are identified to determine the possible reasons of a crash. HSM Part B Section 6.2.2 (HSM p. 6-3) lists the most common contributing factors associated with a variety of crash types. Users can also refer to NCHRP Report 500: *Guidance for Implementation of the AASHTO Strategic Highway Safety Plan* for more details about the contributing factors for specific crash types. Each site and corresponding crash history are unique, and identification of crash contributing factors can only be completed by careful consideration of all the facts gathered during the diagnosis process.

Appropriate safety countermeasures can be selected after contributing factors have been identified. Countermeasure selection is used to develop potential engineering, education, enforcement, or emergency response treatments to address the contributing factors under consideration. Only crash-based countermeasures are covered in this edition of the *Highway Safety Manual User Guide*.
The FHWA CMF Clearinghouse contains a comprehensive list of CMFs (FHWA, 2013).

Engineering judgment and local knowledge are required when comparing contributing factors to potential safety countermeasures. When selecting countermeasures, users should also consider why the contributing factor(s) might be occurring, what could address the factor(s), and what is physically, financially, and politically feasible in the jurisdiction. For each specific site, one countermeasure or a combination of countermeasures could be considered to address the contributing factor. Users can refer to HSM Part D for countermeasures with quantitative CMFs.

In some cases, contributing factors may not be easily identifiable, even when there is a clear crash pattern. In such cases, a review of the road environment upstream or downstream of the site may provide some insights to whether there is any influence at the project location.

2.2.4 HSM Chapter 7: Economic Appraisal

The main objectives for the economic appraisal of a safety countermeasure or combination of countermeasures are to determine whether a project is economically justifiable, and determine which project or alternative is the most cost-effective. There are two methods for conducting economic appraisals, benefit-cost analysis and cost-effectiveness analysis. Both methods quantify the benefits of the proposed countermeasure(s). For benefit-cost analysis, the change in crash frequency or severity is converted to monetary values and compared to the cost of implementing the safety countermeasure. Additional project benefits such as savings in travel time or fuel consumption are common considerations during project evaluation, but the HSM only considers changes in crash frequency or severity. Users can refer to the AASHTO publication, *A Manual of User Benefit Analysis for Highway and Bus-Transit Improvements* (AASHTO Redbook) for considering other project benefits. For cost-effectiveness analysis, the change in crash frequency is compared directly to the project cost and is not quantified as monetary value. This approach provides a method to understand the value of countermeasure(s) implementation when the agency does not support the monetary crash costs values used to convert benefits to dollar value.

The HSM suggests that the change in average crash frequency caused by the application of a safety countermeasure should be estimated using the HSM Part C predictive method. The expected change in average fatal, injury, and property damage only (PDO) crash frequency can be converted to a monetary value using the societal crash costs. Users can apply the accepted state state/local societal crash cost by crash severity and collision type, if available. They can also refer to the FHWA report, *Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries* for other relevant values. HSM Table 7-1 (HSM p. 7-5) provides societal crash cost estimates by crash severity. The annual monetary value can be further converted to a present value using a discount rate and the service life of the safety countermeasures.

The project costs include the present value of right-of-way acquisition, construction, operation, and maintenance costs throughout the service life of the project. Users can refer to Chapter 6 of the AASHTO Redbook for additional guidance regarding the categories of costs and their proper treatments in an economic appraisal.

The net present value (NPV) or benefit-cost ratio (BCR) can be used to determine if a project is economically justifiable, and the cost-effectiveness index can be used to determine which project or alternative is most cost-effective. Users can refer to HSM Section 7.6 (HSM p. 7-8) for step-by-step instructions for each of these methods. After the economic appraisal is completed, the safety countermeasures for a given site can be ranked in descending or ascending order by project costs, BCR, cost-effectiveness index, and so forth.

2.2.5 HSM Chapter 8: Prioritize Projects

Project prioritization begins by reviewing potential projects for construction/implementation and sorts them based on the results of ranking and optimization processes. Project prioritization methods are primarily applicable to the development of optimal improvement programs for an entire roadway system or across multiple sites, but they can also be applied for alternative evaluation of a single site.

Chapter 8 provides three prioritization methods: ranking by economic effectiveness measures, incremental benefit-cost analysis, and optimization methods. The first two provide a list of projects prioritized based on specific criterion (refer to HSM Chapter 8.2 for additional details).

Optimization methods are used to prioritize projects, which are already determined to be economically justified. The prioritization is based on determining the most cost-effective project or set of projects that fit a given budget and other constraints. The HSM includes three specific optimization methods to be used to prioritize safety projects including Linear Programming optimization, Integer Programming Optimization and Dynamic Programming Optimization. HSM Appendix 8A (HSM p. 8-13) provides more detailed information about these methods. Most recently, the Integer Programming Optimization has become the most widely used method for project optimization.

All the project prioritization methods aforementioned are directly applicable when crash reduction is the only consideration. However, typical highway projects involve many other factors that influence project selection and prioritization. The HSM provides a reference to a class of decision-making algorithms known as multi-objective resource allocation, which can be used to quantify the effect of multiple factors i.e. safety in terms of reduction of crashes, traffic operations in terms of vehicle hours of delay reduced, air quality benefits in terms of the emissions reduced, etc.

Users can refer to HSM Table 8-1 (HSM p. 8-6) for selecting the appropriate project prioritization method. Computer software programs are available to prioritize projects or project alternatives efficiently and effectively.

Results from these prioritization methods can be incorporated into the decision-making process.

2.2.6 HSM Chapter 9: Safety Effectiveness Evaluation

Safety effectiveness evaluation is the final step of the roadway safety management process. It is the assessment of how crash frequency or severity has changed because of a specific treatment or safety countermeasure, or a set of treatments or projects, and how well funds have been invested in reducing crashes. When one treatment is applied to several similar sites, the safety effectiveness evaluation could also help estimate a CMF for the treatment. The safety effectiveness evaluation could be performed with the following objectives:

- Evaluate a single project at a specific site to document the safety effectiveness of that specific project
- Evaluate a group of similar projects to document the safety effectiveness of those projects
- Evaluate a group of similar projects for the specific purpose of quantifying a CMF for a countermeasure
- Assess the overall safety effectiveness of specific types of projects or countermeasures in comparison to their costs

Safety effectiveness evaluations may use several different types of performance measures, such as a percentage reduction in crashes, a shift in the proportion of crashes by collision type or severity level, a CMF for a treatment, or a comparison of the crash reduction benefits achieved in relation to the cost of a project or treatment. It should be pointed out that the evaluation is more complex than simply comparing before and after crash data at treatment sites because consideration should also be given to what changes in crash frequency would have occurred at the evaluation sites between the periods before and after the treatment, even if the treatment had not been implemented. To consider these impacts, most evaluations use data for both treatment and non-treatment sites and for periods both before and after implementation of the treatments.

Three basic study designs are used for safety effectiveness evaluation: observational before/after studies, observational cross-sectional studies, and experimental before/after studies. Selection of the appropriate study design for safety effectiveness evaluation depends on the nature of the treatment, the types of sites at which the treatment has been implemented, and the periods for which data are available for those sites. Refer to HSM Table 9-4 (HSM p. 9-6) for selecting the observational before/after evaluation method. Detailed procedures for implementing different safety evaluation methods including data needs and input, pre-evaluation activities, and computational procedures are provided in HSM Part B Section 9.4 (HSM p. 9-7).

2.3 HSM Part C: Predictive Method

2.3.1 Overview of the Predictive Method

HSM Part C provides a predictive method for calculating the predicted and/or expected average crash frequency of a network, facility, or individual site and introduces the concept of safety performance functions (SPFs). These methods focus on the use of statistical models to address the inherent randomness in crashes. The chapters in HSM Part C provide the predictive method for roadway segments and intersections for the following facility types, as listed in Table 2.

TABLE 2
HSM Part C Chapters

HSM Chapter	Undivided Roadway Segments	Divided Roadway Segments	Intersections			
			Stop Control on Minor Leg(s)		Signalized	
			Three-Leg	Four-Leg	Three-Leg	Four-Leg
10 – Predictive Method for Rural, Two-Lane, Two-Way Roads	✓		✓	✓		✓
11 – Predictive Method for Rural Multilane Highways	✓	✓	✓	✓		✓
12 – Predictive Method for Urban and Suburban Arterials	✓	✓	✓	✓	✓	✓

Predictions of average crash frequency as a function of traffic volume and roadway characteristics can be used for making decisions relating to designing, planning, operating, and maintaining roadway networks. The approach is applicable for both safety-specific studies and as an element of a more traditional transportation study or environmental analysis.

The predictive method has been outlined in 18 steps in a flowchart format and discussed in detail in HSM Part C, Section C.6 (HSM p. C-12). This method provides detailed guidance on dividing a facility into individual sites; selecting the period of analysis; obtaining geometric data and observed crash data; and applying the predictive models and EB adjustment method. Where a facility consists of a number of contiguous sites, or crash estimation is desired for a period of several years, some steps may be repeated. Depending on the roadway or roadside conditions proposed by an alternative, the use of the EB method may not be appropriate.

The predictive method can be used to assess crashes for existing conditions, alternatives to existing conditions, or proposed new roadways. The predicted average crash frequency can be modeled with the geometric design, traffic control features, and traffic volumes of that site. When observed crash frequency is available, the expected average crash frequency could be determined with the EB method. Figure 2 lists common scenarios in which the HSM predictive method or EB method could be used to model the predicted or expected average crash frequency. There are situations when the expected average crash frequency cannot be computed, such as when crash data is not available or is considered unreliable; when a project on new alignment or new location is contemplated; and when a substantial change to a location or facility is being considered such that the observed crash data are irrelevant. An example of this is a two-lane rural road being reconstructed as a four-lane divided highway. A detailed explanation of **observed** crash frequency, **predicted** average crash frequency, and **expected** average crash frequency is provided in Section 2.3.3 of this guide, and in the Frequently Asked Questions (FAQ) section.

Figure 3 describes the facility type definitions included in each HSM Part C chapter.

Scenarios for HSM Predictive Method Application

- Existing traffic under past or future traffic volume
- Alternative designs for an existing facility under past or future traffic volumes
- Designs for a new facility under future (forecast) traffic volumes
- Estimated effectiveness of countermeasures after a period of implementation
- Estimated effectiveness of proposed countermeasures on an existing facility (prior to implementation)

Figure 2: Scenarios for HSM Predictive Method Application

HSM Part C Chapters and Facility Site Types

Part C Chapter	Facility Types
Chapter 10 - Predictive Method for Rural Two-Lane, Two-Way Roads	• All rural highways with two lanes and two-way traffic operation. This includes two-lane highways with center two-way left-turn lanes (TWLTL) and sections with passing or climbing lanes. • Three- and four-leg intersections with minor-road stop control and four-leg signalized intersections.
Chapter 11 - Predictive Method for Rural Multilane Highways	• All rural multilane highways without full access control with four travel lanes, except for two-lane highways with side-by-side passing lanes. • Three- and four-leg intersections with minor-road stop control and four-leg signalized intersections.
Chapter 12 - Predictive Method for Urban and Suburban Arterials	• All arterials without full access control with two or four through lanes in urban and suburban areas. • Three- and four-leg intersections with minor-road stop control or traffic signal control.

Figure 3: HSM Part C Chapters and Facility Types

2.3.2 HSM Part C Relationship to HSM Parts A, B, and D

HSM Part A – Introduction, Human Factors, and Fundamentals. This section presents background information to understand the methods provided in the HSM to analyze and evaluate crash frequencies. It also includes information related to SPFs and CMFs. Good understanding of the fundamentals of SPFs and CMFs is recommended before using HSM Part C.

HSM Part B – Roadway Safety Management Process. Material presented in this section is used for monitoring, improving, and maintaining an existing roadway network. Applying methods from HSM Part B can help identifying sites that exhibit more crashes than what would be expected; diagnosing crash patterns at specific sites; selecting appropriate safety countermeasures to mitigate crashes; benefits and costs of potential alternatives; establishing projects prioritization; and assessing projects effectiveness after implementation. The predictive method in HSM Part C provides tools to estimate the predicted and/or expected average crash frequency for application in HSM Chapter 4, Network Screening, and HSM Chapter 7, Economic Appraisal.

HSM Part D – Crash Modification Factors. The CMFs in HSM Part D present information regarding the effects of various safety treatments that are used to quantify the change in average crash frequency and the statistical reliability of those countermeasures. Although some HSM Part D CMFs are included in HSM Part C for use with specific SPFs, only the CMFs included in HSM Part C are intended to be used with the models in HSM Part C.

2.3.3 Predicted versus Expected Crash Frequency

The HSM predictive method can calculate both the **predicted** crash frequency and the **expected** crash frequency under different scenarios. The **predicted** average crash frequency of an individual site is the crash frequency calculated with the SPFs and CMFs based on the geometric design, traffic control features, and traffic volume of the site. This method will be used to estimate the crash frequency for a past or future year, or when the **observed** crash frequency is not available. The **observed** crash frequency refers to the historical crash data observed/reported at the site during the period of analysis.

When the **observed** crash frequency is available, the **expected** crash frequency can be calculated. The **expected** crash frequency uses the EB method to combine the **observed** crash frequency with the **predicted** average crash frequency to produce a more statistically reliable measure. A weighted factor is applied to both estimates; this reflects the statistical reliability of the SPFs. The **expected** crash frequency is the long-term average crash frequency that would be expected from the specific site and is more statistically reliable compared with the predicted crash frequency.

Figure 4 illustrates the **observed**, **predicted,** and **expected** average crash frequencies for a site.

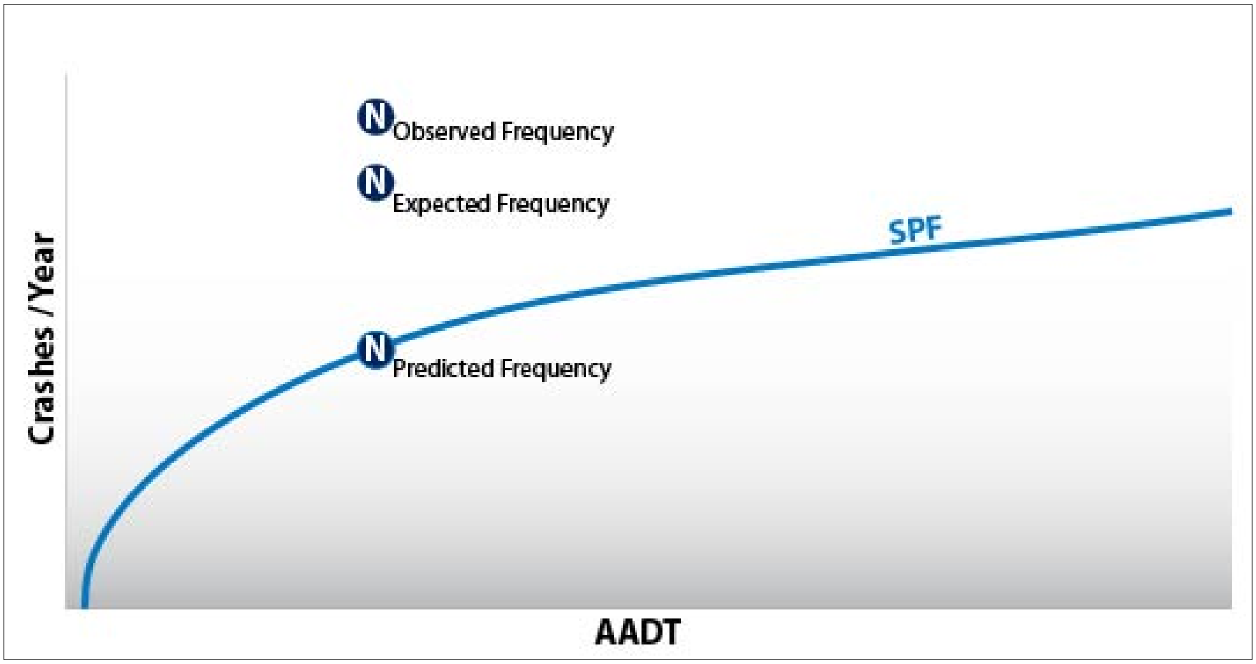


Figure 4: Illustration of Observed, Predicted, and Expected Crash Frequency Estimates

2.3.4 Safety Performance Functions

SPFs are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections. SPFs are developed through statistical regression techniques using historical crash data collected over a number of years at "base" sites with similar characteristics. The regression parameters are determined with the assumption that crash frequencies follow a negative binomial distribution, which is an extension of the Poisson distribution typically used for count data. The negative binomial regression allows the variance to differ from the mean through the incorporation of an additional parameter called the dispersion parameter. In cases where the variance is greater than the mean, the data is said to be overdispersed. The overdispersion parameter has positive values. This value is used to compute a weighted adjustment factor that is applied in the EB method described in HSM Section C.6.6. (HSM p. C-18)

The dependent variable is the predicted average crash frequency for a facility type under base conditions. The independent variables are the segment length and average annual daily traffic (AADT) (for roadway segments) or the AADT on the major and minor roads (for intersections). Figure 5 shows a sample SPF developed for the Colorado Department of Transportation.

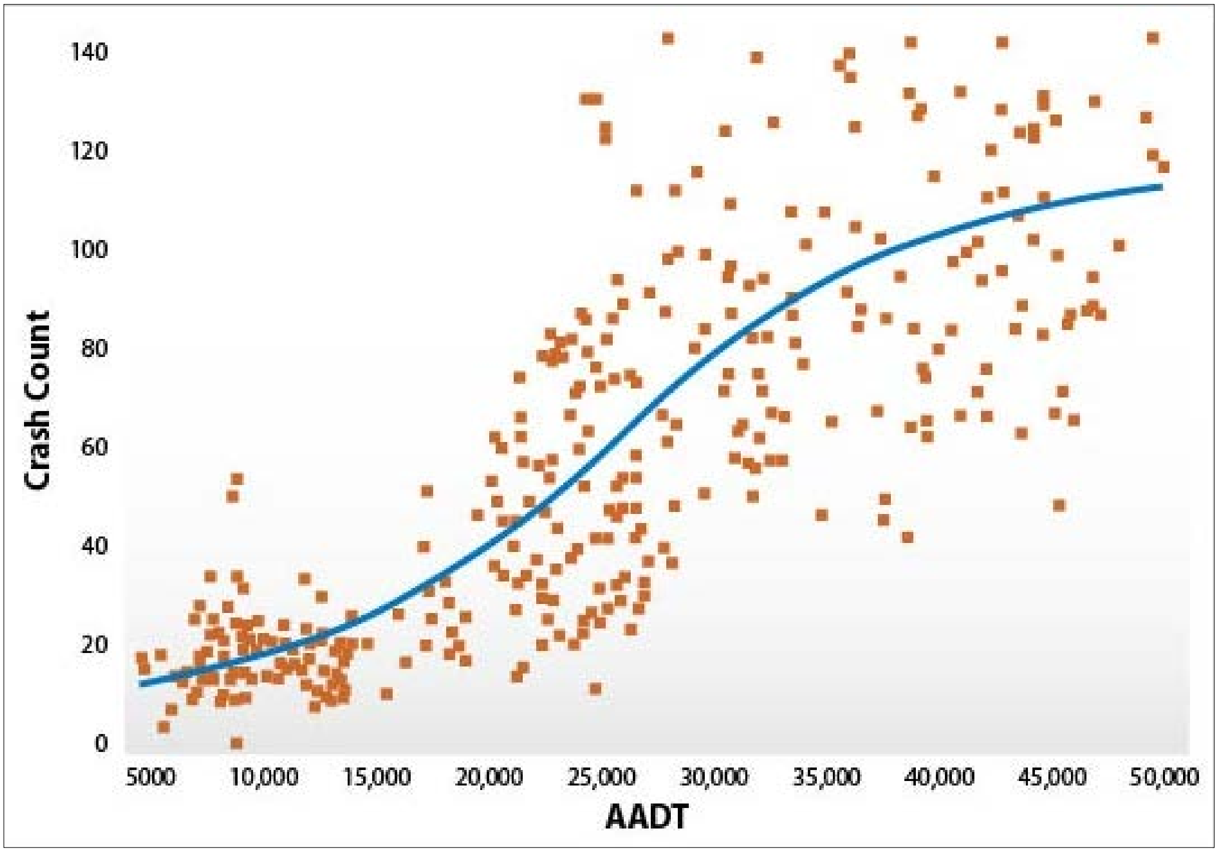


Figure 5: Sample SPF – Colorado Department of Transportation (Source: Kononov, 2011)

Multivariate models, or Level II SPFs, incorporate a variety of variables other than traffic volume only. Variables such as roadway geometry elements, access density, and weather can be used to estimate the dependent variable.

The SPFs are developed for total crash frequency including all crash severity levels and, in some cases, collision types. However, SPFs for specific collision types and/or crash severity levels are also developed in some cases (see Table 3 for the list of SPFs included in HSM Part C). The user should select the appropriate SPFs when calculating the crash frequency for a specific site.

TABLE 3
List of SPFs in HSM Part C

Chapter	Facility Type	SPF for Collision Type	SPF for Crash Severity Level
Chapter 10	Roadway Segment	• All collision types	• All severity levels
	Intersection	• All collision types	• All severity levels
Chapter 11	Roadway Segment	• All collision types	• All severity levels • Fatal-and-injury crashes
	Intersection	• All collision types	• All severity levels • Fatal-and-injury crashes
Chapter 12	Roadway Segment	• Single-vehicle crashes	• All severity levels • Fatal-and-injury crashes • PDO crashes

TABLE 3
List of SPFs in HSM Part C

Chapter	Facility Type	SPF for Collision Type	SPF for Crash Severity Level
		• Multiple-vehicle non-driveway collision	• All severity levels • Fatal-and-injury crashes • PDO crashes
		• Multiple-vehicle driveway-related collision	• All severity levels
		• Vehicle-pedestrian collision	• All severity levels
		• Vehicle-bicycle collision	• All severity levels
	Intersection	• Multiple-vehicle collision	• All severity levels • Fatal-and-injury crashes • PDO crashes
		• Single-vehicle crashes	• All severity levels • Fatal-and-injury crashes • PDO crashes
		• Vehicle-pedestrian collision	• All severity levels
		• Vehicle-bicycle collision	• All severity levels

2.3.5 Crash Modification Factors

HSM Part C base models are developed using a given set of site characteristics and are used to estimate the predicted average crash frequency. The Part C CMFs are used to adjust the base models to local conditions. A CMF represents the relative change in estimated average crash frequency due to differences for each specific condition and provides an estimate of the effectiveness of the implementation of a particular countermeasure. For example, paving gravel shoulders, adding a left-turn lane, or increasing the radius of a horizontal curve.

Part D includes all CMFs in the HSM. Some Part D CMFs are included in Part C for use with specific SPFs, since they are specific to the SPFs developed in those chapters. The remaining Part D CMFs can be used with the outcomes of the predictive method to estimate the change in crash frequency for a given countermeasure under the conditions described in HSM Section C.7 (HSM p. C-19). See also section 2.3.9 of this guide.

All CMFs included in the HSM were selected through an expert panel review process and contain a combination of base conditions; setting and road type; AADT range in which the CMF is applicable; crash type and severity addressed by the CMF; CMF value; standard error; CMF source; and attributes of the original studies (if available). Part C CMFs have the same base conditions as their corresponding SPFs in Part C.

2.3.6 Weighting Using the Empirical Bayes Method

The EB method can be used to calculate the expected average crash frequency for past and future periods and applied at either the site or the project level. Application at the project level is done when users do not have location-specific observed crash data for the individual roadway segments or intersections that are part of the project and when data is aggregated across all sites.

The EB method combines the observed crash frequency with the predicted average crash frequency. This adjustment is only applied when observed crash data for a minimum of 2 years are available for either the specific site or the entire facility.

The EB method uses a weighted factor (w) which is a function of the SPF's overdispersion parameter (k) to combine the two estimates. As the value of the overdispersion parameter increases, the weighted adjustment factor decreases; thus, more emphasis is placed on the observed/reported crashes rather than the SPF predicted crash frequency. This estimate depends on the data characteristics (dispersed versus small overdispersion) used to develop the prediction models. Additional details can be found in HSM Part C, Appendix A.2 (HSM p. A-15)

2.3.7 Calibration versus Development of Local SPFs

The predictive models in HSM Part C are composed of three basic elements: SPFs, CMFs, and a calibration factor. The HSM SPFs were developed using data from a subset of states. Difference in crash data quality, roadway inventory, traffic counts, crash reporting thresholds, and weather conditions are some of the factors that vary among states that may affect the prediction of the number and severity of crashes. Therefore, for the predictive method to provide results that are reliable for each jurisdiction that uses them, it is important that the SPFs in HSM Part C are calibrated to account for local conditions. Several DOTs have calibrated or are in the process of calibrating the HSM default SPFs. Some agencies are developing jurisdiction-specific SPFs using their own data to further enhance the reliability of the HSM Part C predictive method. The sophistication of state-specific SPFs may vary and require additional statistical analysis expertise. Calibration and SPF development are prepared by the agency rather than by individual users.

During the calibration development period, HSM users can still use the HSM Part C to assess relative differences among alternatives within the same facility type and control type. However, the output from an HSM SPF cannot be used to describe an actual prediction, as it lacks the necessary calibration factor.

2.3.8 Crash Severity and Collision Type Distribution for Local Conditions

Application of the HSM SPFs results in total predicted crash frequency or by specific severity. The HSM also provides distributions of crash frequency by severity and collision type. These tables may be used to separate the crash frequencies into different severity levels and collision types. These distributions can be used in cases where there is concern regarding certain collision types or crash severity levels.

Users can refer to SPFs for specific injury levels or SPFs for total crashes combined with crash severity and type distribution to estimate specific injury levels. The crash severity and collision-type distribution tables in the HSM were developed using specific state data. Agencies may provide jurisdiction-specific tables to be used instead of the HSM default tables. Application of agency-specific tables may provide predictions that are more accurate.

2.3.9 Methods for Estimating the Safety Effectiveness of a Proposed Project

The following are the four HSM methods for estimating change in expected average crash frequency for a project, listed in order of predictive reliability:

- Method 1: Apply the HSM Part C predictive method to calculate the predicted average crash frequency of existing and proposed conditions.
- Method 2: Apply the HSM predictive method to calculate the predicted average crash frequency of existing conditions, and application of appropriate HSM Part D CMFs to calculate the safety performance of the proposed condition.

- Method 3: For cases where HSM Part C predictive method is not available, but an SPF for a facility not included in the HSM is available. Apply the SPF to calculate the predicted average crash frequency of existing conditions, and apply an appropriate HSM Part D CMF to estimate the safety performance of the proposed condition. A locally derived project CMF can also be used as part of this method.
- Method 4: Apply the observed crash frequency to calculate the expected average crash frequency of existing conditions, and apply the appropriate HSM Part D CMF to the existing conditions expected average crash frequency to obtain the expected average crash frequency of the proposed condition.

In all four methods, the delta between existing and proposed expected average crash frequencies is used as the project effectiveness estimate.

2.3.10 Limitations of the HSM Predictive Method

The HSM predictive method has been developed using U.S. roadway data. The predictive models incorporate the effects of several geometric design elements and traffic control features. Variables not included in the predictive models were not necessarily excluded because they have no effect in crash frequency; it may merely mean that the effect is not fully known or has not been quantified at this time.

In addition to the geometric features, the predictive method incorporates the effect of non-geometric factors in a general sense. One example of this limitation is the variation in driver populations. Different sites experience significant variations in demographics and behavioral factors including age distribution, years of driving experience, seatbelt usage, and alcohol usage. The calibration process accounts for the statewide influence of such crash factors on crash occurrence; however, these factors are not taken into account in site-specific variations, which may be substantial. The case is similar for the effect of weather, which might be incorporated through the calibration process.

Another factor not included in the predictive method is the effect of traffic volume variations throughout the day or proportions of different vehicle types. This is mainly because these effects are not fully understood.

Lastly, the predictive method treats the effects of individual geometric design and traffic control features as independent of one another and does not account for potential interactions between them. It is likely that such interactions exist, and, ideally, they should be accounted for in the predictive models. At present, such interactions are not fully understood and are difficult to quantify.

2.3.11 HSM Part C Summary

HSM Part C provides the basic methodology for calculating the predicted and/or expected crash frequency for selected highway facilities under given traffic and geometric conditions.
The following concepts (Figure 6) were incorporated in the procedure:

- Safety performance functions: SPFs are regression equations that are used to calculate the predicted crash frequency for a specific site (with specified base conditions) as a function of annual average daily traffic, and (in the case of roadway segments) the segment length.

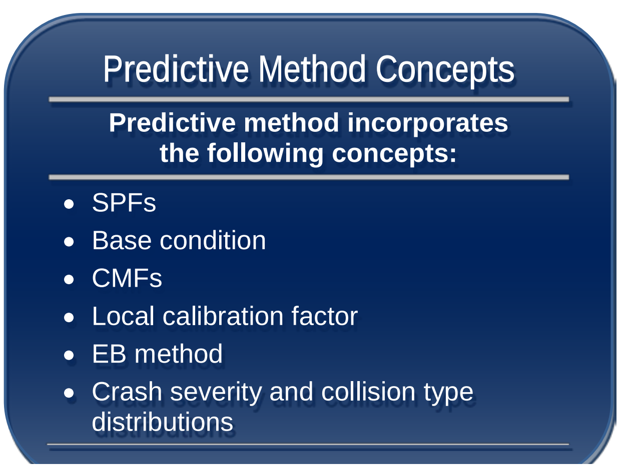


Figure 6: Predictive Method Main Concepts

- Base condition: A specific set of geometric design and traffic control features, under which the SPFs were developed.
- Crash modification factors: HSM Part C CMFs are used to account for the safety effects of differences between the base conditions and the site-conditions of the highway facilities under investigation.
- Local calibration factor: It is used to account for the differences between jurisdictions for which the SPFs were developed. Differences could be associated to factors such as driver population, climate, weather, and/or crash reporting thresholds.
- Empirical Bayes Method: The EB method is used to combine the predicted average crash frequency with the observed crash frequency to obtain the expected average crash frequency for the selected highway facilities.
- Crash severity and collision type distributions: These distributions are applied in the predictive method to determine the crash frequency under specific crash severity and collision types. The crash severity and collision type distribution tables were derived from HSM-related research projects. Some of these distributions can be replaced with locally derived values.

2.3.12 HSM Chapter 10: Predictive Method for Rural Two-Lane, Two-Way Roads

HSM Chapter 10 provides a methodology to estimate the predicted and/or expected average crash frequency, crash severity, and collision types for rural two-lane, two-way facilities. 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, or new sites.

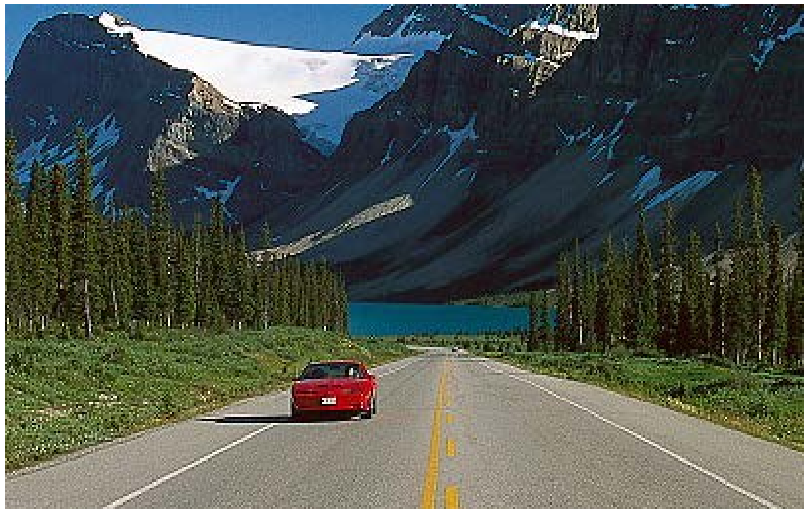


Figure 7: Rural Two-Lane, Two-Way Road

This chapter is applicable to all rural highways with two-lane and two-way traffic operation that do not have access control and are outside of cities or towns with a population greater than 5,000 people (HSM Section 10.3, p. 10-2). Additionally, it can be used on two-lane, two-way highways with center TWLTLs; and with two-lane highways with passing lanes, climbing lanes, or short segments of four-lane cross-sections —*up to 2 miles in length*—where additional lanes are provided to enhance passing opportunities. Longer sections can be addressed with the rural multilane highway procedures outlined in HSM Chapter 11. Figure 7 shows a typical example of a rural two-lane, two-way roadway.

This chapter also addresses three- and four-leg intersections with minor-road stop control and four-leg signalization on all the roadway cross sections. Table 4 includes the site types on rural two-lane, two-way roads for which SPFs have been developed for predicting average crash frequency, severity, and collision type. Figure 8 lists the facility types and definitions provided in HSM Chapter 10.

TABLE 4
Roadway Segment and Intersection Types and Descriptions for Rural Two-Lane, Two-Way Roads

Facility Type	Site Types with SPFs in Chapter 10
Roadway Segments	Undivided rural two-lane, two-way roadway segments (2U)
Intersections	Unsignalized three-leg (stop control on minor-road approaches) (3ST)
	Unsignalized four-leg (stop control on minor-road approaches) (4ST)
	Signalized four-leg (4SG)

Rural Two-Lane, Two-Way Roads Facility Type Definitions

Facility Type	Definition
Undivided roadway segment	A roadway consisting of two lanes with a continuous cross-section providing two directions of travel in which the lanes are not physically separately by either distance or a barrier. Additionally, segments with a TWLTL or passing lanes are included as part of this definition.
Unsignalized three-leg intersection with stop control	An intersection of a rural two-lane, two-way road and a minor road. A stop sign is provided on the minor road approach to the intersection only.
Unsignalized four-leg intersection with stop control	An intersection of a rural two-lane, two-way road, and two minor roads. A stop sign is provided on both minor road approaches to the intersection.
Signalized four-leg intersection	An intersection of a rural two-lane, two-way road and two other rural two-lane, two-way roads. Signalized control is provided at the intersection by traffic lights.

Figure 8: Rural Two-Lane, Two-Way Roads Facility Types and Definitions

HSM Chapter 10 also provides guidance on how to define roadway segments and intersections (HSM Section 10.5, p. 10-11).

A roadway segment is defined as a section of continuous traveled way that provides two-way operation of traffic uninterrupted by an intersection, and comprises homogeneous geometric and traffic control features. A segment begins and ends at the center of bounding intersections or where there is a change in homogeneous roadway characteristics. When a roadway segment begins or ends at an intersection, the length of the roadway segment is measured from the center of the intersection.

An intersection is defined as the junction of two or more roadway segments. The intersection models estimate the average crash frequency that occurs at the intersection (Region A in Figure 9), and intersection-related crashes that occur on the intersection legs (Region B in Figure 9).

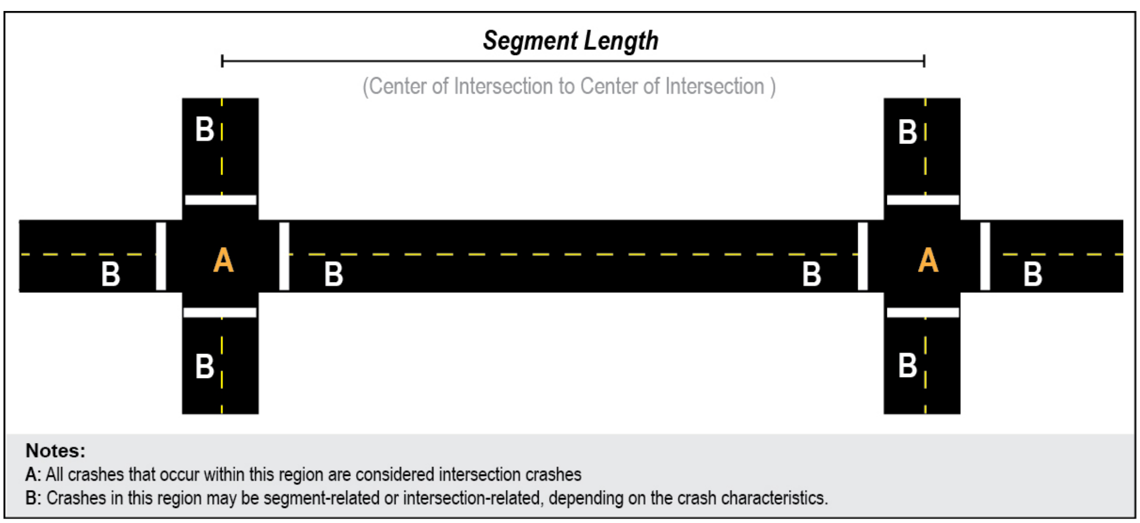


Figure 9: Rural Two-Lane, Two-Way Roads – Definition of Roadway Segments and Intersections

2.3.13 Calculating the Crash Frequency for Rural Two-Lane, Two-Way Roads

HSM Chapter 10 provides the methodology for calculating the predicted and/or expected crash frequency for roadway segments and intersections on rural two-lane, two-way roads. The calculation is for a given period of time during which the geometric design and traffic control features are unchanged and traffic volumes are known. The entire process could be divided into the following steps:

1. Predicted crash frequency under base conditions
2. Predicted crash frequency under site conditions
3. Expected crash frequency with Empirical Bayes method
4. Crash frequency under different collision types and crash severity levels

Step 1: Predicted Crash Frequency under Base Conditions

The predicted average crash frequency for the roadway segments and intersections under base condition could be determined by replacing the AADT and segment length (for roadway segments) or the AADTs for major and minor roads (for intersections) in SPFs with site-specific values. Table 5 lists the SPFs for different facility types included in HSM Chapter 10 and the applicable AADT ranges for the SPFs. Only application to sites within the AADT ranges could provide reliable results.

TABLE 5
Rural Two-Lane, Two-Way Roads SPFs in HSM Chapter 10

Facility Type	HSM Equation	AADT Range
Rural two-lane, two-way roadway segments	Equation 10-6	0 to 17,800 vpd
Three-leg stop-controlled intersection	Equation 10-8	$AADT_{major}$: 0 to 19,500 vpd
		$AADT_{minor}$: 0 to 4,300 vpd
Four-leg stop-controlled intersection	Equation 10-9	$AADT_{major}$: 0 to 14,700 vpd
		$AADT_{minor}$: 0 to 3,500 vpd
Four-leg signalized intersection	Equation 10-10	$AADT_{major}$: 0 to 25,200 vpd
		$AADT_{minor}$: 0 to 12,500 vpd

Notes:
$AADT_{major}$ = average annual daily traffic on the major route
$AADT_{minor}$ = average annual daily traffic on the minor route
vpd = vehicles per day

Step 2: Predicted Crash Frequency under Real Conditions

Each SPF listed in Table 5 is used to estimate the predicted crash frequency of a roadway segment or intersection under base conditions, which is later adjusted to site-specific conditions. The base conditions are a specific set of geometric design and traffic control features under which the SPFs were developed and are not necessarily the same for all facilities. The base conditions for roadway segments and intersections on rural two-lane, two-way roads are listed in Figure 10.

Rural Two-Lane, Two-Way Roads Base Conditions

Roadway Segments	Intersections
• Lane width: 12 feet • Shoulder width: 6 feet • Shoulder type: Paved • Roadside hazard rating: 3 • Driveway density: 5 driveways per mile • No horizontal curvature • No vertical curvature • No centerline rumble strips • No passing lanes • No two-way left-turn lanes • No lighting • No automated speed enforcement • Grade level: 0%	• Intersection skew angle: 0 degrees • No intersection left-turn lanes on approaches without stop control • No intersection right-turn lanes on approaches without stop control • No lighting

Figure 10: Rural Two-Lane, Two-Way Roads Base Conditions

CMFs are applied to account for the differences between the specific site under investigation and the base condition for the facility type. CMFs are used to adjust the SPF estimate of predicted average crash frequency for the effect of individual geometric design and traffic control features. The CMF for the SPF base condition of each geometric design and traffic control feature has a value of 1.00. CMF values less than 1.00 indicate the treatments reduce the predicted average crash frequency in comparison to the base condition. Similarly, CMF values greater than 1.00 indicate the treatments increase the predicted crash frequency. The CMFs presented in HSM Chapter 10 and the specific site types to which they apply are listed in Table 6.

TABLE 6
CMFs for Rural Two-Lane Highway Segments and Intersections

Facility Type	CMF	CMF Description	HSM CMF Equations and Tables
Roadway Segments	CMF_{1r}	Lane width	Definition (HSM p. 10-23 to 10-25)
			Table 10-8 (HSM p. 10-24)
			Equation 10.11 (HSM p. 10-24)
	CMF_{2r}	Shoulder width and type	Definition (HSM p. 10-25 to 10-27)
			Table 10-9 (HSM p. 10-26)
			HSM Equation 10-12 (HSM p. 10-27)
	CMF_{3r}	Horizontal curves: length, radius, and spiral transitions	Definition (HSM p. 10-27)
			HSM Equation 10-13 (HSM p. 10-27)
	CMF_{4r}	Horizontal curves: superelevation	Definition (HSM p. 10-28)
			HSM Equations 10-14, 10-15, and 10-16 (HSM p. 10-28)
	CMF_{5r}	Grades	Definition (HSM p. 10-28)

TABLE 6
CMFs for Rural Two-Lane Highway Segments and Intersections

Facility Type	CMF	CMF Description	HSM CMF Equations and Tables
			HSM Table 10-11 (HSM p. 10-28)
	CMF_{6r}	Driveway density	Definition (HSM p. 10-28 to 10-29)
			HSM Equation 10-17 (HSM p. 10-28)
	CMF_{7r}	Centerline rumble strips	Definition (HSM p. 10-29)
	CMF_{8r}	Passing lanes	Definition (HSM p. 10-29)
	CMF_{9r}	Two-way left-turn lanes	Definition (HSM p. 10-29 to 10-30)
			HSM Equations 10-18 and 10-19 (HSM p. 10-30)
	CMF_{10r}	Roadside design	Definition (HSM p. 10-30)
			HSM Appendix 13A (HSM p. 13-59 to 13-63)
			HSM Equation 10-20 (HSM p. 10-30)
	CMF_{11r}	Lighting	Definition (HSM p. 10-30)
			HSM Equation 10-21 (HSM p. 10-31)
			HSM Table 10-12 (HSM p. 10-31)
	CMF_{12r}	Automated speed enforcement	Definition (HSM p. 10-31)
Intersections	CMF_{1i}	Intersection skew angle	HSM Equation 10-22 (HSM p. 10-31)
			HSM Equation 10-23 (HSM p. 10-32)
	CMF_{2i}	Intersection left-turn lanes	HSM Table 10-13 (HSM p. 10-32)
	CMF_{3i}	Intersection right-turn lanes	HSM Table 10-14 (HSM p. 10-33)
	CMF_{4i}	Lighting	HSM Equation 10-24 (HSM p. 10-33)
			HSM Table 10-15 (HSM p. 10-33)

The SPFs were developed in HSM-related research from the most complete and consistent available data sets. However, the predicted crash frequencies may vary substantially from one jurisdiction to another for a variety of reasons. Calibration factors provide a method for incorporating local data to improve the estimated crash frequencies for individual locations. The local calibration factor accounts for the differences between the jurisdiction under investigation and the jurisdictions that were used to develop the default HSM SPFs. The local calibration factor is calculated using local crash data and other roadway characteristic data. The process for determining calibration factors for the predictive models is described in HSM Part C, Appendix A.1 (HSM p. A-1).

The predicted crash frequency under real conditions can be calculated using Equation 1:

$$N_{predicted} = N_{spf\ x} \times C_x \times (CMF_{1x} \times CMF_{2x} \times CMF_{3x} \times \ \ldots \times CMF_{yx}) \qquad \text{(Eq. 1)}$$

where:

$N_{predicted}$ = predicted average crash frequency for a specific year for site type x

$N_{spf\ x}$ = predicted average crash frequency determined for base conditions of the SPF developed for site type x

CMF_{yx} = CMFs specific to site type x and specified geometric design and traffic control features y

C_x = calibration factor to adjust SPF for local conditions for site type x

Step 3: Expected Crash Frequency with Empirical Bayes Method

This step can be omitted if no recorded crash data for the specific site under investigation were available or the data are considered unreliable. When historical crash data are available, the EB method (either site-specific or project-level) can be used to combine the HSM Chapter 10 predicted average crash frequency with the observed crash frequency. The expected average crash frequency is a more statistically reliable estimate. The expected average crash frequency can be determined using Equation 2:

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed} \qquad \text{(Eq. 2)}$$

where:

w = the weighted adjustment to be placed on the predictive model estimate.

This value can be calculated using Equation 3:

$$w = \frac{1}{1 + k \times \sum_{all\ study\ years} N_{predicted}} \qquad \text{(Eq. 3)}$$

where:

k = the overdispersion parameter of the associated SPF used to estimate $N_{predicted}$.

Table 7 lists the k values for SPFs of different facility types.

TABLE 7
Overdispersion Parameters for SPFs in HSM Chapter 10

Facility Type	Overdispersion Parameter (k)
Rural two-lane, two-way roadway segments	0.236 per length of the roadway segment
Three-leg stop-controlled intersection	0.54
Four-leg stop-controlled intersection	0.24
Four-leg signalized intersection	0.11

Step 4: Crash Frequency under Different Collision Types and Crash Severity Levels

HSM Chapter 10 provides the crash severity and collision type distribution table for all the facility types included, as listed in Table 8. The crash frequency under different severity levels and collision types could be determined based on the distribution table after the predicted or expected crash frequencies were determined. These proportions can be updated based on local data for a particular jurisdiction as part of the calibration process.

TABLE 8
Crash Severity and Collision Type Distribution Table for Different Facility Types

Facility Type	Crash Severity Distribution	Collision Type Distribution
Rural two-lane, two-way roadway segments	HSM Table 10-3	HSM Table 10-4
Three-leg stop-controlled intersection	HSM Table 10-5	HSM Table 10-6
Four-leg stop-controlled intersection	HSM Table 10-5	HSM Table 10-6
Four-leg signalized intersection	HSM Table 10-5	HSM Table 10-6

Figure 11 shows the HSM Chapter 10 predictive method flowchart for calculating the predicted and expected average crash frequency for rural two-lane, two-way roads.

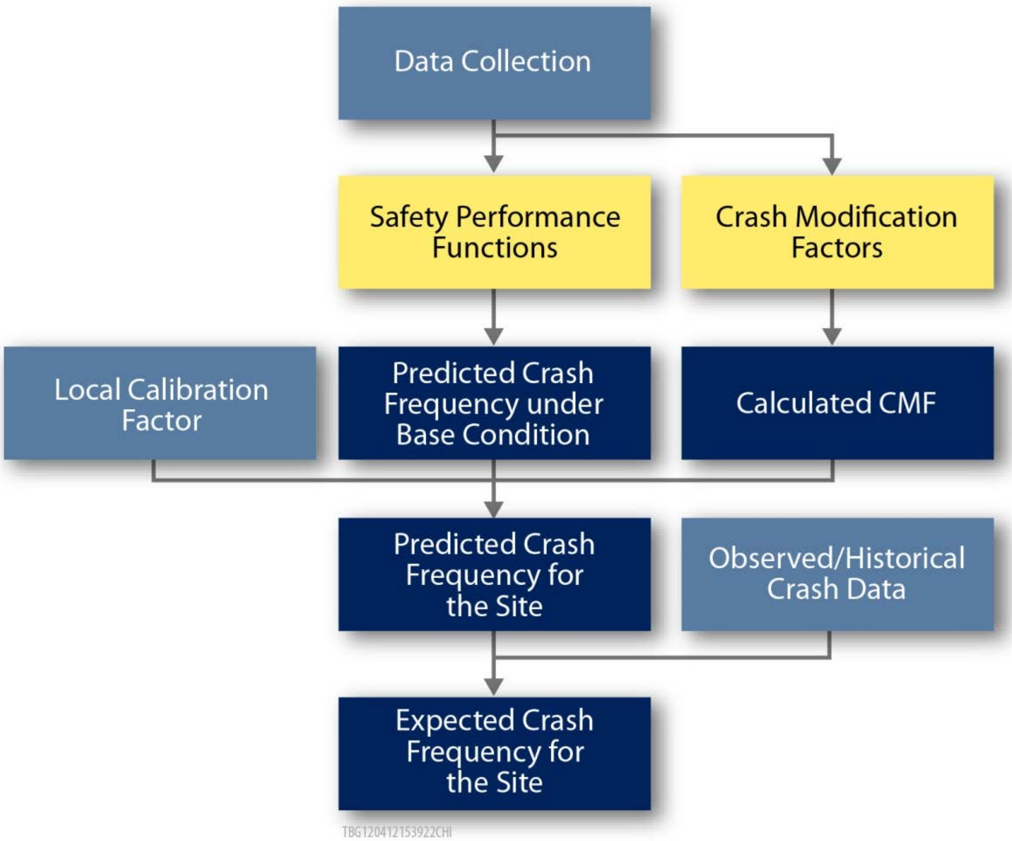


Figure 11: Flowchart for Calculating Expected Crash Frequency on Rural Two-Lane, Two-Way Roads

2.3.14 Data Requirements for Rural Two-Lane, Two-Way Roads

For the study period, it is important to determine the availability of AADT volumes and, for an existing roadway, the availability of observed/reported crash data to determine whether the EB method is applicable.

A good understanding of the SPFs' base conditions will help determine relevant data needs and avoid unnecessary data collection. The base conditions for rural two-lane, two-way roads are defined in Section 2.3.12, and in HSM Section 10.6.1 (HSM p. 10-14) for roadway segments and HSM Section 10.6.2 (HSM p. 10-17) for intersections.

General data for intersections and segments can be collected from different sources. Examples of data sources include commercial aerial maps, design plans, and states' roadway inventory system. Data needed for this example are summarized in the following sections.

Intersection Data

Generally, the effect of major and minor road traffic volumes (AADT) on crash frequency is incorporated through an SPF, while the effects of geometric design and traffic controls are incorporated through the CMFs. Data required to apply the predictive method for intersections are listed in Table 9.

TABLE 9
Intersection Data Requirements for Rural Two-Lane, Two-Way Roads

Intersections	Units/Description
Intersection type	Unsignalized three-leg (3ST), unsignalized four-leg (4ST), and signalized four-leg (4SG)
Traffic flow major road	$AADT_{major}$ (vpd)
Traffic flow minor road	$AADT_{minor}$ (vpd)
Intersection skew angle	degrees
Number of signalized or uncontrolled approaches with a left-turn lane	From 0 to 4
Number of signalized or uncontrolled approaches with a right-turn lane	From 0 to 4
Intersection lighting	Present or not present
Calibration factor (C_i)	Derived from calibration process
Observed crash data	Applicable only with the EB method; crashes that occur at the intersection or on an intersection leg, and are related to the presence of an intersection during the period of study

Note:
C_i = intersection calibration factor
vpd = vehicles per day

Roadway Segment Data

The effect of traffic volume in crash frequency is incorporated through an SPF, while the effects of geometric design and traffic control features are incorporated through the CMFs. There is no minimum roadway segment length when applying the predictive method. However, when dividing the facility into small homogeneous sections, it is recommended to keep the minimum roadway segment length as 0.10 mile to minimize the calculation efforts and avoid modifying the results. Table 10 includes data requirements for roadway segment locations.

TABLE 10
Roadway Segment Data Requirements for Rural Two-Lane, Two-Way Roads

Roadway Segments	Units/Description
Segment length	miles
Traffic volume	AADT (vpd)
Lane width	feet
Shoulder width	feet
Shoulder type	Paved, gravel, composite, or turf
Length of horizontal curve	miles
Radius of curvature	feet
Spiral transition curve	Present or not present
Superelevation variance	feet/feet
Grade	percent (%)
Driveway density	Driveways per mile
Centerline rumble strips	Present or not present

TABLE 10
Roadway Segment Data Requirements for Rural Two-Lane, Two-Way Roads

Roadway Segments	Units/Description
Passing lanes	Present (1 lane), present (2 lanes), or not present
Two-way left-turn lane	Present/not present
Roadside hazard rating	Scale: 1 to 7 (1 = the safest, 7 = the most dangerous)
Segment lighting	Present or not present
Auto speed enforcement	Present or not present
Calibration factor (Cr)	Derived from calibration process
Observed crash data	Applicable only with the EB method; crashes that occur between intersections and are not related to the presence of an intersection during the period of study

Note:
vpd = vehicles per day

More information about the roadside hazard rating can be found in HSM Part D, Appendix 13A (p. 13-59).

2.3.15 HSM Chapter 11: Predictive Method for Rural Multilane Highways

HSM Chapter 11 provides a method to estimate the predicted and/or expected average crash frequency, crash severity, and collision types for rural multilane highway facilities. 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. Estimates of crash frequency can be made for a period of time that occurred in the past or will occur in the future.

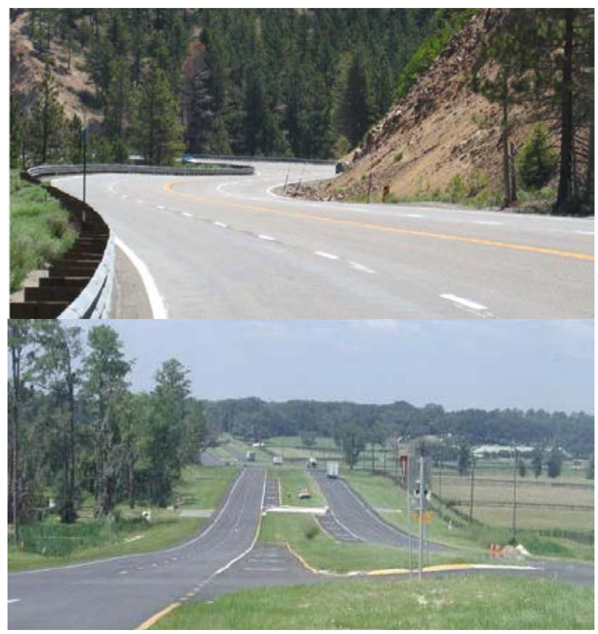


Figure 12: Rural Multilane Highways

This chapter is applicable to all rural multilane highways without full access control that are outside urban areas that have a population less than 5,000 persons. This comprises all rural non-freeways with four through travel lanes, with the exception of two-lane highways with side-by-side passing lanes. Moreover, this chapter addresses three- and four-leg intersections with minor-road stop and four-leg signalized intersections on all the roadway cross-sections. Figure 12 shows typical examples of undivided and divided rural multilane highways.

Table 11 includes the different site types for which SPFs have been developed for estimating expected average crash frequency, severity, and collision type. Figure 13 lists the facility types and definitions provided in HSM Chapter 11.

TABLE 11
Roadway Segment and Intersection Types and Descriptions for Rural Two-Lane, Two-Way Roads

Facility Type	Site Types with SPFs in HSM Chapter 11
Roadway Segments	Rural four-lane undivided segments (4U)
	Rural four-lane divided segments (4D)
Intersections	Unsignalized three-leg (stop control on minor-road approaches) (3ST)
	Unsignalized four-leg (stop control on minor-road approaches) (4ST)
	Signalized four-leg (4SG) [a]

Note:
[a] The four-leg signalized intersection models do not have base conditions; therefore, these models can be used only for generalized predictions of crash frequency.

Multilane Rural Roads Facility Type Definitions

Facility Type	Definition
Undivided four-lane roadway segment (4U)	A roadway segment consisting of four lanes with a continuous cross-section that provides two directions of travel in which lanes are not physically separated by either distance or a barrier. Multilane roadways where opposing lanes are separated by a flush/ nontraversable median or similar means are considered undivided facilities. However, HSM Chapter 11 predictive methods do not address multilane highways with flush separators.
Divided four-lane roadway segment (4D)	Divided highways are nonfreeway facilities (such as facilities without full control access) that have lanes in two directions of travel separated by a raised, depressed, or flush median that is not designed to be traversed by a vehicle; this may include raised or depressed medians with or without physical median barrier, or flush medians with physical median barriers.
Three-leg intersections with stop control (3ST)	An intersection of a rural multilane highway (such as four-lane divided or undivided roadway) and a minor road. A STOP sign is provided on the minor-road approach to the intersection only.
Four-leg intersection with stop control (4ST)	An intersection of a rural multilane highway (such as four-lane divided or undivided roadway) and two minor roads. A STOP sign is provided on both minor-road approaches to the intersection.
Four-leg signalized intersection (4SG)	An intersection of a rural multilane highway (such as four-lane divided or undivided roadway) and two other rural roads, which may be two-lane or four-lane rural highways. Signalized control is provided at the intersection by traffic lights.

Figure 13: Multilane Rural Roads Facility Types and Definitions

In order to apply the predictive method, the roadway within the defined study area limits must be divided into homogenous individual sites, segments and intersections. Roadway segment boundaries begin at the center of an intersection and end at either the center of the next intersection, or where there is a change in the segment's cross-section (homogeneous segment). The length of the roadway segment is measured from the center of the intersection.

An intersection is defined as the junction of two or more roadway segments. The intersection predictive models estimate the predicted average crash frequency of crashes within the intersection limits (Region A in Figure 14) and intersection-related crashes that occur on the intersection legs (Region B in Figure 14).

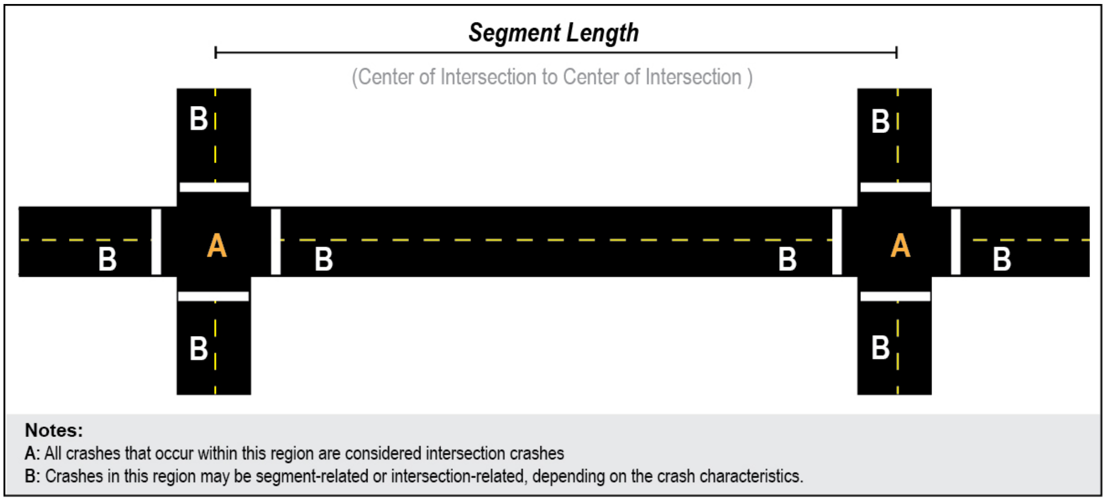


Figure 14: Rural Multilane Highways – Definition of Roadway Segments and Intersections

2.3.16 Calculating the Crash Frequency for Rural Multilane Highways

HSM Chapter 11 provides the methodology for calculating the predicted and/or expected crash frequency for roadway segments and intersections on rural multilane highways. The calculation is for a given period of time during which the geometric design and traffic control features are unchanged and traffic volumes are known. The whole process could be divided into the following steps:

1. Predicted crash frequency under base conditions
2. Predicted crash frequency under site conditions
3. Expected crash frequency with Empirical Bayes method
4. Crash frequency under different collision types and crash severity levels

Step 1: Predicted Crash Frequency under Base Conditions

The predicted average crash frequency for the roadway segments and intersections under the base condition may be determined by replacing the AADT and segment length (for roadway segments) or the AADTs for major and minor roads (for intersections) in SPFs with site-specific values. Table 12 lists the SPFs for different facility types included in HSM Chapter 11 and the applicable AADT ranges for the SPFs. Only application to sites within the AADT ranges is likely to provide reliable results.

NOTE: SPFs for 4SG on rural multilane highways have no specific base conditions and, therefore, can only be applied for generalized predictions. No CMFs are provided for 4SG intersections, and predictions of average crash frequencies cannot be made for intersections with specific geometric design and traffic control features.

TABLE 12
Rural Multilane Highways SPFs in HSM Chapter 11

Facility Type	Equation in HSM	AADT Range
Rural four-lane undivided segments (4U)	HSM Equation 11-7	Up to 33,200 vpd
Rural four-lane divided segments (4D)	HSM Equation 11-9	Up to 89,300 vpd
Unsignalized three-leg (stop control on minor-road approaches) (3ST)	HSM Equation 11-11	$AADT_{major}$ 0 to 78,300 vpd
		$AADT_{minor}$ 0 to 23,000 vpd
Unsignalized three-leg (stop control on minor-road approaches) (4ST)	HSM Equation 11-11	$AADT_{major}$ 0 to 78,300 vpd
		$AADT_{minor}$ 0 to 7,400 vpd
Signalized four-leg (4SG)	HSM Equations 11-11 and 11-12	$AADT_{major}$ 0 to 43,500 vpd
		$AADT_{minor}$ 0 to 18,500 vpd

Notes:
$AADT_{major}$ = average annual daily traffic on the major route
$AADT_{minor}$ = average annual daily traffic on the minor route
vpd = vehicles per day

Highway agencies may wish to develop their own jurisdiction-specific SPFs derived from local conditions and crash experience. These SPFs may be substituted for models presented in HSM Chapter 11. The HSM provides criteria for development of SPFs and is presented in HSM Part C, Appendix A.1.2 (HSM p. A-9).

Step 2: Predicted Crash Frequency under Site Conditions

The crash frequency calculated using the SPFs shown in the previous section is the predicted crash frequency for the roadway segments or intersections under base conditions. Base conditions are the prevalent conditions under which the SPFs were developed and are not necessarily the same for all facilities. The base conditions for roadway segments and intersections on rural multilane highways are listed in Figure 15.

Rural Multilane Highways Base Conditions

Undivided Roadways

- Lane width: 12 feet
- Shoulder width: 6 feet
- Shoulder type: Paved
- Sideslopes: 1:7 (vertical: horizontal) or flatter
- No lighting
- No automated speed enforcement

Divided Roadways

- Lane width: 12 feet
- Right shoulder width: 8 feet
- Median width: 30 feet
- No lighting
- No automated speed enforcement

Intersections

- Intersection skew angle: 0 degrees
- No intersection left-turn lanes except on stop-controlled approaches
- No intersection right-turn lanes except on stop-controlled approaches
- No lighting

Figure 15: Rural Multilane Highway Base Conditions

CMFs are applied to account for the differences between the specific site under investigation and the base condition for the facility type. CMFs are used to adjust the SPF estimate of predicted average crash frequency for the effect of individual geometric design and traffic control features. The CMF for the SPF base condition of each geometric design and traffic control feature has a value of 1.00. CMF values less than 1.00 indicate the treatments reduce the predicted average crash frequency in comparison to the base condition. Similarly, CMF values greater than 1.00 indicate the treatments increase the predicted crash frequency. The CMFs presented in HSM Chapter 11 and the specific site types to which they apply are listed in Table 13.

TABLE 13
CMFs for Rural Multilane Highway Segments and Intersections

Facility Type	CMF	CMF Description	HSM CMF Equations and Tables
Undivided Roadway Segments	CMF_{1u}	Lane width on undivided segments	Definition (HSM p. 11-26 to 11-27)
			HSM Table 11-11 (HSM p. 11-26)
			HSM Equation 11-13 (HSM p. 11-26)
	CMF_{2u}	Shoulder width and type	Definition (HSM p. 11-27 to 11-28)
			HSM Tables 11-12 and 11-13 (HSM p. 11-27)
			HSM Equation 11-14 (HSM p. 11-27)
	CMF_{3ru}	Side slopes	Definition (HSM p. 11-28)
			HSM Table 11-14 (HSM p. 11-28)
	CMF_{4ru}	Lighting	Definition (HSM p. 11-28 to 11-29)
			HSM Equation 11-15 (HSM p. 11-28)
			HSM Table 11-15 (HSM p. 11-29)
	CMF_{5ru}	Automated speed enforcement	Definition (HSM p. 11-29)
			See text (HSM p. 11-29)
Divided Roadway Segments	CMF_{1d}	Lane width on undivided segments	Definition (HSM p. 11-29 to 11-30)
			HSM Table 11-16 (HSM p. 11-30)
			HSM Equation 11-16 (HSM p. 11-29)
	CMF_{2d}	Right shoulder width on divided roadway segment	Definition (HSM p. 11-30 to 11-31)
			HSM Table 11-17 (HSM p. 11-31)
	CMF_{3rd}	Median width	Definition (HSM p. 11-31)
			HSM Table 11-18 (HSM p. 11-31)
	CMF_{4rd}	Lighting	Definition (HSM p. 11-31 to 11-32)
			HSM Equation 11-17 (HSM p. 11-31)
			HSM Table 11-19 (HSM p. 11-32)
	CMF_{5rd}	Automated speed enforcement	Definition (HSM p. 11-32)
			See text (HSM p. 11-32)

TABLE 13
CMFs for Rural Multilane Highway Segments and Intersections

Facility Type	CMF	CMF Description	HSM CMF Equations and Tables
Three- and Four-Leg Stop-Controlled Intersections	CMF_{1i}	Intersection angle (3ST and 4ST)	Definition (HSM p. 11-33 to 11-34)
			3ST: HSM Equations 11-18 and 11-19 (HSM p. 11-33)
			4ST: HSM Equations 11-18 and 11-19 (HSM p. 11-33)
	CMF_{2i}	Left-turn lane on major road	Definition (HSM p. 11-34)
			HSM Table 11-22 (HSM p. 11-34)
	CMF_{3i}	Intersection right-turn lanes	Definition (HSM p. 11-34 to 11-35)
			HSM Table 11-23 (HSM p. 11-35)
	CMF_{4i}	Lighting	Definition (HSM p. 11-35)
			HSM Equation 11-22 (HSM p. 11-35)
			HSM Table 11-24 (HSM p. 11-35)

The SPFs were developed in HSM-related research from the most complete and consistent available data sets. However, the predicted crash frequencies may vary substantially from one jurisdiction to another for a variety of reasons. Calibration factors provide a method for incorporating local data to improve the estimated crash frequencies for individual locations. The local calibration factor accounts for the differences between the jurisdiction under investigation and the jurisdictions that were used to develop the default HSM SPFs. The local calibration factor is calculated using local crash data and other roadway characteristic data. The process for determining calibration factors for the predictive models is described in HSM Part C, Appendix A.1 (HSM p. A-1).

The predictive crash frequency under real conditions can be calculated using equation 4:

$$N_{predicted} = N_{spf\ x} \times C_x \times (CMF_{1x} \times CMF_{2x} \times CMF_{3x} \times \ \dots \times CMF_{yx}) \qquad \text{(Eq. 4)}$$

where:

$N_{predicted}$ = predicted average crash frequency for a specific year for site type x

$N_{spf\ x}$ = predicted average crash frequency determined for base conditions of the SPF developed for site type x

CMF_{yx} = crash modification factors specific to site type x and specified geometric design and traffic control features y

C_x = calibration factor to adjust SPF for local conditions for site type x

Step 3: Expected Crash Frequency with Empirical Bayes Method

This step can be omitted if recorded crash data for the specific site under investigation were unavailable or data are considered unreliable. When historical crash data is available, the EB method (either site-specific or project-level) is used to combine the HSM Chapter 11 predicted average crash frequency with the observed crash frequency. The expected average crash frequency is a more statistically reliable estimate. The expected average crash frequency can be determined using Equation 5:

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed} \qquad \text{(Eq. 5)}$$

where:

w = the weighted adjustment to be placed on the predictive model estimate.

This value can be calculated using Equation 6:

$$w = \frac{1}{1 + k \times \sum_{all\ study\ years} N_{predicted}} \quad \text{(Eq. 6)}$$

where:

k = the overdispersion parameter of the associated SPF used to estimate $N_{predicted}$.

Table 14 lists the k values for SPFs of different facility types.

TABLE 14
Chapter 11 SPFs Overdispersion Parameters

Facility Type	Overdispersion Parameter (k)
Rural four-lane undivided segments (4U)	$1/e^{(c + \ln(L))}$
Rural four-lane divided segments (4D)	$1/e^{(c + \ln(L))}$
Unsignalized three-leg (stop control on minor-road approaches) (3ST)	Coefficients listed in HSM Table 11-7
Unsignalized three-leg (stop control on minor-road approaches) (4ST)	Coefficients listed in HSM Table 11-7
Signalized four-leg (4SG)[1]	Coefficients listed in HSM Table 11-8

Note:
[1] The four-leg signalized intersection models do not have base conditions and, therefore, can be used only for generalized predictions of crash frequency

Step 4: Crash Frequency under Different Collision Types and Crash Severity Levels

HSM Chapter 11 safety performance functions provide regression coefficients to estimate not only total crashes, but also fatal-and-injury crashes for segments and intersections. These coefficients can be found in HSM Table 11-3 (HSM p. 11-15), and Table 11-5 (HSM p. 11-19) for undivided and divided roadway segments, and HSM Tables 11-7 and 11-8 (HSM p. 11-22) for intersections. PDO crashes are computed as the difference between total and fatal-and-injury crashes.

In addition, collision type distribution tables are included for all the facility types, as listed in Table 15. The crash frequency for different collision types can be determined based on the distribution table after the predicted or expected crash frequencies are calculated. These proportions can be updated based on local data for a particular jurisdiction as part of the calibration process.

TABLE 15
Rural Multilane Highway Collision Type Distributions

Facility Type	Collision Type Distribution
Rural four-lane undivided segments (4U)	HSM Table 11-4
Rural four-lane divided segments (4D)	HSM Table 11-6
Unsignalized three-leg (stop control on minor-road approaches) (3ST)	HSM Table 11-9
Unsignalized three-leg (stop control on minor-road approaches) (4ST)	HSM Table 11-9
Signalized four-leg (4SG)	HSM Table 11-9

Figure 16 shows the HSM Chapter 11 predictive method flowchart for calculating the predicted and expected crash frequency for rural multilane highways.

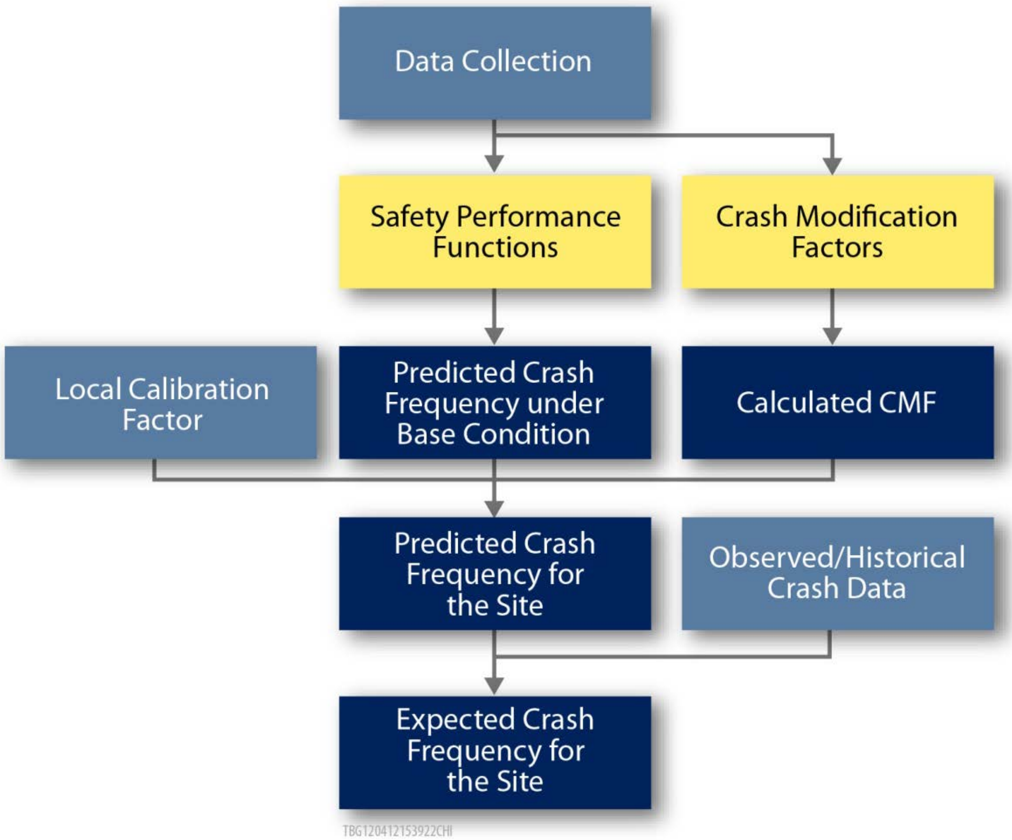


Figure 16: Flowchart for Calculating Predicted and Expected Crash Frequency on Rural Multilane Highways

2.3.17 Data Requirements for Rural Multilane Highways

For the study period, it is important to determine the availability of AADT volumes, and for an existing roadway, the availability of observed crash data to determine whether the EB method is applicable.

A good understanding of the SPFs' base conditions will help determine relevant data needs and avoid unnecessary data collection. The base conditions for rural multilane highways are defined in Section 2.3.16, as well as in HSM Sections 11.6.1 (HSM p. 11-14) and 11.6.2 (HSM p. 11-17) for roadway segments and HSM Section 11.6.3 (HSM p. 11-20) for intersections.

General data for intersections and segments can be collected from different sources. Examples of data sources include commercial aerial maps, design plans, and states' roadway inventory systems. Data needed for this example are summarized in the following sections.

Intersection Data

Generally, the effect of major and minor road traffic volumes (AADT) on crash frequency is incorporated through SPFs, while the effects of geometric design and traffic controls are incorporated through the CMFs. Data required to apply the predictive method for intersections are listed in Table 16.

TABLE 16
Intersection Data Requirements for Rural Multilane Highways

Intersections	Units/Description
Intersection type	Unsignalized three-leg (3ST), unsignalized four-leg (4ST), and signalized four-leg (4SG)
Traffic flow major road	$AADT_{major}$ (vpd)
Traffic flow minor road	$AADT_{minor}$ (vpd)
Intersection skew angle	degrees
Number of signalized or uncontrolled approaches with a left-turn lane	From 0 to 4
Number of signalized or uncontrolled approaches with a right-turn lane	From 0 to 4
Intersection lighting	Present or not present
Calibration factor (C_i)	Derived from the calibration process
Observed crash data	Applicable only with the EB method; crashes that occur at the intersection or on an intersection leg, and are related to the presence of an intersection during the period of study

Notes:
C_i = intersection calibration factor
vpd = vehicles per day

Roadway Segment Data

The effect of traffic volume on crash frequency is incorporated through an SPF, while the effects of geometric design and traffic control features are incorporated through the CMFs. There is no minimum roadway segment length when applying the predictive method. However, when dividing the facility into small homogeneous sections, it is recommended to keep the minimum roadway segment length as 0.10 mile to minimize the calculation efforts and avoid affecting the results. Table 17 includes data requirements for roadway segment locations.

TABLE 17
Roadway Segment Data Requirements for Rural Multilane Highways

Roadway Segments	Units/Description
Segment length	miles
Traffic volume	AADT (vpd)
Lane width	feet
Shoulder width	feet
Shoulder type – right shoulder for divided	Paved, gravel, composite, or turf
Median width (**Divided Only**)	feet
Side slopes (**Undivided Only**)	miles
Segment lighting	Present or not present
Auto speed enforcement	present or not present

TABLE 17
Roadway Segment Data Requirements for Rural Multilane Highways

Roadway Segments	Units/Description
Calibration factor (Cr)	Derived from the calibration process
Observed crash data	Applicable only with the EB method; crashes that occur between intersections and are not related to the presence of an intersection during the period of study

Notes:
AADT = average annual daily traffic
C_r = roadway segment calibration factor
vpd = vehicles per day

2.3.18 HSM Chapter 12: Predictive Method for Urban and Suburban Arterials

HSM Chapter 12 provides a structured methodology for estimating the predicted and/or expected average crash frequency, crash severity, and collision types for urban and suburban arterial facilities. Crashes involving all vehicle types, bicycles, and pedestrians are included, with the exception of crashes between bicycles and pedestrians. The method is applicable to existing sites, design alternatives to existing sites, new sites, and alternative traffic volume projections.

This chapter is applicable to all arterials that are inside urban boundaries where the population is greater than 5,000 people (HSM Section 12.3, p. 12-2). The term suburban refers to outlying portions of an urban area.

This chapter includes arterials without full access control, other than freeways, with two- or four-lane undivided facilities, four-lane divided and three- and five-lane roads with center TWLTLs in urban and suburban areas. HSM Chapter 12 includes three- and four-leg intersections with minor-road stop control or traffic signal control on all of the roadway cross sections to which the chapter applies.

Table 18 contains the site types on urban and suburban arterials for which SPFs have been developed for predicting average crash frequency, severity, and collision type.

TABLE 18
Roadway Segment and Intersection Types and Descriptions for Urban and Suburban Arterials

Facility Type	Site Types with SPFs in HSM Chapter 12
Roadway Segments	Two-lane undivided arterials (2U)
	Three-lane arterials with a center TWLTL (3T)
	Four-lane undivided arterials (4U)
	Four-lane divided arterials (4D)
	Five-lane arterials including a center TWLTL (5T)
Intersections	Unsignalized three-leg (stop control on minor-road approaches) (3ST)
	Signalized three-leg intersections (3SG)
	Unsignalized four-leg (stop control on minor-road approaches) (4ST)
	Signalized four-leg (4SG)

Figure 17 lists the facility types and definitions provided in HSM Chapter 12.

Urban and Suburban Arterials Facility Type Definitions

Facility Type	Definition
Two-lane undivided arterials	Two-lane roadway with a continuous cross-section providing two directions of travel in which the lanes are not physically separately by either distance or a barrier.
Three-lane arterials	Three-lane roadway with a continuous cross-section providing two directions of travel, with a TWLTL in the center.
Four-lane undivided arterials	Four-lane roadway 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	Two-lane roadway with a continuous cross-section providing two directions of travel in which the lanes are physically separated by either distance or a barrier. Roadways with raised or depressed median are also included in this category.
Five-lane arterials including a center TWLTL	Five-lane roadway with a continuous cross-section providing two directions of travel in which the center lane is a TWLTL.
Unsignalized three-leg intersection with stop control	Intersection of an urban/suburban arterial with a minor road. Stop sign is present on the minor road approach.
Signalized three-leg intersections	Intersection of an urban/suburban arterial with a minor road. Traffic light is provided at the intersection.
Unsignalized four-leg (stop control on minor-road approaches	Intersection of an urban/suburban arterial with two minor roads. Stop sign is present on both the minor road approaches.
Signalized four-leg intersection	Intersection of an urban/suburban arterial with two minor roads. Traffic light is provided at the intersection.

Figure 17: Urban and Suburban Arterials Facility Types and Definitions

Commonly, a roadway consists of a contiguous group of sites (intersections and roadway segments). On each roadway, multiple site types may exist, including divided and undivided segments and signalized and unsignalized intersections. To apply the predictive method, the roadway is divided into individual homogeneous segments and intersections. HSM Chapter 12 provides guidance on how to define roadway segments and intersections (HSM Section 12.5, p. 12-9).

A roadway segment is defined as a section of continuous traveled way that provides two-way operation of traffic uninterrupted by an intersection and consists of homogeneous geometric and traffic control features. A segment begins and ends at the center of bounding intersections, or where there is a change in homogeneous roadway characteristics. When a roadway segment begins or ends at an intersection, the length of the roadway segment is measured from the center of the intersection.

An intersection is defined as the junction of two or more roadway segments. The intersection predictive models estimate the predicted and/or expected average crash frequencies within the intersection limits (Region A in Figure 18) and intersection-related crashes that occur on the intersection legs (Region B in Figure 18).

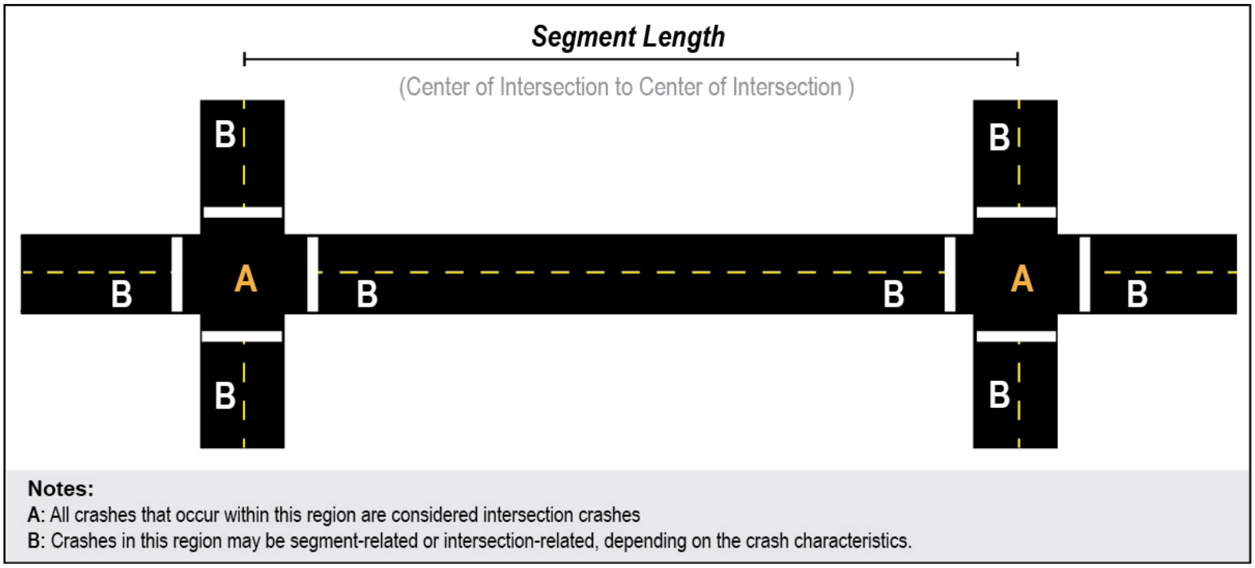


Figure 18. Urban and Suburban Arterials – Definition of Roadway Segments and Intersections

2.3.19 Calculating the Crash Frequency for Urban and Suburban Arterials

HSM Chapter 12 provides the methodology for calculating the predicted and/or expected crash frequency for roadway segments and intersections on urban and suburban arterials. The calculation is for a given period of time during which the geometric design and traffic control features are unchanged and traffic volumes are known. The entire process could be divided into the following steps:

1. Predicted crash frequency under base conditions
2. Predicted crash frequency under site conditions
3. Expected crash frequency with Empirical Bayes method
4. Crash frequency under different collision types and crash severity levels

Step 1: Predicted Crash Frequency under Base Conditions

The predicted crash frequency for the roadway segments and intersections under the base condition can be determined by replacing the AADT and segment length (for roadway segments) or the AADTs for major and minor roads (for intersections) in SPFs with site-specific values. Table 19 lists the different facility types included in HSM Chapter 12 and the applicable AADT ranges for the SPFs. Only application to sites within the AADT ranges would provide reliable results.

TABLE 19
Urban and Suburban Arterials Facility Types and AADT Ranges

Item	Facility Type	AADT Range
Roadway Segments	Two-lane undivided arterials (2U)	Up to 32,600 vpd
	Three-lane arterials with TWLTL (3T)	Up to 32,900 vpd
	Four-lane undivided arterials (4U)	Up to 40,100 vpd
	Four-lane divided arterials (4D)	Up to 66,000 vpd
	Five-lane arterials with TWLTL (5T)	Up to 53,800 vpd
Intersections	Three-leg intersection with stop control on minor approach (3ST)	$AADT_{major}$ 0 to 45,700 vpd
		$AADT_{minor}$ 0 to 9,300 vpd
	Three-leg signalized intersection (3SG)	$AADT_{major}$ 0 to 46,800 vpd
		$AADT_{minor}$ 0 to 5,900 vpd
	Four-leg intersection with stop control on minor approach (4ST)	$AADT_{major}$ 0 to 58,100 vpd
		$AADT_{minor}$ 0 to 16,400 vpd
	Four-leg signalized intersection (4SG)	$AADT_{major}$ 0 to 67,700 vpd
		$AADT_{minor}$ 0 to 33,400 vpd
	4SG intersections pedestrian models	$AADT_{major}$ 0 to 82,000 vpd
		$AADT_{minor}$ 0 to 49,100 vpd
		$Pedestrian_{vol}$ 0 to 34,200 ped/day

Note:
$Pedestrian_{vol}$ = pedestrians per day crossing all four legs combined

SPFs are provided for different collision types: multiple-vehicle nondriveway, single vehicle, multiple-vehicle driveway-related, and vehicle-pedestrian collisions. Adjustment factors are provided for vehicle-bicycle and stop-controlled intersection vehicle-pedestrian collisions. Table 20 summarizes the different SPFs by collisions type for roadway segments and intersections.

TABLE 20
Urban and Suburban Arterials SPFs in HSM Chapter 12

Facility Type	SPF Components by Collision Type	HSM Equation
Roadways Segments	Multiple-vehicle nondriveway collisions	HSM Equations 12-10, 12-11, and 12-12 (HSM p. 12-18 and 12-20)
	Single-vehicle crashes	HSM Equations 12-13, 12-14, and 12-15 (HSM p. 12-20 to 12-21)
	Multiple-vehicle driveway-related collisions	HSM Equations 12-16, 12-17, and 12-18 (HSM p. 12-22 and 12-27)
	Vehicle-pedestrian collisions	HSM Equation 12-19 (HSM p. 12-27)
	Vehicle-bicycle collisions	HSM Equation 12-20 (HSM p. 12-27)

TABLE 20
Urban and Suburban Arterials SPFs in HSM Chapter 12

Facility Type	SPF Components by Collision Type	HSM Equation
Intersections	Multiple-vehicle collisions	HSM Equations 12-21, 12-22, and 12-23 (HSM p. 12-29)
	Single-vehicle crashes	HSM Equations 12-24, 12-25, 12-26, and 12-27 (HSM p. 12-32 to 12-33 and p. 12-36)
	Vehicle-pedestrian collisions	HSM Equations 12-28, 12-29, and 12-30 (HSM p. 12-36 and 12-38)
	Vehicle-bicycle collisions	HSM Equation 12-31 (HSM p. 12-38)

Step 2: Predicted Crash Frequency under Real Conditions

Each SPF listed in Table 20 is used to estimate the predicted crash frequency of a roadway segment or intersection under base conditions, which is later adjusted to site-specific conditions. Base conditions are a specific set of geometric design and traffic control features under which the SPFs were developed, and are not necessarily the same for all facilities. The base conditions for roadway segments and intersections on urban and suburban arterials are listed in Figure 19.

Urban and Suburban Arterials Base Conditions

Roadway Segments	Intersections

• Absence of on-street parking
• Absence of fixed objects
• For divided facilities: median width of 15 feet
• Absence of lighting
• Absence of Automated Speed Enforcement
	• Absence of left-turn lanes
	• Permissive left-turn signal phasing
	• Absence of right-turn lanes
	• Permitting right turn on red (RTOR)
	• Absence of intersection lighting
	• Absence of RLR cameras
	• Signalized: vehicle-pedestrian collisions
	  - Absence of bus stops within 1,000 feet
	  - Absence of schools within 1,000 feet of intersection
	  - Absence of alcohol sales establishments within 1,000 feet of intersection

Figure 19: Urban and Suburban Arterials Base Conditions

CMFs are applied to account for the differences between the specific site under investigation and the base conditions. CMFs are used to adjust the SPF estimate of predicted average crash frequency for the effect of individual geometric design and traffic control features. The CMF for the SPF base condition of each geometric design and traffic control feature has a value of 1.00. CMF values less than 1.00 indicate the treatments reduce the predicted average crash frequency in comparison to the base condition.

Similarly, CMF values greater than 1.00 indicate the treatments increase the predicted crash frequency. The CMFs presented in HSM Chapter 12 and the specific site types to which they apply are listed in Table 21.

TABLE 21
CMFs for Urban and Suburban Arterials Roadway Segments and Intersections

Facility Type	CMF	CMF Description	CMF Equations and Tables
Roadway Segments	CMF_{1r}	On-street parking	Definition (HSM p. 12-40)
			HSM Table 12-19 (HSM p. 12-40)
			HSM Equation 12-32 (HSM p. 12-40)
	CMF_{2r}	Roadside fixed objects	Definition (HSM p. 12-41)
			HSM Tables 12-20 and 12-21 (HSM p. 12-41)
			HSM Equation 12-33 (HSM p. 12-40)
	CMF_{3r}	Median width	Definition (HSM p. 12-41)
			HSM Table 12-22 (HSM p. 12-42)
	CMF_{4r}	Lighting	Definition (HSM p. 12-42)
			HSM Equation 12-34 (HSM p. 12-42)
			HSM Table 12-23 (HSM p. 12-42)
	CMF_{5r}	Automated speed enforcement	Definition (HSM p. 12-43)
			See text (HSM p. 12-43)
Multiple-Vehicle Collisions and Single-Vehicle Crashes at Intersections	CMF_{1i}	Intersection left-turn lanes	Definition (HSM p. 12-43)
			HSM Table 12-24 (HSM p. 12-43)
	CMF_{2i}	Intersection left-turn signal phasing	Definition (HSM p. 12-43 to 12-44)
			HSM Table 12-25 (HSM p. 12-44)
	CMF_{3i}	Intersection right-turn lanes	Definition (HSM p. 12-44)
			HSM Table 12-26 (HSM p. 12-44)
	CMF_{4i}	Right-turn-on-red	Definition (HSM p. 12-44)
			HSM Equation 12-35 (HSM p. 12-44)
	CMF_{5i}	Lighting	Definition (HSM p. 12-45)
			HSM Table 12-27 (HSM p. 12-45)
			HSM Equation 12-36 (HSM p. 12-45)
	CMF_{5i}	Red-light cameras	Definition (HSM p. 12-45 to 12-46)
			HSM Equations 12-37, 12-38, and 12-39 (HSM p. 12-45)
Vehicle-Pedestrian Collisions at Signalized Intersections	CMF_{1p}	Bus stops	Definition (HSM p. 12-46)
			HSM Table 12-28 (HSM p. 12-46)
	CMF_{2p}	Schools	Definition (HSM p. 12-46)
			HSM Table 12-29 (HSM p. 12-46)
	CMF_{3p}	Alcohol sales establishments	Definition (HSM p. 12-47)
			HSM Table 12-30 (HSM p. 12-47)

The SPFs were developed in HSM-related research from the most complete and consistent available data sets. However, the predicted crash frequencies may vary substantially from one jurisdiction to another for a variety of reasons. Calibration factors provide a method for incorporating local data to improve the estimated crash frequencies for individual locations. The local calibration factor accounts for the differences between the jurisdiction under investigation and the jurisdictions that were used to develop the default HSM SPFs. The local calibration factor is calculated using local crash data and other roadway characteristic data. The process for determining calibration factors for the predictive models is described in HSM Part C, Appendix A.1.

The predicted crash frequency under real conditions can be calculated using Equation 7:

$$N_{predicted} = \left(N_{spf\ x} \times \left(CMF_{1x} \times CMF_{2x} \times CMF_{3x} \times \ \dots \times CMF_{yx}\right) + N_{pedx} + N_{bikex}\right) \times C_x \quad \text{(Eq. 7)}$$

$$\textit{Segments:}\ \ N_{spf\ x} = N_{bmv\ x} + N_{bsv\ x} + N_{bdwy\ x} \quad \text{(Eq. 8)}$$

$$\textit{Intersections:}\ \ N_{spf\ x} = N_{bmv\ x} + N_{bsv\ x} \quad \text{(Eq. 9)}$$

where:

$N_{predicted}$ = predicted average crash frequency for a specific year on site type x

$N_{spf\ x}$ = base conditions predicted average crash frequency for site type x

$N_{bmv\ x}$ = base conditions predicted average crash frequency multiple-vehicle nondriveway collisions for site type x

$N_{bsv\ x}$ = base conditions predicted average crash frequency single-vehicle crashes for site type x

$N_{bdwy\ x}$ = base conditions predicted average crash frequency multiple-vehicle driveway-related collisions for site type x

N_{pedx} = predicted average crash frequency of vehicle-pedestrian collisions per year for site type x

N_{bikex} = predicted average crash frequency of vehicle-bicycle collisions per year for site type x

CMF_{yx} = CMFs specific to site type x and specified geometric design and traffic control features y

C_x = calibration factor to adjust SPF for local conditions for site type x

Step 3: Expected Crash Frequency with Empirical Bayes Method

This step can be omitted if no recorded crash data for the specific site under investigation were available or considered unreliable. When historical crash data is available, the EB method (either site-specific or project-level) is used to combine the HSM Chapter 12 predicted average crash frequency with the observed crash frequency. The expected crash frequency is a more statistically reliable estimate. The expected average crash frequency can be determined using Equation 10:

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed} \quad \text{(Eq. 10)}$$

where:

w = the weighted adjustment to be placed on the predictive model estimate.

The value can be calculated using the following equation:

$$w = \frac{1}{1 + k \times \sum_{all\ study\ years} N_{predicted}} \quad \text{(Eq. 11)}$$

where:

k = the overdispersion parameter of the associated SPF used to estimate $N_{predicted}$.

Table 22 lists the overdispersion values for urban and suburban arterials.

TABLE 22
SPFs Overdispersion Parameters in Chapter 12

Facility Type	Overdispersion Parameter (k)
Segments multiple-vehicle nondriveway collisions	Coefficients listed in HSM Table 12-3
Segments single-vehicle crashes	Coefficients listed in HSM Table 12-5
Segments multiple-vehicle driveway-related collisions	Coefficients listed in HSM Table 12-7
Intersections multiple-vehicle collisions	Coefficients listed in HSM Table 12-10
Intersections single-vehicle crashes	Coefficients listed in HSM Table 12-12
Intersections vehicle-pedestrian collisions	Coefficients listed in HSM Table 12-14

Step 4: Crash Frequency under Different Collision Types and Crash Severity Levels

HSM Chapter 12 provides the collision type distribution tables based on the crash severity level for roadway segments and intersections (Table 23). The crash frequency under different severity levels and collision types can be determined based on the distribution table after the predicted or expected crash frequencies are calculated. These proportions can be updated based on local data for a particular jurisdiction as part of the calibration process.

TABLE 23
Urban and Suburban Arterial Crash Severity and Collision Type Distributions

Facility Type	Collision Type	Crash Severity and Collision Type Distribution
Roadways Segments	Multiple-vehicle nondriveway collisions	HSM Table 12-4
	Single-vehicle crashes	HSM Table 12-6
	Multiple-vehicle driveway-related collisions	HSM Table 12-7
	Vehicle-pedestrian collisions	HSM Table 12-8
	Vehicle-bicycle collisions	HSM Table 12-9
Intersections	Multiple-vehicle collisions	HSM Table 12-11
	Single-vehicle crashes	HSM Table 12-13
	Vehicle-pedestrian collisions	HSM Table 12-16[a]
	Vehicle-bicycle collisions	HSM Table 12-17

Note:
[a] Pedestrian crash adjustment factors for stop-controlled intersections

Figure 20 illustrates the HSM Chapter 12 predictive method flowchart for calculating the predicted and expected crash frequency for urban and suburban arterial roads.

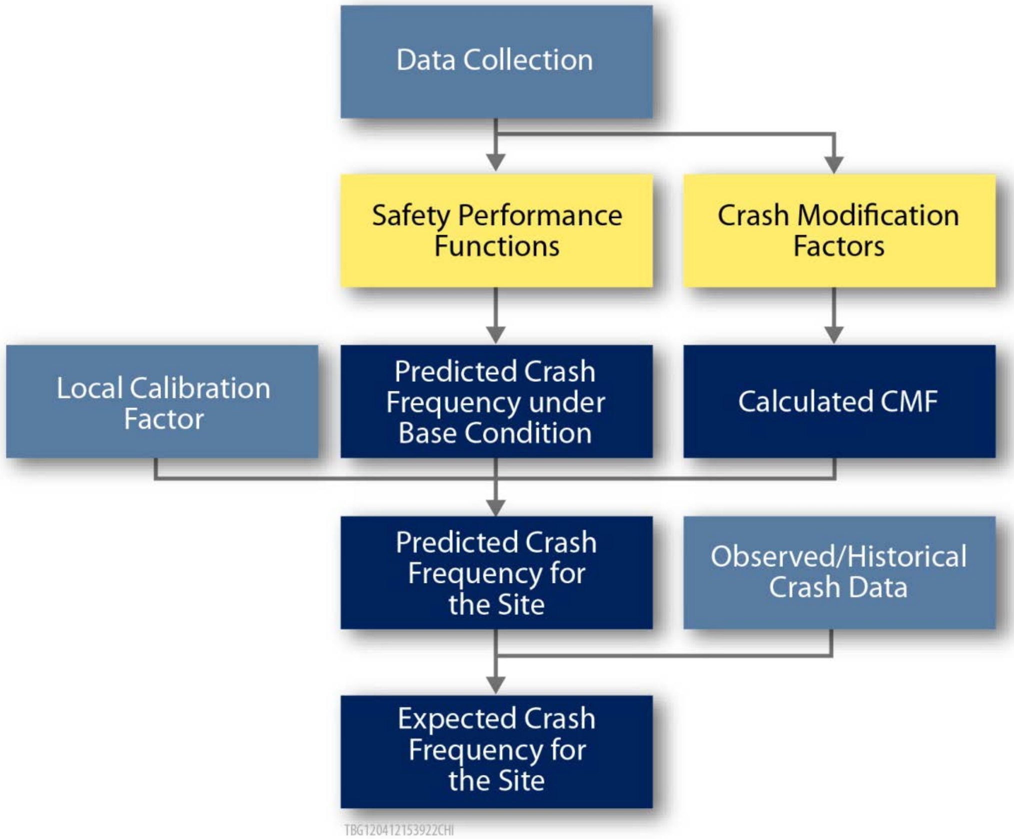


Figure 20: Flowchart for Calculating Expected Crash Frequency on Urban and Suburban Arterials

2.3.20 Data Requirements for Urban and Suburban Arterials

For the study period, it is important to determine the availability of AADT volumes, and for an existing roadway, the availability of observed crash data to determine whether the EB method is applicable.

In order to determine the relevant data needs and avoid unnecessary data collection, it is important to understand the SPFs' base conditions. The base conditions for urban and suburban arterials are defined in Section 2.3.19, as well as in HSM Section 12.6.1 (HSM p. 12-17) for roadway segments and in HSM Section 12.6.2 (HSM p. 12-28) for intersections.

General data for intersections and roadway segments can be collected from different sources. Examples of data sources include commercial aerial maps, design plans, and states' roadway inventory systems. Data needed for this example are summarized in the following sections.

Intersection Data

Generally, the effect of major and minor road traffic volumes (AADT) on crash frequency is incorporated through SPFs, while the effects of geometric design and traffic controls are incorporated through the CMFs. Data required to apply the predictive method for intersections are listed in Table 24.

TABLE 24
Intersection Data Requirements for Urban and Suburban Arterials

Intersections	Units/Description
Intersection type	Include unsignalized three-leg (3ST), signalized three-leg (3SG), unsignalized four-leg (4ST), and signalized four-leg (4SG)
Traffic flow major road	AADT (vpd)
Traffic flow minor road	AADT (vpd)
Intersection lighting	Present or not present
Calibration factor	Derived from calibration process
Data for unsignalized intersections only	
Number of major-road approaches with left-turn lanes	0, 1, or 2
Number of major-road approaches with right-turn lanes	0, 1, or 2
Data for signalized intersections only	
Number of approaches with left-turn lanes	0, 1, 2, 3, or 4
Number of approaches with right-turn lanes	0, 1, 2, 3, or 4
Number of approaches with left-turn signal phasing	0, 1, 2, 3, or 4
Type of left-turn signal phasing for all legs	Not applicable, permissive, protected, protected/permissive, or permissive/protected
Number of approaches with right-turn-on-red prohibited	0, 1, 2, 3, or 4
Intersection red-light cameras	Present or not present
Sum of all pedestrian crossing volumes-only signalized intersection	Sum of pedestrian volume
Maximum number of lanes crossed by a pedestrian	Number of lanes
Number of bus stops within 1,000 feet (300 meters) of intersection	Number
Schools within 1,000 feet (300 meters) of intersection	Number
Number of alcohol sales establishments within 1,000 feet (300 meters)	Number
Observed crash data	Applicable only with the EB method; crashes that occur at the intersection or intersection legs, and are related to the presence of an intersection during the period of study

Roadway Segment Data

The effect of traffic volume (AADT) on crash frequency is incorporated through an SPF, while the effects of geometric design and traffic control features are incorporated through the CMFs. Table 25 includes data requirements for roadway segment locations.

TABLE 25
Roadway Segment Data Requirements for Urban and Suburban Arterials

Roadway Segments	Units/Description
Roadway type (2U, 3T, 4U, 4D, ST)	Include two-lane undivided arterials (2U), three-lane arterials (3T) including a center TWLTL, four-lane undivided arterials (4U), four-lane divided arterials (4D), and five-lane arterials (5T) including a center TWLTL
Segment length	miles
Traffic volume	AADT (vpd)
Type of on-street parking	None, parallel, or angle
Proportion of curb length with on-street parking	percent
Median width – for divided only	Not present, or select from scale 10 feet to 100 feet
Lighting	Present or not present
Auto speed enforcement	Present or 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	Posted speed 30 mph or greater
Roadside fixed object density	Fixed objects per mile
Offset to roadside fixed objects	Length (feet)
Calibration factor	Derived from calibration process
Observed crash data	Applicable only with the EB method; crashes that occur between intersections and are not related to the presence of an intersection during the period of study

Note:
mph = miles per hour

2.4 HSM Part D: CMF Applications Guidance

HSM Part D provides information on estimating how effective a treatment, geometric characteristic, and operational characteristic will be in reducing crashes or injuries at a specific location. The effectiveness is expressed in terms of CMFs, trends, or no effect. The CMFs can be used to evaluate the expected average crash frequency with or without a particular treatment, or estimate the expected average crash frequency with one treatment versus a different treatment. CMFs are provided for roadway segments (HSM Chapter 13), intersections (HSM Chapter 14), interchanges (HSM Chapter 15), special facilities and geometric situations (HSM Chapter 16), and road networks (HSM Chapter 17).

Part D includes all CMFs in the HSM. Some Part D CMFs are included in Part C for use with specific SPFs. The remaining Part D CMFs can be used with the outcomes of the predictive method to estimate the change in crash frequency described in HSM Section C.7 (HSM p. C-19).

HSM Part D can be applied to the different stages of the project development process, as listed in Table 26.

TABLE 26
Stages of the Project Development Process

	System Planning	Project Planning	Preliminary Design	Final Design	Construction/ Implementation	Operation	Maintenance
HSM Part D		✓	✓	✓	✓	✓	✓
Other Relevant Chapters in the HSM	HSM Part B	HSM Part C HSM Ch. 5 HSM Ch. 6 HSM Ch. 7	HSM Part C HSM Ch. 6 HSM Ch. 7	HSM Part C HSM Ch. 6 HSM Ch. 7	HSM Part C HSM Ch. 6 HSM Ch. 7	HSM Ch. 5 HSM Ch. 6 HSM Ch. 7	HSM Ch. 5 HSM Ch. 6 HSM Ch. 7

HSM Part D introduces the following concepts:

- **Crash modification factor** – An index of how much crash experience is expected to change following a modification in design or traffic control. CMF is the ratio between the number of crashes per unit of time expected after a modification or measure is implemented and the number of crashes per unit of time estimated if the change does not take place.
- **Precision** – The degree to which repeated measurements are close to each other.
- **Standard error** – Indicates the precision of an estimated CMF. It is used as a measure of reliability of the CMF estimate. The smaller the standard error, the more reliable (less error) the estimate becomes. A CMF with a relatively high standard error means that a high range of results could be obtained with that treatment. It can also be used to calculate a confidence interval for the estimated change in expected average crash frequency. Refer to HSM Appendix 3C (HSM p. 3-44) for additional details about CMF and standard error.

- **CMF confidence interval** – It can be used to consider the possible range of the CMFs. For CMFs with high standard errors, the upper end of the confidence interval could be greater than 1.0 even if the CMF itself is relatively small, which means that the treatment could potentially result in an increase in crashes. Some CMFs in Part D are accompanied by a superscript when special awareness of the standard error is required.
- **Trend** – If the standard error was greater than 0.10, the CMF value was not sufficiently accurate, precise, and stable to be included in HSM Part D. In these cases, HSM Part D indicates a trend, if sufficient information is available. HSM Part D includes such information in the appendix at the end of each chapter. The HSM appendix also lists treatments with unknown crash effects.
- **Accuracy** – A measure of the proximity of an estimate to its actual or true value.

Part D CMFs were evaluated by an expert panel for inclusion in the HSM based on their standard error. Standard error values were used to determine the level of reliability and stability of the CMFs to be presented in the HSM. A standard error of 0.10 or less indicates a CMF value that is sufficiently accurate, precise, and stable. Some CMFs are expressed as functions, and do not have specific standard errors that could be used.

Understanding the standard error and reliability of the different CMFs will help analysts to build awareness of what can be expected from each safety treatment. A CMF with a high standard error does not mean that it should not be used; it means that if the CMF is used, the user should keep in mind the range of results that could be obtained.

2.4.1 HSM Chapter 13: Roadway Segments

HSM Chapter 13 provides the information used to identify effects on expected average crash frequency resulting from treatments applied to roadway segments. A roadway segment is defined as a continuous portion of a roadway with similar geometric, operational, and vehicular characteristics.

The more than 80 roadway segment treatments are classified based on the treatment characteristics. For each treatment category, the treatment CMF availability is provided in a table. The HSM table numbers for treatment summary information are listed in Table 27.

TABLE 27
Roadway Segments – HSM Table Number for Information on Treatment Summary

Treatment Category	HSM Table Number for Treatment Summary
Roadway element	HSM Table 13-1
Roadside element	HSM Table 13-17
Alignment element	HSM Table 13-26
Roadway sign	HSM Table 13-29
Roadway delineation	HSM Table 13-34
Rumble strip	HSM Table 13-43
Traffic calming	HSM Table 13-47
On-street parking	HSM Table 13-49
Roadway treatment for pedestrians and bicyclists	HSM Table 13-54
Highway lighting	HSM Table 13-55
Roadway access management	HSM Table 13-57
Weather issue	HSM Table 13-59

The CMFs for different treatments are usually provided in the format of figures, equations, or tables. Users may then determine the CMFs and relevant standard errors based on the treatment and the facility characteristics. When determining the CMFs for a specific treatment on a particular facility type, special attention should be paid to the AADT range, setting, and crash types for which the CMFs were developed.

For treatments without CMF values, the user can refer to HSM Appendix 13A to obtain information about the trend in crashes or user behavior (if available). HSM Appendix 13A also lists some treatments with unknown crash effects at the time the HSM was developed.

2.4.2 HSM Chapter 14: Intersections

HSM Chapter 14 provides information used to identify effects on expected average crash frequency resulting from treatments applied at intersections. An intersection is defined as the general area where two or more roadways join or cross, including the roadway and roadside facilities for traffic movements within the area.

There are more than 50 intersection treatments included in the HSM Part D, and they are classified based on the treatment characteristics. CMFs are organized into the following three categories: CMF is available; information available was sufficient to present a trend but not a CMF; and quantitative information is not available. For each treatment category, the treatment CMF availability is provided in a table. The HSM table numbers for treatment summary information are listed in Table 28.

TABLE 28
Intersections – HSM Table Number for Information on Treatment Summary

Treatment Category	HSM Table Number for Treatment Summary
Intersection type	HSM Table 14-1
Access management	HSM Table 14-8
Intersection design elements	HSM Table 14-9
Intersection traffic control and operational elements	HSM Table 14-19

The CMFs for different treatments are usually provided in the format of figures, equations, or tables. The users could then determine the CMFs and relevant standard errors based on the treatment and the facility characteristics. Special attention should be paid to the AADT range, setting, and the crash types used to develop the CMFs. This is particularly important when determining a CMF for a specific treatment on a particular facility type.

Treatments without CMF values indicate that research quantitative information was not enough to be included in the HSM. HSM Appendix 14A lists some treatments with unknown crash effects at the time the HSM was being developed.

2.4.3 HSM Chapter 15: Interchanges

HSM Chapter 15 provides the information used to identify effects on expected average crash frequency resulting from treatments applied at interchanges and interchange ramp terminals. An interchange is defined as a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways or highways on different levels, and an interchange ramp terminal is defined as an at-grade intersection where a freeway interchange ramp intersects with a non-freeway cross street.

The crash effects of interchange design elements are included in this chapter. The list of treatments included under interchange design elements and availability of relevant CMFs for different facility types are presented in HSM Table 15-1 at the beginning of HSM Section 15.4 (see Table 29).

TABLE 29
Interchanges – HSM Table Number for Information on Treatment Summary

Treatment Category	HSM Table Number for Treatment Summary
Interchange design elements	HSM Table 15-1

For treatments without CMF values, the user can refer to HSM Appendix 15A to determine whether sufficient information about potential trend in crashes or user behavior for the treatment could be found. HSM Appendix 15A lists some treatments with unknown crash effects at the time the HSM was being developed.

2.4.4 HSM Chapter 16: Special Facilities and Geometric Situations

HSM Chapter 16 provides CMFs for design, traffic control, and operational elements at various special facilities and geometric situations including highway-rail grade crossings, work zones, TWLTLs, and passing and climbing lanes. For each special facility or geometric situation, the list of treatments included and the availability of relevant CMFs for different facility types are provided in tables. Information from this table can be used to check the availability of the CMF for a specific treatment on a particular facility type. The HSM table numbers for treatment summary information are listed in Table 30.

TABLE 30
Special Facilities and Geometric Situations – HSM Table Number for Information on Treatment Summary

Treatment Category	HSM Table Number for Treatment Summary
Highway-rail grade crossing traffic control and operational elements	HSM Table 16-1
Work zone design elements	HSM Table 16-4
TWLTL elements	HSM Table 16-5
Passing and climbing lanes	HSM Table 16-6

For treatments without CMF values, the user can refer to HSM Appendix 16A to determine whether sufficient information about potential trends in crashes or user behavior for the treatment could be found. HSM Appendix 16A lists some treatments with unknown crash effects at the time the HSM was developed.

2.4.5 HSM Chapter 17: Road Networks

The information presented in HSM Chapter 17 is used to identify effects on expected average crash frequency resulting from treatments applied to road networks.

Nearly 20 treatments for road networks are included in HSM Chapter 17. The treatments for road networks are classified into categories based on the treatment characteristics. For each treatment category, a summary of treatments related to the specific treatment category, including a list of treatment and the availability of relevant CMFs for different facility types is provided in a table. Information from this table can be used to check the availability of the CMF for a specific treatment on a particular facility type. The HSM table numbers for treatment summary information are listed in Table 31.

TABLE 31
Road Networks – HSM Table Number for Information on Treatment Summary

Treatment Category	HSM Table Number for Treatment Summary
Network planning and design approaches/ elements	HSM Table 17-1
Network traffic control and operational elements	HSM Table 17-2
Road-use culture network considerations and treatments	HSM Table 17-4

For treatments without CMF values, the user can refer to HSM Appendix 17A to determine whether sufficient information about potential trend in crashes or user behavior for the treatment could be found. HSM Appendix 17A lists some treatments with unknown crash effects at the time the HSM was being developed.

Integrating the HSM in the Project Development Process

Program and project decisions are typically based on evaluation of costs, right-of-way, traffic operations, and environmental factors. The HSM provides science-based methods and a reliable approach for quantifying safety impacts in terms of crash frequency and severity, allowing agencies to incorporate it throughout the project development process. This section of the *Highway Safety Manual User Guide* provides examples for incorporating HSM approaches into each of the stages of the project development process: Planning, Alternatives Development and Analysis, Preliminary Design, Final Design and Construction, and Operations and Maintenance.

In the ***Planning*** phase, agencies assess conditions, evaluate future multimodal projects, identify locations with potential for crash reduction, and develop policies to address long-term transportation system needs, among other tasks. The planning phase includes development of the agency's long-range program. The program may account for projects for the next 5 years and that are prioritized based on a number of factors including safety. Application of safety in decisions at a planning level may include developing statewide policies that incorporate quantitative safety implications to reduce the number and severity of crashes in the long term. Incorporation of safety performance at this stage improves the likelihood of cost-effective resource allocation. HSM Part B provides information for planning applications.

Individual projects derived from the agency planning efforts move into the ***Alternatives Development and Analysis*** phase. In this phase, multiple alternatives are developed and evaluated. Project decisions are based on evaluation of costs, right-of-way, traffic operations, environmental assessment, and safety. The HSM Part C predictive methods allow agencies to quantify a project's potential for crash reduction, or to apply the predictive method and compare the safety performance of different alternatives associated with a change in traffic volume, traffic control, or geometrics.

After a preferred alternative has been selected, the next phase is the ***Preliminary and Final Design***. Tools provided in the HSM can help designers reach informed decisions throughout ***Final Design and Construction***. Some applications include the incorporation of human factor considerations into design, analysis, decision-making, and documentation of the quantitative safety effects of a proposed design exception.

The HSM can also be used in the ***Operations and Maintenance*** of an agency's daily operations. The HSM can be incorporated into processes used to monitor system performance, such as considering the impact of changes or upgrades in mobility, decisions related to access, setting maintenance policies and priorities, and other operational considerations on safety performance.

This guide will provide examples of HSM applications in the different phases of the project development process, with the intent to provide agencies with opportunities to use safety performance as a consideration in their decision-making process.

3.1 HSM in the Planning Phase

3.1.1 Overview

The main goal of system planning is to provide decision makers with the information needed to make choices about investments in their transportation system. In the planning phase, agencies evaluate the multimodal transportation system and identify priorities, programs, and policies to address long-range transportation needs. The HSM can be used to estimate the safety performance of alternative transportation networks and understand safety implications so that reducing the cost of crashes and saving lives can be compared with other performance metrics.

HSM Part B provides the process for planning applications and presents steps to monitor and reduce crash frequency and severity on existing road networks. The HSM Part B includes methods useful for identifying sites for improvement (HSM Chapter 4), diagnosis (HSM Chapter 5), safety countermeasure selection (HSM Chapter 6), economic appraisal (HSM Chapter 7), project prioritization (HSM Chapter 8), and effectiveness evaluation (HSM Chapter 9).

The following section includes an application of HSM in planning using the different chapters included in Part B.

3.1.2 Example Problem 1: Planning Application using HSM Part B

Introduction

A county DOT is working on preparing their long-range safety program and chose to use the HSM roadway safety management process to maximize limited safety resources to save lives and reduce serious injuries for routes and intersections within its jurisdiction.

The roadway safety management process from network screening to safety effectiveness evaluation will be applied in this example.

Step 1: Network Screening

Data Requirements

- Crash data for the selected county route system
- Roadway network information for the selected county route system

Analysis

Since the county DOT will identify projects for the county safety program, all sites within the county route system should be screened. Both intersections and roadway segments will be included as elements for the network screening process. The reference populations include rural two-lane undivided highways for roadway segments, as well as all-way stop-controlled intersections and four-leg signalized intersections.

The roadway segment mileage information was available and the selected performance measure needed to account for regression-to-the-mean. The excess expected average crash frequency with EB adjustments was selected as the performance measure for the intersections and roadway segments (Figure 21). The screening methods used for the intersections and roadway segments are the simple ranking method and the sliding-window method, respectively.

More data; Account for RTM bias

Performance Measure	Accounts for RTM Bias	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 Severity Index	No	Yes
Critical Rate	Considers data variance; does not account for RTM bias	Yes
Excess Predicted Average Crash Frequency using Method of Moments	Considers data variance; does not account for RTM bias	Yes
Level of Service of Safety	Considers data variance; 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; does not account for RTM bias	Yes
Excess Proportions of Specific Crash Types	Considers data variance; does not account for RTM bias	Yes
Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency at the site
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency at the site
Excess Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency per year at the site

Greater Reliability

Figure 21: Available Performance Measures (HSM Table 4-2 [HSM p. 4-9])

Results and Discussion

After the network screening process, the average excess expected average crash frequency (EEACF) with EB adjustments was calculated for all the intersections and roadway segments, and ranked in descending order. The top five intersections and top five roadway segments (listed in Table 32) will be selected as candidates for the next step.

TABLE 32
Example Problem 1 – Network Screening Process – Intersection and Roadway Segment Rankings

Rank	Intersection ID	EEACF with EB	Rank	Roadway Segment ID	EEACF with EB
1	17	18.2	1	52	7.6
2	83	16.4	2	72	5.1
3	25	15.8	3	105	5.0
4	68	12.2	4	35	3.3
5	46	9.8	5	81	3.5

Notes:
EB = Empirical Bayes
EEACF = excess expected average crash frequency
ID = identification number

Step 2: Diagnosis

Data Requirements

- Crash data for the selected five roadway segments and five intersections
- Supporting documentation including current traffic volumes for all travel modes, inventory of field conditions, and relevant photo or video logs

Analysis

Descriptive crash statistics for the selected five roadway segments and five intersections were developed. Information on crash type, crash severity, roadway, and environmental conditions was displayed with bar charts, pie charts, and tabular summaries to gain a better understanding of potential issues. Locations for intersection and roadway segment crashes were summarized using collision diagrams and crash maps, respectively.

Additional supporting documentation was reviewed; including traffic signs, traffic control devices, number of travel lanes, and posted speed limits. A field visit was arranged by the county traffic engineers to understand issues identified and to verify opportunities to reduce crash potential.

Results and Discussion

Based on the analysis results, a large proportion of the roadway segment crashes were roadway departure involving high speed. The percentages of nighttime crashes, during inclement weather conditions, and that were ice/snow-related were also relatively high for roadway segments.
For intersections, rear-end and angle crashes are overrepresented. Most of the rear-end crashes occurred on the major road approaches, and a high percentage of the at-fault vehicles were speeding. The angle crashes were overrepresented at some unsignalized intersections, which were the result of vehicles making left turns from minor roads or driveways onto the main road.

The field condition inventory indicated that the posted speed limit was 50 mph for most roadway segments. The field visit revealed that no Chevrons were installed for curves on roadway segments, and most of the roadway segments did not have rumble strips. The pavement drainage was operating at less than full capacity, resulting in potential flooding for some roadway segments and intersections.
The clearance time at some signalized intersections did not seem to provide enough time for vehicles to clear the intersections, and there were some obstructions (such as bushes) on the roadside limiting the stopping sight distance on the main road, minor roads, and driveways. At intersections, traffic signals did not have backplates, and there was only one signal head for all through travel lanes.

Step 3: Select Countermeasures

Data Requirements

No additional data are required for selecting the proper safety countermeasures.

Analysis

The contributing factors for roadway segment and intersection crashes were identified based on the information derived from the crash data analysis and the field visit process. The possible contributing factors for prevailing crash types were identified from the perspective of human, vehicle, and roadway before, during, and after the crashes.

Results and Discussion

The contributing factors and selected safety countermeasures for different facility types and crash types are listed in Table 33.

TABLE 33
Example Problem 1 – Contributing Factors and Selected Safety Countermeasures

Facility Type	Crash Type	Contributing Factor	Safety Countermeasure	Selected Location ID
Intersection	Rear-end	High approach speed	Install automated speed enforcement	83
		Slippery pavement	Install high-friction surface treatment	25
		Poor visibility of signals	Install one traffic signal head per lane and add backplates	68, 25
			Install flashing beacons as advance warning	25
	Angle	Limited sight distance	Increase sight distance triangle	17, 25
		High approach speed	Install automated speed enforcement	46, 17
		Poor visibility of signal	Install one traffic signal head per lane and add backplates	25
Roadway Segment	Roadway departure	Poor delineation	Install Chevrons on curved segment	105, 81
		Excessive speed	Install automated speed enforcement	35, 105
		Drive inattention	Install shoulder rumble strips	52, 72
		Slippery pavement	Install high-friction surface treatment	81

Step 4: Economic Appraisal

Data Requirements

- Crash data for selected roadway segments and intersections
- Current and future AADT values
- CMFs for all safety countermeasures under consideration
- Construction and implementation costs for each countermeasure
- Monetary value of crashes by severity
- Service life of the countermeasures

Analysis

The economic appraisal process outlined in this example only considers changes in crash frequency and does not consider project benefits from travel time, environmental impacts, or congestion relief. The method selected for conducting the economic appraisal of this example is the benefit-cost ratio (BCR).

The predictive method presented in HSM Part C was applied to determine the expected crash frequency for existing conditions and proposed alternatives. The expected change in average fatal, injury, and PDO crash frequency was then converted to a monetary value using the societal cost of crashes listed in HSM Table 7-1. The annual monetary value was further converted to present value using a discount rate.

The costs for implementing the selected safety countermeasures, right-of-way acquisition, construction material costs, utility relocation, maintenance, and other costs were added together to obtain the present values of the project costs. The BCR for each project was calculated based on the present value of the project benefits and project costs.

Results and Discussion

The benefits and costs for each proposed project and the relevant BCR are listed in Table 34.

TABLE 34
Example Problem 1 – Proposed Projects Benefit-Cost Ratio

Project	Facility ID	Benefit	Cost	Benefit-Cost Ratio
Increase triangle sight distance	Intersection 17	$34,500	$9,000	3.8
	Intersection 25	$32,000	$11,000	2.9
Install one traffic signal head per lane and add backplates	Intersection 68	$26,300	$7,800	3.4
	Intersection 25	$28,650	$6,900	4.2
Install flashing beacons as advanced warning	Intersection 25	$30,750	$10,600	2.9
Install Chevrons	Roadway segment 105	$200,500/mile	$80,700/mile	2.5
	Roadway segment 81-1	$180,650/mile	$59,800/mile	3.0
Install shoulder rumble strips	Roadway segment 72	$90,800/mile	$38,500/mile	2.4
	Roadway segment 52	$102,500/mile	$42,980/mile	2.4
Install high-friction surface treatment	Roadway segment 81-2	$250,200/mile	$190,080/mile	1.3
	Intersection 25	$85,650	$59,000	1.5
Install automated speed enforcement	Intersection 83	$57,000	$25,000	2.3
	Intersection 46	$63,000	$27,500	2.5
	Intersection 17	$72,000	$26,000	2.9
	Roadway segment 35	$87,000	$23,000	3.5
	Roadway segment 105	$92,000	$29,000	3.7

Step 5: Prioritize Projects

Data Requirements

No additional data are required for selecting the proper safety countermeasures.

Analysis

An incremental benefit-cost analysis was conducted for the project prioritization.

The incremental BCR is an extension of the BCR method. Projects with a BCR greater than 1.0 are arranged in increasing order based on their estimated cost. Then HSM Equation 8-3 (Page 8-11) is applied to project pairs. If the incremental BCR is greater than 1.0, the higher cost project is the preferred one. Conversely, if the incremental BCR is less than 1.0, or is zero or negative, the lower cost project is preferred to the higher cost project. The calculations continue comparing the preferred project from the first pair to the next highest cost. Additional details on this method can be found in HSM Section 8.2.1, Ranking Procedures (HSM p. 8-3). Table 35 provides the first sequence of incremental benefit-cost comparisons needed to assign priority to projects. From this table, the improvement project for roadway segment 81 – Install Chevrons receives the highest priority (green text in the table).

TABLE 35
Example Problem 1 – Incremental BCR Analysis

Comparison	Project	Project ID	$PV_{benefits}$	PV_{costs}	BCR	Incremental BCR	Preferred Project
1	Install one traffic signal head per lane and add backplates	Int 25	$28,650	$6,900	4.15	(2.61)	Int 25
	Install one traffic signal head per lane and add backplates	Int 68	$26,300	$7,800	3.37		
2	Install one traffic signal head per lane and add backplates	Int 25	$28,650	$6,900	4.15	2.79	Int 17
	Increase triangle sight distance	Int 17	$34,500	$9,000	3.83		
3	Increase triangle sight distance	Int 17	$34,500	$9,000	3.83	(2.34)	Int 17
	Install flashing beacons as advanced warning	Int 25	$30,750	$10,600	2.9		
4	Increase triangle sight distance	Int 17	$34,500	$9,000	3.83	(1.25)	Int 17
	Increase triangle sight distance	Int 25	$32,000	$11,000	2.91		
5	Increase triangle sight distance	Int 17	$34,500	$9,000	3.83	2.88	Seg 105
	Install automated speed enforcement	Seg 105	$92,000	$29,000	3.68		
6	Install automated speed enforcement	Seg 105	$92,000	$29,000	3.68	0.83	Seg 35
	Install automated speed enforcement	Seg 35	$87,000	$23,000	3.48		
7	Install automated speed enforcement	Seg 35	$87,000	$23,000	3.48	(5.00)	Seg 35
	Install automated speed enforcement	Int 17	$72,000	$26,000	2.88		
8	Install automated speed enforcement	Seg 35	$87,000	$23,000	3.48	(5.33)	Seg 35
	Install automated speed enforcement	Int 46	$63,000	$27,500	2.52		
9	Install automated speed enforcement	Seg 35	$87,000	$23,000	3.48	(15.00)	Seg 35
	Install automated speed enforcement	Int 83	$57,000	$25,000	2.28		

TABLE 35
Example Problem 1 – Incremental BCR Analysis

Comparison	Project	Project ID	$PV_{benefits}$	PV_{costs}	BCR	Incremental BCR	Preferred Project
10	Install automated speed enforcement	Seg 35	$87,000	$23,000	3.48	0.25	Seg 35
	Install shoulder rumble strips	Seg 72	$90,800	$38,500	2.36		
11	Install automated speed enforcement	Seg 35	$87,000	$23,000	3.48	0.78	Seg 35
	Install shoulder rumble strips	Seg 52	$102,500	$42,980	2.38		
12	Install automated speed enforcement	Seg 35	$87,000	$23,000	3.48	(0.04)	Seg 35
	Install high-friction surface treatment	Int 25	$85,650	$59,000	1.45		
13	Install automated speed enforcement	Seg 35	$87,000	$23,000	3.48	2.54	Seg 81
	Install Chevrons	Seg 81	$180,650	$59,800	3.02		
14	Install Chevrons	Seg 81	$180,650	$59,800	3.02	0.95	Seg 81
	Install Chevrons	Seg 105	$200,500	$80,700	2.48		
15	**Install Chevrons**	**Seg 81**	**$180,650**	**$59,800**	**3.02**	0.53	Seg 81
	Install high-friction surface treatment	Seg 81	$250,200	$190,080	1.32		

Notes:
Int = intersection
PV = present value
Seg = roadway segment
Green text = highest priority

The process is repeated to assign priorities to the remaining projects. Successive series of incremental BCR calculations are performed, each time removing the projects previously prioritized.

Results and Discussion

The incremental BCR analysis results are summarized in Table 36. This method provides a priority-ranking list of projects based on whether the expenditure represented by each increment of additional cost is economically justified. BCR analysis provides additional insight into priority ranking but does not necessarily incorporate a formal budget constraint.

TABLE 36
Example Problem 1 – Ranking Results of Incremental BCR Analysis

Rank	Project ID	Project
1	Roadway segment 81	Install Chevrons
2	Roadway segment 105	Install Chevrons
3	Roadway segment 35	Install automated speed enforcement
4	Roadway segment 105	Install automated speed enforcement

TABLE 36
Example Problem 1 – Ranking Results of Incremental BCR Analysis

Rank	Project ID	Project
5	Roadway segment 81	Install high-friction surface treatment
6	Roadway segment 52	Install shoulder rumble strips
7	Roadway segment 72	Install shoulder rumble strips
8	Intersection 17	Install automated speed enforcement
9	Intersection 46	Install automated speed enforcement
10	Intersection 83	Install automated speed enforcement
11	Intersection 25	Install high-friction surface treatment
12	Intersection 17	Increase triangle sight distance
13	Intersection 25	Install one traffic signal head per lane and add backplates
14	Intersection 25	Increase triangle sight distance
15	Intersection 25	Install flashing beacons as advanced warning
16	Intersection 68	Install one traffic signal head per lane and add backplates

Step 6: Safety Effectiveness Evaluation

Data Requirements

- Minimum of 10 sites at which the treatment has been implemented
- Minimum of 3 years of crash data and traffic volume for the period before implementation
- Minimum of 3 years of crash data and traffic volume for the period after implementation
- Safety performance function for the facility types being evaluated

Analysis

An EB before/after safety evaluation method was conducted for the safety effectiveness evaluation.

The county DOT decided to upgrade all its signalized intersections to one signal head per travel lane. The safety effectiveness is analyzed using the EB before/after safety evaluation method to assess the overall effect of signal upgrades. To simplify things, the county DOT assumed a constant AADT across all years for both the before and after periods. The county DOT also assumed that all the intersections match the base conditions; therefore, the applicable CMFs and calibration factor are 1.0.

The EB method is used to compare crash frequencies at a group of sites before and after a treatment is implemented. The EB method addresses the regression-to-the-mean (RTM) issue.

The process begins by estimating the before and after predicted average crash frequency using the sites' SPF. Then, an adjustment factor is calculated to account for differences between the before and after periods in number of years and traffic volume at each site. The adjustment factor is obtained by dividing the after predicted crash frequency by the before predicted crash frequency. Next, the expected average crash frequency over the entire after period, in the absence of the treatment, is calculated. The safety effectiveness of the treatment at each site is estimated by dividing the observed crash frequency in the after period by the expected average crash frequency in the after period without treatment. The safety effectiveness is then converted into a percentage crash change for each site. Lastly, the overall unbiased safety effectiveness as a percentage change in crash frequency is obtained using the overall effectiveness of the treatment for all sites, overall variance, and total expected average crash frequency in the after period without treatment. A similar example that can be used as a reference can be found in HSM Section 9.10.

Results and Discussion

Results of this evaluation indicated that there is an overall positive safety effectiveness of 25.5 percent (reduction in total crash frequency) with a standard error of 12.7 percent after the application of the treatment or an overall safety benefit between 12.8 and 38.2. The statistical significance of the estimated safety effectiveness is 2.3 (greater than 2), which indicates that the treatment is significant at the 95-percent confidence level.

Tools Available for Part B Application

The *SafetyAnalyst* set of software tools was developed as a cooperative effort by the FHWA and participating state and local agencies. It provides analytical tools for use in the decision-making process to identify and manage a system-wide program of site-specific improvements to enhance highway safety by cost-effective means. *SafetyAnalyst* software tools are used by state and local highway agencies for highway safety management. AASHTO manages distribution, technical support, maintenance, and enhancement of *SafetyAnalyst* as a licensed AASHTOware product.

3.2 HSM in the Alternatives Development and Analysis Phase

3.2.1 Overview

After the multiyear programs are developed and system-wide and corridor needs are identified, the next step is implementation of the elements of the program. Projects are selected for development. The scope of work and purpose and need are established, and the project moves forward to the *Alternatives Development and Analysis* phase. In this phase, multiple alternatives are developed and evaluated to address in the project's purpose and need. Typically, project decisions are based on evaluation of costs, right-of-way, traffic operations, environmental assessment, and safety evaluation. Agencies can now apply the HSM science-based methods to support explicit consideration of quantitative safety. The HSM allows agencies to quantify a project's potential for crash reduction, or to apply the predictive method and compare the safety performance of different alternatives associated with a change in traffic volume or traffic control.

The following section provides examples of HSM application to different facility types for which SPFs have been developed.

3.2.2 Example Problem 2: Rural, Two-Lane, Two-Way Roads and Rural Multilane Highway

Introduction

A State Route (SR) has been identified by the state DOT as one of the top 5 percent locations in the 2012 Highway Safety Improvement Program (HSIP) report. This 3.9-mile, rural two-lane road, classified as a principal arterial, runs in an east-west direction (Figure 22). The SR has 12-foot lanes with 1-foot gravel shoulders, and the posted speed limit is 55 mph. Trees and vegetation are present along the edge of the road. There are three stop-controlled three-leg intersections located at mileposts (MP) 100.00, 100.78, and 102.95.

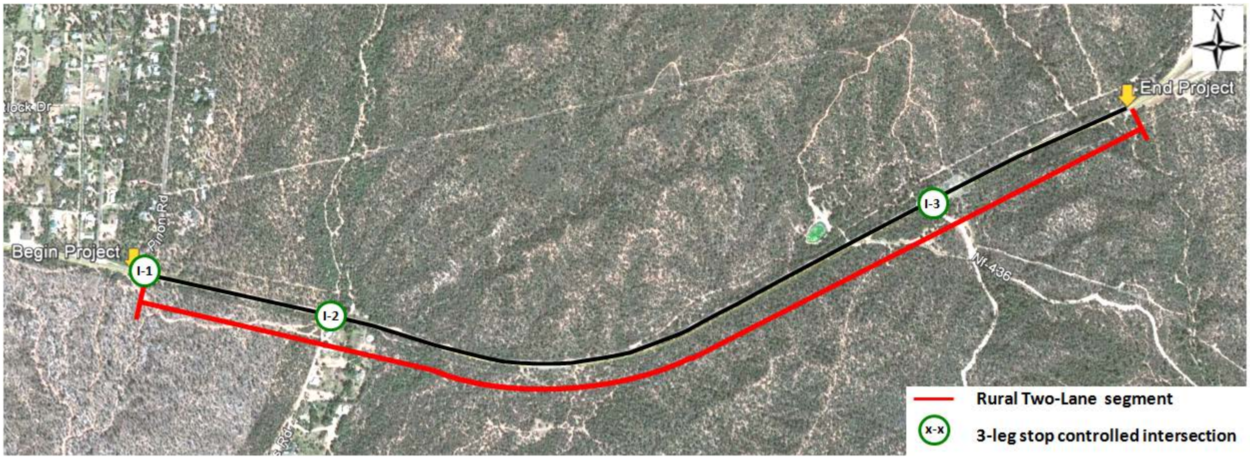


Figure 22: State Route Rural Two-Lane, Two-Way Road

Results from the crash analysis indicate a high proportion of head-on, sideswipe-opposing, and fixed object crashes along the roadway, particularly in the curve. A high proportion of angle crashes have also occurred at the intersections. In addition, descriptive crash statistics indicate that there could be an issue with drivers speeding on the SR.

There are 5 years of observed crash data (2008 to 2012) and traffic volumes available for the analysis. Evaluations of existing and future conditions and the conversion to a four-lane divided roadway are described in the following subsections.

Objectives

This example was developed to evaluate the existing safety performance of the SR corridor, perform an alternatives analysis, and determine the safety impacts of the conversion from a two-lane rural road to a four-lane divided roadway for future conditions. Similarly, different improvements for intersections were tested.

The first part of the example shows how to calculate the predictive average crash frequency for a rural two-lane stop-controlled intersection and a rural two-lane roadway segment; how to combine intersections and roadway segments as part of a corridor study; and the analysis of the two different alternatives. A third alternative involving a conversion from two-lane to four-lane divided is also considered. However, the CMF for conversion from two- to four-lane highways is only applicable to a short length of highway. Longer roadway segments are out of the scope of the two-lane rural roads methodology and can be addressed with the rural multilane highway procedures.

The second part shows how to calculate the predictive average crash frequency for a rural multilane stop-controlled intersection and rural multilane divided roadway under existing and future conditions (2030) and how to combine intersections and roadway segments as part of a corridor study. The different rural two-lane and rural multilane alternatives under future conditions (2030) will be compared. The last part is focused on discussing the results of the analysis.

The objective of the example is to show how various HSM analysis tools can be applied to assist traffic analysts, engineers, planners, and decision-makers in making sound investment decisions. In some situations, this amount of analysis would not be necessary to make an informed decision, but the issues presented herein should always be considered to assure the final decision is consistent with safety performance objectives.

After reviewing this example, the user should be able to:

- Understand what input data are required and the assumptions that are commonly made regarding default values for the HSM procedures
- Calculate the predicted and expected crash frequency of rural two-lane two-way road intersections and roadway segments using the HSM
- Calculate the predicted crash frequency of rural multilane intersections and segments using the HSM
- Understand how to reasonably interpret the results from an HSM analysis, and how these results can be used to support a particular decision
- Understand the limitations of the HSM procedures and when it is appropriate to use other models or computational tools

3.2.3 Part 1 – Rural Two-Lane Two-Way Roads

Data Requirements for Part 1

The sample corridor was divided into three roadway segment sections (two tangents and one curve), as shown in Figure 23. Crash data were assigned to intersections and roadway segments. The intersections and roadway segments characteristics are summarized in Tables 37 and 38. AADT information provided in the summary table corresponds to year 2012.

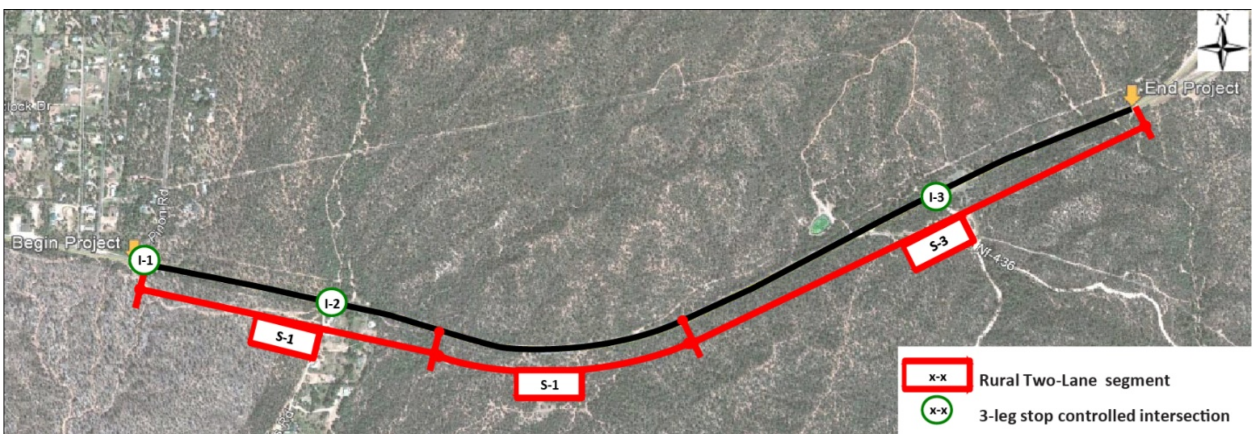


Figure 23: Example Problem 1 – Sample Rural Two-Lane, Two-Way Road

Intersection Data

Table 37 lists intersection input data for the example.

TABLE 37
Example Problem 2 – Intersections Input Data

Intersection Characteristics	Input Data		
	Intersection 1	Intersection 2	Intersection 3
Intersection type	3ST	3ST	3ST
Traffic flow major road (vpd)	9,000	9,000	9,000
Traffic flow minor road (vpd)	2,500	3,000	1,200
Intersection skew angle (degrees)	0	0	15

TABLE 37
Example Problem 2 – Intersections Input Data

Intersection Characteristics	Input Data		
	Intersection 1	Intersection 2	Intersection 3
Number of signalized or uncontrolled approaches with a left-turn lane	0	0	0
Number of signalized or uncontrolled approaches with a right-turn lane	0	0	0
Intersection lighting	Not present	Not present	Not present
Calibration factor (C_i)	1.17	1.17	1.17
Observed crash data (crashes/year)	4	5	2

Note:
vpd = vehicles per day

Roadway Segment Data

Table 38 summarizes the roadway segment input data for the example.

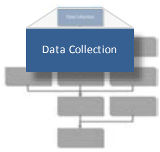
TABLE 38
Example Problem 2 – Roadway Segment Input Data

Characteristics	Input Data		
	Roadway Segment 1	Roadway Segment 2	Roadway Segment 3
Segment length (miles)	1.17	0.78	1.95
Traffic volume (vpd)	9,000	9,000	9,000
Lane width (feet)	12	12	12
Shoulder width (feet)	1	1	1
Shoulder type	Paved	Paved	Paved
Length of horizontal curve (miles)	0	0.78	0
Radius of curvature (feet)	0	2650	0
Spiral transition curve	Not present	Not present	Not present
Superelevation variance (feet/foot)	0	0.02	0
Grade (%)	2	2	2
Driveway density	1.7	0	4.5
Centerline rumble strips	Not present	Not present	Not present
Passing lanes	Not present	Not present	Not present
TWLTL	Not present	Not present	Not present
Roadside hazard rating	5	5	5
Segment lighting	Not present	Not present	Not present
Auto speed enforcement	Not present	Not present	Not present
Calibration factor (C_r)	1.30	1.30	1.30
Observed crash data (crashes/year)	11	40	11

Analysis

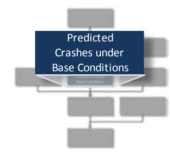
The rural two-lane, two-way predictive method for intersections and roadway segments under existing conditions (year 2012) was applied in the following subsections. For illustrative purposes, detailed calculations are included only for Intersection 3 and Roadway Segment 2.

Intersections



The first part of the predictive method is focused on defining the limits, facility type, and study period as well as obtaining and preparing input datasets required to apply the predictive models. Detailed information related to data collection can be found in HSM Section 10.4. The data summary for this example is provided in Table 37.

Select and Apply SPFs



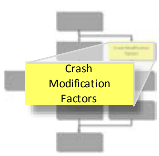
HSM Chapter 10 intersection SPFs are used to calculate the total predicted average crash frequency per year for crashes that occur within the limits of the intersection.

To determine the predictive average crash frequency of the sample intersection, select and apply the appropriate SPF for the facility type and traffic control features.
The predicted crash frequency for a three-leg, stop-controlled intersection (Intersection 3) can be calculated using HSM Equation 10-8 (HSM p. 10-18):

$$N_{spf\ 3ST} = e^{-9.86+0.79\times ln(AADT_{major})+0.49\times ln(AADT_{minor})}$$

$$N_{spf\ 3ST} = e^{-9.86+0.79\times ln(9{,}000)+0.49\times ln(1{,}200)} = 2.24\ crashes/year$$

Apply HSM Part C Crash Modification Factors



The SPF predictions are then multiplied by the appropriate CMFs to adjust the estimated crash frequency for base conditions to the site-specific geometry and traffic features.

Intersection Skew Angle (CMF_{1i})

CMF_{1i} can be calculated using HSM Equation 10-22 (HSM p. 10-31) for a 3ST intersection. The intersection skew angle is 15 degrees:

$$CMF_{1i\ 3ST} = e^{(0.004\times skew)}$$

$$CMF_{1i\ 3ST} = e^{(0.004\times 15)} = 1.06$$

Intersection Left-Turn Lanes (CMF_{2i})

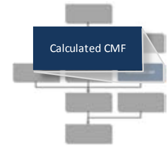
No left-turn lanes are present at the example intersection; HSM Table 10-13 (HSM p. 10-32) provides the CMFs for the presence of left-turn lanes. The selected site does not have left-turn lanes; therefore, a CMF of 1.00 is used.

Intersection Right-Turn Lanes (CMF_{3i})

No right-turn lanes are present at the example intersection; HSM Table 10-14 (HSM p. 10-33) provides the CMFs for the presence of right-turn lanes. The selected site does not have right-turn lanes; therefore, a CMF of 1.00 is applied.

Intersection Lighting (CMF_{4i})

HSM Equation 10-24 and HSM Table 10-15 (HSM p. 10-33) are used to estimate the CMF for lighting. Lighting is not present at the sample intersection; therefore, a CMF of 1.00 is applied.



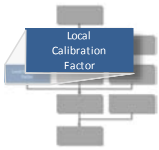
The combined CMF is calculated by multiplying all the intersection CMFs:

$$CMF_{combined} = CMF_{1i} \times CMF_{2i} \times CMF_{3i} \times CMF_{4i}$$

$$CMF_{combined} = 1.06 \times 1.00 \times 1.00 \times 1.00$$

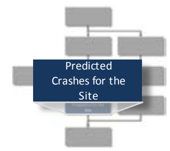
$$CMF_{combined} = 1.06$$

Apply Calibration Factor



The next step is to multiply the results obtained above by the appropriate calibration factor. For this example, the calibration factor for stop-controlled three-leg intersections has been assumed to be 1.17. Users can use a local calibration factor, if available.

Obtain the Predicted Crash Frequency for the Site



The predicted average crash frequency for Intersection 3 is calculated using HSM Equation 10-3 (HSM p. 10-4), combining results from previous steps:

$$N_{predicted\ int} = N_{spf\ int} \times C_i \times (CMF_{1i} \times CMF_{2i} \times CMF_{3i} \times CMF_{4i})$$

$$N_{predicted\ int} = 2.24 \times 1.17 \times (1.06 \times 1.00 \times 1.00 \times 1.00)$$

$$N_{predicted\ 3ST} = 2.78\ crashes/year$$

Multiyear Analysis

Since 5 years of data are available, all the previous steps need to be repeated four more times. In this example, a growth rate of 2 percent is assumed. Table 39 summarizes the calculations for Intersection 3.

TABLE 39
Example Problem 2 – Intersection 3 Multiyear Analysis Results

Intersection 3	Year				
	2008	2009	2010	2011	2012
$AADT_{major}$	8,315	8,481	8,651	8,824	9,000
$AADT_{minor}$	1,109	1,131	1,153	1,176	1,200
Crashes/year	1	0	4	3	2
$N_{spf\ 3ST}$	2.03	2.08	2.13	2.19	2.24
$CMF_{1i\ 3ST}$	1.06	1.06	1.06	1.06	1.06
$CMF_{2i\ 3ST}$	1.00	1.00	1.00	1.00	1.00
$CMF_{3i\ 3ST}$	1.00	1.00	1.00	1.00	1.00
$CMF_{4i\ 3ST}$	1.00	1.00	1.00	1.00	1.00
CMF_{comb}	1.06	1.06	1.06	1.06	1.06
C_i	1.17	1.17	1.17	1.17	1.17
$N_{predicted\ int}$	2.52	2.58	2.65	2.71	2.78

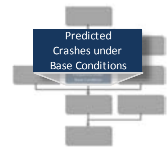
Notes:
$AADT_{major}$ = average annual daily traffic on the major route
$AADT_{minor}$ = average annual daily traffic on the minor route
CMF_{comb} = combined CMF
N_{spf} = predicted average crash frequency estimated for base conditions
$N_{predicted\ int}$ = predicted average crash frequency for the intersection

The average predicted crash frequency for Intersection 3 is obtained by the arithmetic average of the annual predicted crash frequencies ($N_{predicted\ int}$). For this example, this value is 2.65 crashes per year.

Roadway Segments

Roadway segment data required to apply the predictive method are summarized in Table 38. Roadway Segment 2 is a curve with a radius of 2,650 feet. No spiral transitions are present. Information on different recommendations related to data collection is presented in HSM Section 10.4.

Select and Apply SPFs

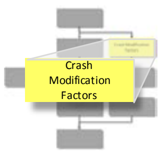


For the selected site, apply the SPF appropriate for rural two-lane, two-way roads. The SPF can be calculated using HSM Equation 10-6 (HSM p. 10-15):

$$N_{spf} = AADT \times L \times 365 \times 10^{-6} \times e^{-0.312}$$

$$N_{spf} = 9{,}000 \times 0.78 \times 365 \times 10^{-6} \times e^{-0.312} = 1.88\ crashes/year$$

Apply HSM Part C Crash Modification Factors



Multiply the result obtained above by the appropriate CMFs to adjust the estimated crash frequency for base conditions to the site-specific geometry and traffic features.

Lane Width (CMF_{1r})

CMF_{1r} can be calculated using HSM Equation 10-11 (HSM p. 10-24):

$$CMF_{1r} = (CMF_{ra} - 1) \times p_{ra} + 1$$

CMF_{ra} is estimated using HSM Table 10-8 (HSM p. 10-24). For a 12-foot lane width and AADT greater than 2,000, the CMF for the effect of lane width on related crashes (such as single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes) is 1.00.

For this example, the default crash severity distribution (HSM Table 10-4 [HSM p. 10-17]) is assumed, yielding the total percent of related crashes as:

$$p_{ra} = \%\ \text{run off road} + \%\ \text{head} - \text{on} + \%\ \text{sideswipe}$$

$$= 52.1 + 1.6 + 3.7$$

$$= 57.4\%$$

The lane width CMF is then:

$$CMF_{1r} = (CMF_{ra} - 1) \times p_{ra} + 1$$

$$CMF_{1r} = (1 - 1) \times 0.574 + 1 = 1.00$$

Shoulder Width and Type (CMF_{2r})

CMF_{2r} can be calculated using HSM Equation 10-12 shown below. For this example, a 1-foot paved shoulder yields a CMF_{wra} of 1.4 (shoulder width, HSM Table 10-9 [HSM p. 10-25]) and CMF_{tra} of 1.0 (shoulder type, HSM Table 10-10 [HSM p. 10-26]). The percentage of related crashes is the same as that calculated for the lane width CMF:

$$CMF_{2r} = (CMF_{wra} \times CMF_{tra} - 1) \times p_{ra} + 1$$

$$CMF_{2r} = (1.4 \times 1.0 - 1) \times 0.574 + 1 = 1.23$$

Horizontal Curve (CMF_{3r})

For this example, the length of curve is 0.8 mile with a radius of curvature of 2,650 feet and no spiral transitions. Calculate the CMF calculation using HSM Equation 10-13 (HSM p. 10-27):

$$CMF_{3r} = \frac{1.55 \times L_c + \frac{80.2}{R} - 0.012 \times S}{1.55 \times L_c}$$

$$CMF_{3r} = \frac{1.55 \times 0.78 + \frac{80.2}{2650} - 0.012 \times 0}{1.55 \times 0.78} = 1.03$$

Superelevation (CMF_{4r})

In this example, the superelevation variance is assumed to be 0.02 foot/foot. Calculate the superelevation using HSM Equation 10-16 (HSM p. 10-28):

$$CMF_{4r\ SV \geq 0.02} = 1.06 + 3 \times (SV - 0.02)$$

$$CMF_{4r\ SV \geq 0.02} = 1.06 + 3 \times (0.02 - 0.02) = 1.06$$

Grades (CMF_{5r})

A 2-percent grade section falls under the level grade category in HSM Table 10-11 (HSM p. 10-28), resulting in a CMF of 1.00.

Driveway Density (CMF_{6r})

Driveway density of less than five driveways per mile leads to a CMF_{6R} of 1.00. Otherwise, the CMF is calculated using HSM Equation 10-17 (HSM p. 10-29):

$$CMF_{6r} = \frac{0.322 + DD \times [0.05 - 0.005 \times ln(AADT)]}{0.322 + 5 \times [0.05 - 0.005 \times ln(AADT)]}$$

$$CMF_{6r\ \leq 5} = 1.00$$

Centerline Rumble Strips (CMF_{7r})

The segment example does not include centerline rumble strips; therefore, a CMF of 1.00 is applied. See HSM p. 10-29 for additional details.

Passing Lanes (CMF_{8r})

Passing lanes are not available in the example; therefore, a CMF of 1.00 is appropriate. See HSM p. 10-29 for additional details.

Two-way, Left-turn Lane (CMF_{9r})

Two-way, left-turn lanes are not present; therefore, a CMF of 1.00 is applied for this example. See HSM p. 10-29 for additional details.

Roadside Design (CMF_{10r})

From the data input of this example, a roadside hazard rating of 5 applies to the segment. Using HSM Equation 10-20 (HSM p. 10-30), the CMF is:

$$CMF_{10r} = \frac{e^{-0.6869 + 0.0668 \times RHR}}{e^{-0.4865}}$$

$$CMF_{10r} = \frac{e^{-0.6869 + 0.0668 \times 5}}{e^{-0.4865}} = 1.14$$

Lighting (CMF_{11r})

Lighting is not present along the example segment; therefore, a CMF of 1.00 is applied. See HSM p. 10-30 for additional details.

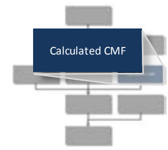
Automated speed enforcement (CMF_{12r})
The example roadway segment does not have automated speed enforcement available; therefore, a CMF of 1.00 is applied. See HSM p. 10-30 for additional details.

The combined CMF is then calculated by multiplying all segment CMFs:

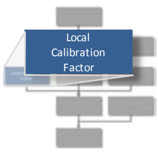
$$CMF_{combined} = CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{11r} \times CMF_{12r}$$

$$CMF_{combined} = 1.00 \times 1.23 \times 1.03 \times 1.06 \times 1.00 \times 1.00 \times 1.00 \times 1.00 \times 1.00 \times 1.14 \times 1.00 \times 1.00$$

$$CMF_{combined} = 1.527$$



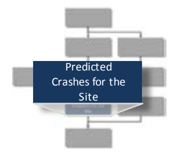
Apply Calibration Factor



Multiply the predicted average crash frequency and CMFs results obtained in previous steps by the appropriate calibration factor. For this example, the calibration factor has been assumed to be 1.30.

Obtain the Predicted Crash Frequency for the Site

The predicted average crash frequency is calculated using HSM Equation 10-2 (HSM p. 10-3), combining results from previous steps:



$$N_{predicted\ rs} = N_{spf\ rs} \times C_r \times (CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{12r})$$

$$N_{predicted\ rs} = 1.88 \times 1.30 \times (1.00 \times 1.23 \times \ldots \times 1.00) = 3.72\ crashes/year$$

Multiyear Analysis

Since 5 years of data are available, all the preceding steps need to be repeated four more times. In this example, a growth rate of 2 percent is assumed. Table 40 summarizes the calculations for the study period.

TABLE 40
Example Problem 2 – Roadway Segment 2 Multiyear Analysis Results

Roadway Segment 2	Year				
	2008	2009	2010	2011	2012
AADT	8,315	8,481	8,651	8,824	9,000
Crashes/year	29	45	48	38	40
N_{spf}	1.73	1.77	1.80	1.84	1.88
CMF_{1r}	1.00	1.00	1.00	1.00	1.00
CMF_{2r}	1.23	1.23	1.23	1.23	1.23
CMF_{3r}	1.03	1.03	1.03	1.03	1.03
CMF_{4r}	1.06	1.06	1.06	1.06	1.06
CMF_{5r}	1.00	1.00	1.00	1.00	1.00
CMF_{6r}	1.00	1.00	1.00	1.00	1.00
CMF_{7r}	1.00	1.00	1.00	1.00	1.00
CMF_{8r}	1.00	1.00	1.00	1.00	1.00
CMF_{9r}	1.00	1.00	1.00	1.00	1.00
CMF_{10r}	1.14	1.14	1.14	1.14	1.14
CMF_{11r}	1.00	1.00	1.00	1.00	1.00
CMF_{12r}	1.00	1.00	1.00	1.00	1.00

TABLE 40
Example Problem 2 – Roadway Segment 2 Multiyear Analysis Results

Roadway Segment 2	Year				
	2008	2009	2010	2011	2012
CMF_{comb}	1.527	1.527	1.527	1.527	1.527
C_i	1.30	1.30	1.30	1.30	1.30
$N_{predicted\ seg}$	3.44	3.51	3.58	3.65	3.72

Notes:
CMF_{comb} = combined CMF
N_{spf} = predicted average crash frequency estimated for base conditions
$N_{predicted\ seg}$ = predicted average crash frequency for the roadway segment

The average predicted crash frequency for Roadway Segment 2 is obtained through the arithmetic average of the annual predicted crash frequencies ($N_{predicted\ seg}$). The average for this example is 3.58 crashes per year.

Corridor Analysis (Intersections and Roadway Segments)
Analysis results for intersections and roadway segments can be combined into a corridor analysis. This approach combines the predicted crash frequency of the multiple locations to calculate the corridor predicted average crash frequency. This is done by adding the predicted average crash frequency of all roadway segments and intersections, as shown in Table 41.

TABLE 41
Example Problem 2 – Corridor Predicted Average Crash Frequency

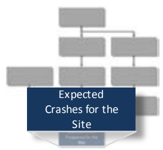
Site Type	Predicted Average Crash Frequency (crashes/year)		
	$N_{predicted}$ (Total)	$N_{predicted}$ (Fatal-and-Injury)	$N_{predicted}$ (PDO)
Roadway Segments			
Roadway Segment 1	4.94	1.59	3.36
Roadway Segment 2	3.58	1.15	2.43
Roadway Segment 3	8.24	2.64	5.59
Intersections			
Intersection 1	3.57	1.48	2.09
Intersection 2	3.91	1.62	2.29
Intersection 3	2.65	1.10	1.55
Combined (sum of column)	26.89	9.58	17.31

HSM Table 10-3 (HSM p. 10-17) provides default proportions for crash severity level on rural two-lane two-way roadway segments. This is used to separate the crash frequencies into *fatal-and-injury* and *PDO* crashes. Fatal-and-injury and PDO default proportions are 32.1 percent and 67.9 percent, respectively. Results from application of severity proportions are included in Table 41. These proportions can be updated using local crash data (refer to HSM Part C Appendix A for details).

Empirical Bayes Adjustment Method

The next step in the process is to update predictions based on the observed/reported crashes. A total of 62 roadway segment crashes and 11 intersection crashes occur each year. Using the predictive models, the total predictive average crash frequencies for roadway segments and intersections are 16.76 crashes and 10.13 crashes per year, respectively.

Empirical Bayes Adjustment Method



The predicted average crash frequency is then adjusted using the EB method by applying the following steps.

In this example, crashes can be assigned accurately between intersections and roadway segments; therefore, the site EB method is applicable. Refer to HSM Sections A.2.4 and A.2.5 (HSM p. A-19 and A-20) for additional details on the different EB methods.

The expected number of crashes for either roadway segments or intersections is calculated using HSM Equation A-4 (HSM p. A-19):

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed}$$

To complete this calculation, weighting adjustment factors are needed for the sample roadway segment and intersection. Calculate using the previous crash predictions, with HSM Equation A-5 (HSM p. A-19):

$$w = \frac{1}{1 + k \times \sum_{\substack{all \\ study \\ years}} N_{predicted}}$$

For this calculation, the overdispersion parameter (k) from each of the applied SPFs is needed. The overdispersion parameter associated with Roadway Segment 2 is 0.303. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. On a per-mile basis, the overdispersion parameter is found by using HSM Equation 10-7 (HSM p. 10-16):

$$k_{seg2} = \frac{0.236}{L}$$

$$k_{seg2} = \frac{0.236}{0.78} = 0.303$$

The overdispersion parameter associated with the three-leg, stop-controlled intersection is 0.54:

$$k_{int\ 3ST} = 0.540$$

Using these overdispersion parameters, the weighting adjustment factors are found to be 0.156 for Roadway Segment 2 and 0.123 for Intersection 3:

$$w_{seg2} = \frac{1}{1 + 0.303 \times (3.44 + 3.51 + 3.58 + 3.65 + 3.72)}$$

$$w_{seg2} = 0.156$$

$$w_{int3} = \frac{1}{1 + 0.123 \times (2.52 + 2.58 + 2.65 + 2.71 + 2.78)}$$

$$w_{int3} = 0.123$$

For this example, there were an average of 40 observed/reported crashes per year on Roadway Segment 2 and an average of 2 observed/reported crashes per year at Intersection 3. The expected number of crashes for roadway segments and intersections is then calculated as follows:

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed}$$

$$N_{expected\ seg2} = 0.156 \times 3.58 + (1 - 0.156) \times 40$$

$$N_{expected\ seg2} = 34.3\ crashes/year$$

$$N_{expected\ int3} = 0.123 \times 2.65 + (1 - 0.123) \times 2$$

$$N_{expected\ int3} = 2.1\ crashes/year$$

Similar analyses are performed for all roadway segments and intersections. Results of the analysis can be found in the sample spreadsheets provided with the *Highway Safety Manual User Guide*.

The total expected average crash frequency for the corridor is the sum of the expected crashes along the roadway segments and intersections. Calculate this sum using HSM Equation 10-4 (HSM p. 10-10):

$$N_{total} = \sum_{\substack{all\ roadway \\ segments}} N_{rs} + \sum_{\substack{all \\ intersections}} N_{int}$$

$$N_{total} = (9.99 + 34.32 + 10.54) + (3.96 + 4.91 + 2.08) = 65.79 \frac{crashes}{year}$$

Table 42 presents a summary of the predictive method calculations. Columns 2 through 4 contain the predicted average crash frequency for total, fatal-and-injury, and PDO crashes. The fifth column contains the observed/reported number of crashes per year. Columns 6 and 7 contain the overdispersion parameter and weighted adjustment to be used to obtain the expected average crash frequency (last column).

HSM Table 10-3 (HSM p. 10-17) provides default proportions for crash severity level on rural two-lane two-way roadway segments. This can also be used to separate the expected average crash frequencies into fatal-and-injury and PDO crashes. The fatal-and-injury and PDO default proportions are 32.1 percent and 67.9 percent, respectively.

TABLE 42
Example Problem 2 – Predicted and Expected Crash Frequency Calculations Summary (2008 to 2012)

Site Type	Predicted Average Crash Frequency (crashes/year)			Observed/ Reported Crashes ($N_{observed}$) (crashes/ year)	Over-dispersion Parameter (k)	Weighted Adjustment (w) (Equation A-5 from HSM Part C, Appendix A)	Expected Average Crash Frequency ($N_{expected}$) (Equation A-4 from HSM Part C, Appendix A)
	$N_{predicted}$ (Total)	$N_{predicted}$ (Fatal-&-Injury)	$N_{predicted}$ (PDO)				
Roadway Segments							
Roadway Segment 1	4.94	1.59	3.36	11	0.202	0.167	9.99
Year 1	4.75	1.52	3.22	10	0.202		
Year 2	4.84	1.55	3.29	12	0.202		
Year 3	4.94	1.59	3.35	14	0.202		
Year 4	5.04	1.62	3.42	8	0.202		
Year 5	5.14	1.65	3.49	11	0.202		

TABLE 42
Example Problem 2 – Predicted and Expected Crash Frequency Calculations Summary (2008 to 2012)

Site Type	Predicted Average Crash Frequency (crashes/year)			Observed/ Reported Crashes ($N_{observed}$) (crashes/ year)	Over-dispersion Parameter (k)	Weighted Adjustment (w) (Equation A-5 from HSM Part C, Appendix A)	Expected Average Crash Frequency ($N_{expected}$) (Equation A-4 from HSM Part C, Appendix A)
	$N_{predicted}$ (Total)	$N_{predicted}$ (Fatal-&-Injury)	$N_{predicted}$ (PDO)				
Roadway Segment 2	3.58	1.15	2.43	40	0.303	0.156	34.32
Year 1	3.44	1.10	2.34	29	0.303		
Year 2	3.51	1.13	2.38	45	0.303		
Year 3	3.58	1.15	2.43	48	0.303		
Year 4	3.65	1.17	2.48	38	0.303		
Year 5	3.72	1.20	2.53	40	0.303		
Roadway Segment 3	8.24	2.64	5.59	11	0.121	0.167	10.54
Year 1	7.91	2.54	5.37	11	0.121		
Year 2	8.07	2.59	5.48	15	0.121		
Year 3	8.23	2.64	5.59	12	0.121		
Year 4	8.40	2.70	5.70	10	0.121		
Year 5	8.57	2.75	5.82	7	0.121		
Intersections							
Intersection 1	3.57	1.48	2.09	4	0.540	0.094	3.96
Year 1	3.40	1.41	1.99	2	0.540		
Year 2	3.48	1.45	2.04	6	0.540		
Year 3	3.57	1.48	2.09	5	0.540		
Year 4	3.66	1.52	2.14	4	0.540		
Year 5	3.76	1.56	2.20	3	0.540		
Intersection 2	3.91	1.62	2.29	5	0.540	0.087	4.91
Year 1	3.71	1.54	2.17	8	0.540		
Year 2	3.81	1.58	2.23	3	0.540		
Year 3	3.91	1.62	2.28	4	0.540		
Year 4	4.01	1.66	2.34	6	0.540		
Year 5	4.11	1.71	2.40	4	0.540		
Intersection 3	2.65	1.10	1.55	2	0.540	0.123	2.08
Year 1	2.52	1.04	1.47	1	0.540		
Year 2	2.58	1.07	1.51	0	0.540		
Year 3	2.65	1.10	1.55	4	0.540		
Year 4	2.71	1.13	1.59	3	0.540		
Year 5	2.78	1.16	1.63	2	0.540		
Total	26.89	9.58	17.31	73	–	–	65.79

Alternatives Analysis

The previous section demonstrated the application of the predictive method for rural two-lane, two-way roadway segments and intersections under existing conditions. The predictive method can also be applied to alternatives analysis. This process is more detailed and specific about the impacts of implementation of project improvements.

The agency develops potential alternatives and compares performance across the alternatives. The two-lane, two-way rural roads predictive method can be applied to compare alternatives, as described in the following paragraphs. Calculations and formulas are the same used in the previous sections, and the results are summarized into tables.

Tables 43 and 44 contain the input data for the current conditions, along with two alternatives to improve the corridor existing safety performance. For simplicity, only geometric elements that are being improved or upgraded are listed in the tables.

Alternative 1, as compared to the No Build scenario, consists of:

- Shoulder widening from 1- to 6-foot shoulders
- Adding an uncontrolled left-turn lane to each intersection

For demonstration purposes, it is assumed the AADT remains the same and the road does not attract any additional traffic.

Alternative 2, in addition to those improvements listed in Alternative 1, consists of:

- Improve roadside hazard rating to Level 3 by removing vegetation along the road
- Install lighting along the roadway segment and at intersections
- Implement auto speed enforcement

Similarly, it is assumed the AADT remains the same, and the road does not attract any additional traffic.

TABLE 43
Example Problem 2 – Roadway Segment Alternatives Input Data

	Input Data by Alternative		
Roadway Segment 1 Characteristics	**No Build**	**Alternative 1**	**Alternative 2**
Shoulder width (feet)	1	6	6
Roadside hazard rating	5	5	3
Segment lighting	Not present	Not present	Present
Auto speed enforcement	Not present	Not present	Present
	Input Data by Alternative		
Roadway Segment 2 Characteristics	**No Build**	**Alternative 1**	**Alternative 2**
Shoulder width (feet)	1	6	6
Roadside hazard rating	5	5	3
Segment lighting	Not present	Not present	Present
Auto speed enforcement	Not present	Not present	Present

TABLE 43
Example Problem 2 – Roadway Segment Alternatives Input Data

Roadway Segment 3 Characteristics	Input Data by Alternative		
	No Build	Alternative 1	Alternative 2
Shoulder width (feet)	1	6	6
Roadside hazard rating	5	5	3
Segment lighting	Not present	Not present	Present
Auto speed enforcement	Not present	Not present	Present

TABLE 44
Example Problem 2 – Intersection Alternatives Input Data

Intersection 1 Characteristics	Input Data by Alternative		
	No Build	Alternative 1	Alternative 2
Number of signalized or uncontrolled approaches with a left-turn lane	0	1	1
Intersection lighting	Not present	Not present	Present
Intersection 2 Characteristics	**Input Data by Alternative**		
	No Build	**Alternative 1**	**Alternative 2**
Number of signalized or uncontrolled approaches with a left-turn lane	0	1	1
Intersection lighting	Not present	Not present	Present
Intersection 3 Characteristics	**Input Data by Alternative**		
	No Build	**Alternative 1**	**Alternative 2**
Number of signalized or uncontrolled approaches with a left-turn lane	0	1	1
Intersection lighting	Not present	Not present	Present

The effect of the multiple treatments (such as widening shoulders, lighting the roadway segments and intersections, adding left-turn lanes) is reflected in the decrease of predicted average number of crashes.

All these different adjustments are taken into account through the CMFs, which are used to adjust the SPF estimate of predicted average crash frequency for the effect of these different individual geometric design and traffic control features. The CMF for the SPF base condition of each geometric design or traffic control feature has a value of 1.00.

Calculations for the No Build scenario are the same as the first part of the example. Table 45 summarizes the results for the No Build scenario and Alternatives 1 and 2. Total predicted, observed, and expected average crash frequencies are in bolded text. As shown in the table, the expected number of crashes under existing conditions is higher (65.79 crashes per year) than for Alternatives 1 and 2. As anticipated, the expected number of crashes for Alternative 2 is lower than Alternative 1. However, an economic evaluation should be conducted to determine which alternative is more cost-effective. Detailed calculations are provided in the sample spreadsheets provided with the *Highway Safety Manual User Guide*.

TABLE 45
Example Problem 2 – Alternatives Analysis Results Summary

Alternative	Site Type	$N_{predicted}$	$N_{observed}$	Overdispersion Parameter (k)	Weighted Adjustment (w)	$N_{expected}$
No Build	Roadway Segment 1	4.94	11	0.202	0.167	9.99
	Roadway Segment 2	3.58	40	0.303	0.156	34.32
	Roadway Segment 3	8.24	11	0.121	0.167	10.54
	Intersection 1	3.57	4	0.540	0.094	3.96
	Intersection 2	3.91	5	0.540	0.087	4.91
	Intersection 3	2.65	2	0.540	0.123	2.08
	Total	**26.89**	**73**	**–**	**–**	**65.79**
Alternative 1	Roadway Segment 1	4.02	11	0.202	0.198	9.62
	Roadway Segment 2	2.91	40	0.303	0.185	33.14
	Roadway Segment 3	6.70	11	0.121	0.198	10.15
	Intersection 1	2.00	4	0.540	0.156	3.69
	Intersection 2	2.19	5	0.540	0.145	4.59
	Intersection 3	1.48	2	0.540	0.200	1.90
	Total	**19.30**	**73**	**–**	**–**	**63.08**
Alternative 2	Roadway Segment 1	3.01	11	0.202	0.248	9.02
	Roadway Segment 2	2.18	40	0.303	0.232	31.21
	Roadway Segment 3	5.02	11	0.121	0.248	9.52
	Intersection 1	1.80	4	0.540	0.170	3.63
	Intersection 2	1.97	5	0.540	0.158	4.52
	Intersection 3	1.34	2	0.540	0.217	1.86
	Total	**15.33**	**73**	**–**	**–**	**59.76**

Discussion of results of this section is provided after the rural multilane highway alternative analysis.

3.2.4 Part 2 – Rural Multilane Highways

The agency also decided to analyze the safety performance of converting the rural two-lane, two-way roads into a four-lane divided roadway. The analysis will be performed for existing and future (2030) conditions. For the purpose of understanding the methodology, the example first shows how to calculate the predictive average crash frequency for a rural multilane stop-controlled intersection and rural multilane divided roadway under existing conditions, and how to combine intersections and roadway segments as part of a corridor study. Next, the predicted average crash frequency for future conditions will be compared with the 2030 rural two-lane road predicted crash frequency. Finally, a discussion of the results will be provided.

Data Requirements for Part 2

Figure 24 shows the different facility types included in this example. Since the rural multilane predictive method does not include a CMF for curves, there is no need to break the corridor into multiple segments. Crash data are available for years 2008 to 2012. The roadway segment and intersections characteristics are summarized in Tables 46 and 47.

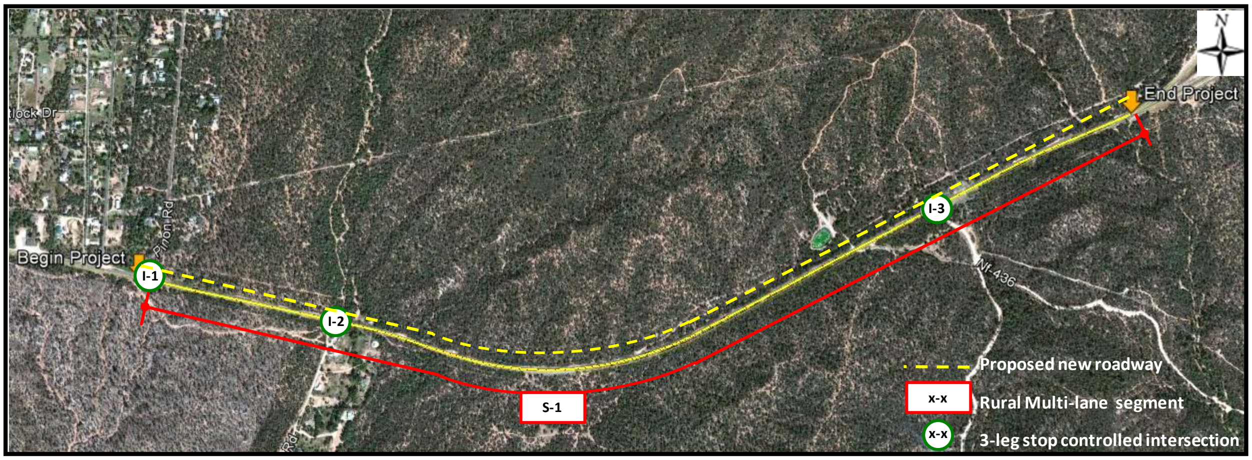


Figure 24: Example Problem 1 – Sample Rural Multilane Highway

Intersection Data

Table 46 summarizes the input data required to apply the predictive method for all intersections.

TABLE 46
Example Problem 2 – Intersections Input Data

Characteristics	Input Data		
	Intersection 1	Intersection 2	Intersection 3
Intersection type	3ST	3ST	3ST
Traffic flow major road (vpd)	9,000	9,000	9,000
Traffic flow minor road (vpd)	2,500	3,000	1,200
Intersection skew angle	0	0	15
Number of signalized or uncontrolled approaches with a left-turn lane	1	0	0
Number of signalized or uncontrolled approaches with a right-turn lane	1	0	0
Intersection lighting	Not present	Not Present	Not Present
Calibration factor (C_i)	1.20	1.20	1.20
Observed/reported fatal-and-injury crashes (crashes/year)	2	2	1
Observed/reported PDO crashes (crashes/year)	2	3	1

Roadway Segment Data

Table 47 summarizes the multilane rural divided roadway segment input data.

TABLE 47
Example Problem 2 – Roadway Segment 1 Input Data

Characteristics	Input Data Roadway Segment 1
Roadway type	Divided
Segment length (miles)	3.90
Traffic volume (vpd)	9,000
Lane width (feet)	12
Shoulder width (feet)	8
Shoulder type	Paved
Median width (feet)	30
Sideslopes	---
Segment lighting	Not present
Auto speed enforcement	Not present
Calibration factor (C_r)	1.08
Observed/reported fatal-and-injury crashes (crashes/year)	19
Observed/reported PDO crashes (crashes/year)	43

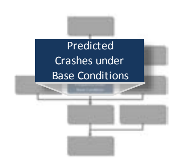
Analysis

The rural multilane highways predictive method for intersections and roadway segments under existing conditions (year 2012) is applied in the following sections. For illustrative purposes, detailed calculations are included only for Intersection 1 and Segment 1.

Intersections

The first part of the predictive method is focused on defining the limits, facility type, and study period as well as obtaining and preparing input datasets required to apply the predictive models. Detailed information related to data collection can be found in HSM Section 11.4. The data summary for this example is provided in Table 46.

Select and Apply SPFs



The intersection SPFs in HSM Chapter 11 estimate the total predicted average crash frequency for intersection-related crashes within the intersection limits and on the intersection legs. SPFs are provided for different intersection types and severity levels.

To determine the predictive average crash frequency of the sample intersection, select and apply the appropriate SPF for the facility type and traffic control features. The total and fatal-and-injury predicted crash frequencies for a stop-controlled intersection (Intersection 1) can be calculated using HSM Equation 11-11 with coefficients from Table 11-7 (HSM p. 11-20 and 11-21):

$$N_{spf\ int} = e^{a+b\times ln(AADT_{major})+c\times ln(AADT_{minor})}$$

$$N_{spf\ 3ST\ total} = e^{-12.526+1.204\times ln(9{,}000)+0.236\times ln(2{,}500)}$$

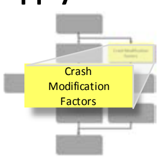
$$N_{spf\ 3ST\ total} = 1.327\frac{crashes}{year}$$

$$N_{spf\ 3ST\ (FI)} = e^{-12.664+1.107\times ln(9{,}000)+0.272\times ln(2{,}500)}$$

$$N_{spf\ 3ST\ (FI)} = 0.633\frac{crashes}{year}$$

A separate set of fatal-and-injury SPFs are also available for agencies that do not wish to consider severity level C (possible injury) on the KABCO scale (see Appendix B of this guide).

Apply HSM Part C Crash Modification Factors



Calculate the appropriate CMFs to adjust the predicted crash frequency for base conditions to the site-specific geometry and traffic features. For this example, all the intersection CMFs are equal to 1.00.

Intersection Skew Angle (CMF_{1i})

CMF_{1i} can be calculated using HSM Equations 11-18 (total) and 11-19 (fatal and injury) for a 3ST intersection (HSM p. 11-33). Intersection 1 is not skewed; therefore, the CMF is 1.00:

$$CMF_{1i\ 3ST\ total} = \frac{0.016\times skew}{0.98+0.016\times skew} + 1 = 1.00$$

$$CMF_{1i\ 3ST\ (FI)} = \frac{0.017\times skew}{0.52+0.017\times skew} + 1 = 1.00$$

Intersection Left-Turn Lanes (CMF_{2i})

One left-turn lanes is present at the example intersection; therefore, CMF of 0.56 for total crashes, and 0.45 for fatal-and-injury crashes are applied. HSM Table 11-22 (HSM p. 11-34) presents CMFs for the presence of left-turn lanes for total and fatal-and-injury crashes.

Intersection Right-Turn Lanes (CMF_{3i})

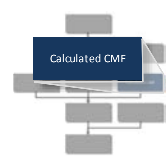
Similarly, a right-turn lane is present at the example intersection; therefore, a CMF of 0.86 for total crashes, and 0.77 for fatal-and-injury crashes are applied. HSM Table 11-23 (HSM p. 11-35) presents CMFs for the presence of right-turn lanes for total and fatal-and-injury crashes.

Intersection Lighting (CMF_{4i})

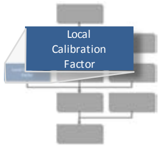
Lighting is unavailable in this example; therefore, a CMF of 1.00 is appropriate. HSM Equation 11-22 (HSM p. 11-35) is used to estimate the CMF for lighting.

The combined CMF is then calculated by multiplying all the intersection CMFs:

$$CMF_{combined} = CMF_{1i} \times CMF_{2i} \times CMF_{3i} \times CMF_{4i}$$



Apply Calibration Factor



The next step is to multiply the results obtained above by the appropriate calibration factor. For this example, the calibration factor has been assumed to be 1.20.

Obtain the Predicted Crash Frequency for the Site

The predicted average crash frequency for Intersection 1 is calculated using HSM Equation 11-4 (HSM p. 11-4), combining results from previous steps:

$$N_{predicted\ int} = N_{spf\ int} \times C_i \times (CMF_{1i} \times CMF_{2i} \times CMF_{3i} \times CMF_{4i})$$

$$N_{predicted\ int\ 1\ total} = 1.327 \times 1.20 \times (1.0 \times 0.56 \times 0.86 \times 1.0)$$

$$N_{predicted\ int\ 1\ total} = 0.767 \frac{crashes}{year}$$

$$N_{predicted\ int\ 1\ (FI)} = 0.633 \times 1.20 \times (1.0 \times 0.45 \times 0.77 \times 1.0)$$

$$N_{predicted\ int\ 1\ (FI)} = 0.263 \frac{crashes}{year}$$

$$N_{predicted\ int\ 1\ (PDO)} = 0.767 - 0.263 = 0.503 \frac{crashes}{year}$$

Multiyear Analysis

Since 5 years of data are available, all the previous steps need to be repeated four more times. In this example, a growth rate of 2 percent is assumed. Table 48 summarizes the calculations for the study period. Only calculations for total crashes are shown in the table. Calculations for fatal-and-injury crashes and other detailed calculations are provided in the sample *Highway Safety Manual User Guide* spreadsheets.

TABLE 48
Example Problem 2 – Intersection 1 Multiyear Analysis Results

Intersection 1	Year				
	2008	2009	2010	2011	2012
$AADT_{major}$	8,315	8,481	8,651	8,824	9,000
$AADT_{minor}$	2,310	2,356	2,403	2,451	2,500
Crashes/year	2	6	5	4	3
$N_{spf\ 3ST}$	1.184	1.218	1.253	1.290	1.327
$CMF_{1i\ 3ST}$	1.00	1.00	1.00	1.00	1.00
$CMF_{2i\ 3ST}$	0.56	0.56	0.56	0.56	0.56
$CMF_{3i\ 3ST}$	0.86	0.86	0.86	0.86	0.86
$CMF_{4i\ 3ST}$	1.00	1.00	1.00	1.00	1.00
CMF_{comb}	0.48	0.48	0.48	0.48	0.48
C_i	1.20	1.20	1.20	1.20	1.20

TABLE 48
Example Problem 2 – Intersection 1 Multiyear Analysis Results

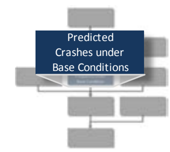
Intersection 1	Year				
	2008	2009	2010	2011	2012
$N_{predicted\ int}$	0.68	0.70	0.72	0.75	0.77

The average predicted crash frequency for Intersection 1 is obtained by the arithmetic average of the annual predicted crash frequencies ($N_{predicted\ int}$). For this example, this value is 0.725 crash per year.

Roadway Segments

Roadway segment data required to apply the predictive method are summarized in Table 47. For this example, the analysis corridor consists of only one four-lane divided segment. Information on different recommendations related to data collection is presented in HSM Section 11.4.

Select and Apply SPFs



Separate SPFs are available for undivided (HSM Equation 11-7 and Table 11-3 [HSM p. 11-15]) and divided (HSM Equation 11-9 and Table 11-5 [HSM p. 11-18]) rural multilane highways for total and fatal-and-injury severity levels:

$$N_{spf\ seg} = e^{a+b\times ln(AADT)+ln(L)}$$

$$N_{spf\ divided\ total} = e^{-9.025+1.049\times ln(9{,}000)+ln(3.9)}$$

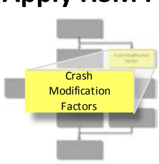
$$N_{spf\ divided\ total} = 6.60\frac{crashes}{year}$$

$$N_{spf\ divided\ (FI)} = e^{-8.837+0.958\times ln(9{,}000)+ln(3.9)}$$

$$N_{spf\ divided\ (FI)} = 3.48\frac{crashes}{year}$$

As with the intersection SPFs, a separate set of fatal-and-injury SPFs are available for agencies that do not wish to consider Severity Level C (possible injury) on the KABCO scale (see Appendix B of this guide).

Apply HSM Part C Crash Modification Factors



Calculate the applicable CMFs to adjust the estimated crash frequency for base conditions to the site-specific geometry and traffic features.

Lane Width (CMF_{1r})

For a 12-foot lane width, the effect of lane width on related crashes (such as single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes) is 1.00, as shown in HSM Table 11-16 (HSM p. 11-30).

Shoulder Width and Type (CMF_{2r})

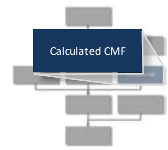
CMF_{2r} can be calculated using HSM Table 11-17 (HSM p. 11-31). The SPF base condition for the right shoulder is 8 feet. NOTE: The CMFs provided in HSM Table 11-17 only apply to paved shoulders. For an 8-foot right-shoulder width, the applicable CMF is 1.00.

Median Width (CMF_{3rd})
The base condition assigned to the median width CMF is 30 feet, assuming no traffic barrier. These base conditions match the characteristics of the example roadway segment, resulting in a CMF of 1.00. HSM Table 11-18 (HSM p. 11-31) contains CMFs for different median widths on divided roadway segments.

Lighting (CMF_{4r})
Lighting is not present along the example roadway segment; therefore, a CMF of 1.00 is applied. See HSM p. 11-31 for additional details.

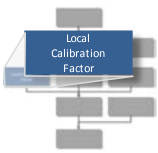
Automated Speed Enforcement (CMF_{5r})
The example segment does not have automated speed enforcement available; therefore, a CMF of 1.00 is applied. See HSM Page 11-32 for additional details.

The combined CMF is then calculated by multiplying all the intersection CMFs:



$$CMF_{combined} = CMF_{1i} \times CMF_{2i} \times CMF_{3i} \times CMF_{4i}$$

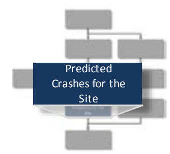
Apply Calibration Factor



Multiply the results obtained in the two previous steps by the appropriate calibration factor. For this example, the calibration factor is assumed to be 1.00.

Obtain the Predicted Crash Frequency for the Site

Lastly, the predicted average crash frequency is calculated using HSM Equation 11-3 (HSM p. 11-4), combining results from the preceding steps:



$$N_{predicted\ seg} = N_{spf\ seg} \times C_r \times (CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{5r})$$

$$N_{predicted\ seg\ total} = 6.60 \times 1.08 \times (1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0)$$

$$N_{predicted\ seg\ total} = 7.13 \frac{crashes}{year}$$

$$N_{predicted\ seg\ (FI)} = 3.48 \times 1.08 \times (1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0)$$

$$N_{predicted\ seg\ (FI)} = 3.76 \frac{crashes}{year}$$

$$N_{predicted\ seg\ (PDO)} = 7.13 - 3.76 = 3.37 \frac{crashes}{year}$$

Multiyear Analysis
Since 5 years of data are available, all the steps above must be repeated four more times. In this example, a growth rate of 2 percent is assumed. Table 49 summarizes Roadway Segment 1 multiyear calculations.

The average predicted crash frequency for Roadway Segment 1 is obtained through the arithmetic average of the annual predicted crash frequencies ($N_{predicted\ seg}$). The average for this example is 6.84 crashes per year.

TABLE 49
Example Problem 2 – Roadway Segment 1 Multiyear Analysis Results

Roadway Segment 1	Year				
	2008	2009	2010	2011	2012
AADT	8,315	8,481	8,651	8,824	9,000
Crashes/year	50	72	74	56	58
N_{spf}	6.07	6.20	6.33	6.47	6.60
CMF_{1ru}	1.00	1.00	1.00	1.00	1.00
CMF_{2ru}	1.00	1.00	1.00	1.00	1.00
CMF_{3ru}	1.00	1.00	1.00	1.00	1.00
CMF_{4ru}	1.00	1.00	1.00	1.00	1.00
CMF_{5ru}	1.00	1.00	1.00	1.00	1.00
CMF_{comb}	1.00	1.00	1.00	1.00	1.00
C_r	1.08	1.08	1.08	1.08	1.08
$N_{predicted\ seg}$	6.56	6.70	6.84	6.98	7.13

Corridor Analysis (Intersections and Roadway Segments)

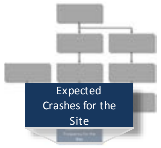
Analysis results for intersections and roadway segments can be combined into a corridor analysis. This approach combines the predicted crash frequency of the multiple locations to come up with corridor predicted average crash frequency. Table 50 summarizes the predicted crash frequency of all roadway segments and intersections, and provides the corridor results.

TABLE 50
Example Problem 2 – Corridor Predicted Average Crash Frequency

Site Type	Predicted Average Crash Frequency (crashes/year)		
	$N_{predicted}$ (Total)	$N_{predicted}$ (Fatal-&-Injury)	$N_{predicted}$ (PDO)
Roadway Segments Divided			
Roadway Segment 1	6.84	3.62	3.22
Intersections			
Intersection 1	0.72	0.25	0.48
Intersection 2	1.57	0.76	0.81
Intersection 3	1.51	0.78	0.73
Combined (sum of column)	10.65	5.41	5.24

HSM Tables 11-4 (HSM p. 11-17), 11-6 (HSM p. 11-20), and 11-9 (HSM p. 11-24) provide default proportions of crashes by collision type and crash severity level for rural multilane undivided, divided roadways, and intersections. These proportions can be applied to the predicted crash frequencies for selected collision types. These proportions can be updated using local crash data (refer to HSM Part C, Appendix A for details).

Empirical Bayes Adjustment Method



The next step in the process is to update predictions based on the observed/reported crashes.

First, it must be determined whether the EB method is applicable to this example. HSM Section A.2.1 (HSM p. A-16) provides guidance on how to determine the applicability of the EB method. Since this project upgrade involves the development of a new alignment for a substantial portion of the project length, the EB method is not applicable. The main reason is the historical observed/reported crash data may not be a good indicator of the crash experience that is likely to occur in the future after the implementation of a major change. If the conversion from two-lane to four-lane divided was done in a short section of the corridor (less than 2 miles) to allow more passing opportunities, then the expected number of crashes could have been calculated using the observed/reported number of crashes from the rural two-lane two-way facility.

Details on the application of the predictive method can be found in HSM Appendix A, Section A.2 (HSM p. A-15).

Alternatives Analysis

Part 1 of this example demonstrated the application of the predictive method for the rural two-lane, two-way roadway segment and intersections under existing conditions, and for alternative analysis. The second part of the problem was focused on the application of the predictive method for the rural multilane rural roadway segment and intersections under existing conditions. The next step in the process is to compare the different alternatives under consideration. Since the construction of the four-lane divided roadway is a future project, predicted average crash frequencies for year 2030 (opening year) will be calculated for the following scenarios: rural two-lane, two-way road No Build, Alternative 1, Alternative 2, and proposed rural multilane corridor.

According to the local metropolitan planning organization (MPO), the projected 2030 AADT for the SR corridor is 13,500 vehicles per hour. This translates to a growth factor of about 2.28 percent per year. Minor road AADTs are obtained by applying the growth factor to the existing AADTs. Table 51 summarizes the traffic volumes for the different facility types.

TABLE 51
Example Problem 2 – Year 2030 AADT for Rural Two-Lane and Rural Multilane Facilities

Facility	Roadway Segment 1	Roadway Segment 2	Roadway Segment 3
Rural two-lane roads	13,500	13,500	13,500
Rural multilane road	13,500		
	Intersection 1	**Intersection 2**	**Intersection 3**
Rural two-lane major road	13,500	13,500	13,500
Rural two-lane minor road	3,750	4,500	1,800
Rural multilane major road	13,500	13,500	13,500
Rural multilane minor road	3,750	4,500	1,800

Using the same input data (rural two-lane Tables 37, 38, 43, and 44, and rural multilane Tables 46 and 47), proposed safety countermeasures, and 2030 AADTs, the predictive average crash frequency is calculated for rural two-lane No Build, Alternative 1, and Alternative 2, and the four-lane divided road (Alternative 3). Table 52 includes the predicted crash frequencies for total, fatal-and-injury, and PDO crashes.

TABLE 52
Example Problem 2 – Future Conditions Alternative Analysis Summary (2030)

Alternative	Site Type	$N_{predicted\ Total}$	$N_{predicted\ FI}$	$N_{predicted\ PDO}$
Rural Two-lane: No Build	Segment 1	7.71	2.47	5.23
	Segment 2	5.58	1.79	3.79
	Segment 3	12.85	4.12	8.72
	Intersection 1	6.31	2.62	3.69
	Intersection 2	6.90	2.87	4.04
	Intersection 3	4.68	1.94	2.74
	Total	**44.04**	**15.82**	**28.22**
Rural Two-Lane: Alternative 1	Segment 1	6.27	2.01	4.26
	Segment 2	4.54	1.46	3.08
	Segment 3	10.45	3.35	7.10
	Intersection 1	3.54	1.47	2.07
	Intersection 2	3.87	1.60	2.26
	Intersection 3	2.62	1.09	1.53
	Total	**31.28**	**10.98**	**20.30**
Rural Two Lane: Alternative 2	Segment 1	4.70	1.51	3.19
	Segment 2	3.41	1.09	2.31
	Segment 3	7.84	2.52	5.32
	Intersection 1	3.19	1.32	1.86
	Intersection 2	3.48	1.45	2.04
	Intersection 3	2.36	0.98	1.38
	Total	**24.98**	**8.87**	**16.11**
Rural Multilane: Alternative 3	Segment 1	10.91	5.54	5.37
	Intersection 1	1.37	0.46	0.91
	Intersection 2	2.98	1.40	1.58
	Intersection 3	2.87	1.45	1.43
	Total	**18.14**	**8.84**	**9.29**

Results and Discussion

From Part 1 of the problem, it was concluded that the proposed countermeasures for rural two-lane Alternative 2 produced the lower predicted and expected crash frequency. However, conducting an economic evaluation was recommended to make a cost-effective decision.

The state DOT also considered modifying the rural two-lane corridor to a four-lane divided facility. The analysis was conducted for future conditions (design year 2030); however, since the EB method was not applicable, the comparison with the other alternatives was conducted using the predicted average crash frequency. (NOTE: If the conversion from two-lane to four-lane divided was done in a short section of the corridor [less than 2 miles] to allow more passing opportunities, then the expected number of crashes could have been calculated.)

The analysis results indicate that the four-lane divided alternative reduces the total crash frequency by 59 percent in comparison to the 2030 No Build scenario. Alternatives 1 and 2 would reduce the total crash frequency by 329 percent and 43 percent, respectively. Predicted crash frequencies for fatal-and-injury and PDO crashes are also provided. (NOTE: Fatal-and-injury and PDO crash frequencies for rural two-lane are calculated based on proportions, and for rural multilane are calculated using fatal-and-injury SPFs.) Alternative 2 and Alternative 3 have the lowest predicted fatal-and-injury crash frequencies.

Based on these results, the four-lane conversion would potentially provide the greatest reduction in crash frequency along the corridor, but an economic evaluation is required to better understand which alternative is the most cost-effective. Refer to HSM Chapter 7, Economic Appraisal, for methods to compare the benefits of potential crash countermeasures to crash costs.

Tools Available for HSM Part C Application

The Interactive Highway Safety Design Model (IHSDM) and spreadsheet tools are available to assist in the HSM Part C predictive method calculations. The HSM Part C spreadsheet tools can be downloaded from the HSM website under the Quick Links section (http://www.highwaysafetymanual.org).

In addition to analyzing safety performance, IHSDM has a design consistency module that may be helpful to users for planning or design.

3.2.5 Example Problem 3: Urban and Suburban Arterials

Introduction

The example facility is a 0.3-mile urban arterial with commercial development. The corridor has two 12-foot lanes in each direction, and a TWLTL that provides access to the driveways along the road. Most of the properties adjacent to the corridor have multiple direct access points. Parallel on-street parking is available along the corridor. The posted speed limit is 35 mph. The corridor is bounded by a four-leg signalized intersection on the north and a three-leg stop-controlled intersection on the south (Figure 25).

Figure 25: Sample Urban and Suburban Arterial

A high proportion of rear-end, angle, and sideswipe crashes have occurred at the facility in the past few years. In addition, a couple fatalities and a serious injury crash were reported. The city has decided to evaluate alternatives to mitigate the safety issues, to improve traffic operations, and make it more pedestrian friendly. Five years of crash data are available for this study (2008 to 2012). Analysis details are provided in the sections below.

Objectives

This example is focused on evaluating the crash reduction potential of various design alternatives for an urban arterial. Several improvements were considered as part of the project, including providing a physical median along the corridor in one section of the corridor, providing dedicated bus pullout areas, widening the sidewalk, and providing a median separation. This example demonstrates the quantitative safety analysis of the existing facility and two additional alternatives along the corridor.

The first part of the problem illustrates how to calculate the predictive average crash frequency for a signalized intersection (Intersection 1) and an urban roadway segment. The second part of the problem illustrates how to combine all intersections and roadway segments as part of a corridor study, and the analysis of the two different alternatives. The objective of each of the problems is to show how various HSM analysis tools can be applied to assist traffic analysts, engineers, planners, and decision-makers in making sound investment decisions. In some situations, this amount of analysis would not be necessary to make an informed decision, but the issues presented herein should always be considered to assure the final decision is consistent with safety performance objectives.

After reviewing this example, the user should be able to:

- Understand what input data are required and the assumptions that are commonly made regarding default values for HSM procedures
- Calculate the predicted and expected crash frequency of urban and suburban intersections and roadway segments using HSM
- Understand how to reasonably interpret the results from an HSM analysis, and how these results can be used to support a particular decision
- Understand the limitations of the HSM procedures and when it is appropriate to use other models or computational tools

Data Requirements

Intersection Data

Table 53 summarizes the input data required to apply the predictive method for urban and suburban arterials at Intersections 1 and 2.

TABLE 53
Example Problem 3 – Intersections Input Data

Characteristics	Input Data	
	Intersection 1	Intersection 2
Intersection type	4SG	3ST
Traffic flow major road (vpd)	23,000	23,000
Traffic flow minor road (vpd)	14,000	1,500
Intersection lighting	Not present	Not present
Calibration factor(C_i)	1.15	1.15
Data for Unsignalized Intersections Only		
Number of major-road approaches with left-turn lanes	0	0
Number of minor-road approaches with right-turn lanes	0	0
Data for Signalized Intersections Only		
Number of approaches with left-turn lanes	0	–

TABLE 53
Example Problem 3 – Intersections Input Data

Characteristics	Input Data	
	Intersection 1	Intersection 2
Number of approaches with right-turn lanes	0	–
Number of approaches with left-turn signal phasing	0	–
Type of left-turn phasing	Not applicable	–
Number of approaches with right-turn-on-red prohibited	Not present	–
Intersection red-light cameras	Not present	–
Sum of all pedestrian crossing volumes	400	–
Maximum number of lanes crossed by a pedestrian	5	–
Number of bus stops within 1,000 feet (300 meters) of the intersection	1	–
Schools within 1,000 feet (300 meters) of the intersection (present/not present)	Not present	–
Number of alcohol sales establishments within 1,000 feet (300 meters) of intersection	1	–

For this example, intersection crash data disaggregated by year and collision type are available. Table 54 shows the crash data details for Intersections 1 and 2.

TABLE 54
Example Problem 3 – Disaggregated Intersection Crash Data for the Study Period

Collision Type	Intersection 1						
	2008	2009	2010	2011	2012	Sum	Average
Multiple-Vehicle Nondriveway	3	6	4	7	4	24	4.8
Single-Vehicle	0	1	0	0	0	1	0.2
Total	3	7	4	7	4	25	5
Collision Type	**Intersection 2**						
	2008	**2009**	**2010**	**2011**	**2012**	**Sum**	**Average**
Multiple-Vehicle Nondriveway	2	6	5	3	4	20	4
Single-Vehicle	0	0	0	0	0	0	0
Total	2	6	5	3	4	20	4

Roadway Segment Data

TABLE 55
Example Problem 3 – Arterial Roadway Segment Input Data

Characteristics	Input Data Roadway Segment 1
Roadway type	5T
Segment length (miles)	0.3
Traffic volume (vpd)	23,000
Type of on-street parking	Parallel (commercial/industrial)
Proportion of curb length with on-street parking (0.5 x L_{pk}/L)	0.4
Median width (feet)	–
Segment lighting	Present
Auto speed enforcement	Not present
Major commercial driveways	2
Minor commercial driveways	8
Major industrial/institutional driveways	–
Minor industrial/institutional driveways	–
Major residential driveways	–
Minor residential driveways	2
Other driveways	–
Speed category	Greater than 30 mph
Roadside fixed object density	20
Offset to roadside fixed objects (feet)	10
Calibration factor (C_r)	1.1

Note:
L_{pk}/L = proportion of curb length with on-street parking

Similarly, roadway segment crash data disaggregated by year and collision type for the study period are shown in Table 56.

TABLE 56
Example Problem 3 – Disaggregated Roadway Segment Crash Data for the Study Period

Collision Type	Roadway Segment						
	2008	**2009**	**2010**	**2011**	**2012**	**Sum**	**Average**
Multiple-Vehicle Nondriveway	5	7	6	8	9	35	7
Single-Vehicle	0	2	1	1	1	5	1
Multiple-Vehicle Driveway-Related	6	5	3	4	2	20	4
Total	**11**	**14**	**10**	**13**	**12**	**60**	**12**

Analysis

The urban and suburban arterial safety analysis differs from the previous two predictive methods since pedestrian and bicycle collisions must be accounted for, with respect to intersections and roadway segments. Each collision type will be analyzed in detail.

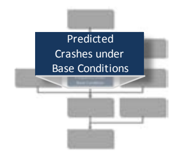
The first part of the analysis will be focused on understanding how to apply the predictive method to one signalized intersection (Intersection 1) and one roadway segment (Roadway Segment 1) independently using 2012 data. Next, these steps will be repeated for each year for which data are available. These results will be combined to perform a corridor analysis consisting of one segment and two intersections.

Lastly, two additional alternatives for roadway improvements will be analyzed as part of an alternatives evaluation.

Intersections

The first part of the predictive method is focused on obtaining input data required to apply the predictive model. Detailed information on different recommendations related to data collection can be found in HSM Section 12.4 (HSM p. 12-6). The intersection data summary for this example is provided in Table 53.

Select and Apply SPF for Multiple- and Single-Vehicle Collisions



For the four-leg signalized intersection, SPF values for multiple-vehicle, single-vehicle, vehicle-pedestrian, and vehicle-bicycle collisions are determined. The general functional form of the multiple- and single-vehicle collision SPFs is shown in the following equations. The SPF for multiple-vehicle collisions is applied to calculate the predicted average crash frequency (total, fatal-and-injury, and PDO crashes) using HSM Equation 12-21 and HSM Table 12-10 (HSM p. 12-29 and 12-30, respectively). The SPF for single-vehicle crashes is applied to calculate the predicted average crash frequency (total, fatal-and-injury, and PDO crashes) using HSM Equation 12-24 and HSM Table 12-12 (HSM p. 12-32 and 12-33).

Multiple-Vehicle Collisions by Severity Level for Intersection 1

$$N_{bimv} = exp\left(a + b \times ln\left(AADT_{major}\right) + c \times ln\left(AADT_{minor}\right)\right)$$

$$N_{bimv} = exp\left(-10.99 + 1.07 \times ln(23{,}000) + 0.23 \times ln(14{,}000)\right)$$

$$N_{bimv} = 7.04 \frac{crashes}{year}$$

$$N_{bimv} = exp\left(-13.14 + 1.18 \times ln(23{,}000) + 0.22 \times ln(14{,}000)\right)$$

$$N'_{bimv\ int\ 1\ (FI)} = 2.25 \frac{crashes}{year}$$

$$N_{bimv} = exp\left(-11.02 + 1.02 \times ln(23{,}000) + 0.24 \times ln(14{,}000)\right)$$

$$N'_{bimv\ int\ 1\ (PDO)} = 4.55 \frac{crashes}{year}$$

Single-Vehicle Collisions by Severity Level for Intersection 1

$$N_{bisv} = exp\big(-10.21 + 0.68 \times ln(23{,}000) + 0.27 \times ln(14{,}000)\big)$$

$$N_{bisv\ int\ 1\ total} = 0.45 \frac{crashes}{year}$$

$$N_{bisv} = exp\big(-9.25 + 0.43 \times ln(23{,}000) + 0.29 \times ln(14{,}000)\big)$$

$$N'_{bisv\ int\ 1\ (FI)} = 0.12 \frac{crashes}{year}$$

$$N_{bisv} = exp\big(-11.34 + 0.78 \times ln(23{,}000) + 0.25 \times ln(14{,}000)\big)$$

$$N'_{bisv\ int\ 1\ (PDO)} = 0.33 \frac{crashes}{year}$$

The following adjustments are applied to the predicted average crash frequency for fatal and injury crashes and for PDO crashes to ensure the sum matches the total predicted number of crashes.

Multiple-Vehicle Collisions by Severity Level for Intersection 1

$$N_{bi} = N_{bi(FI)} + N_{bi(PDO)}$$

$$N_{bi\ int\ (FI)} = N_{bi\ int\ total} \times \frac{N'_{bi\ int\ (FI)}}{N'_{bi\ int\ (FI)} + N'_{bi\ int\ (PDO)}}$$

$$N_{bimv\ int\ 1\ (FI)} = 7.04 \times \frac{2.25}{2.25 + 4.55} = 2.33 \frac{crashes}{year}$$

$$N_{bimv\ int\ 1\ (PDO)} = 7.04 - 2.33 = 4.71 \frac{crashes}{year}$$

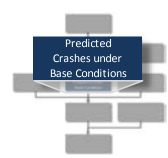
Single-Vehicle Collisions by Severity Level for Intersection 1

$$N_{bisv\ int\ 1\ (FI)} = 0.45 \times \frac{0.12}{0.12 + 0.33} = 0.12 \frac{crashes}{year}$$

$$N_{bisv\ int\ 1\ (PDO)} = 0.45 - 0.12 = 0.33 \frac{crashes}{year}$$

Select and Apply SPF for Vehicle-Pedestrian and Vehicle-Bicycle Collisions at Signalized Intersections

Vehicle-Pedestrian Collisions at Signalized Intersections



Vehicle-pedestrian collisions at signalized and unsignalized intersections are estimated using a different set of SPFs. For signalized intersections, use HSM Equation 12-29 (HSM p. 12-36) with coefficients from HSM Table 12-14 (HSM p. 12-37):

$$N_{pedbase} = exp\left(a + b \times ln(AADT_{total}) + c \times ln\left(\frac{AADT_{minor}}{AADT_{major}}\right) + d \times ln(PedVol) + e \times n_{lanesx}\right)$$

$$N_{pedbase} = exp\left(-9.53 + 0.40 \times ln(37{,}000) + 0.26 \times ln\left(\frac{14{,}000}{23{,}000}\right) + 0.45 \times ln(400) + 0.04 \times 5\right)$$

$$N_{pedbase} = 0.078 \frac{crashes}{year}$$

Vehicle-Bicycle Collisions at Signalized Intersections

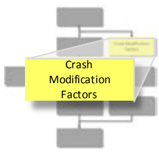
Vehicle-bicycle collisions are accounted for in HSM Equation 12-31 with intersection adjustment factors taken from HSM Table 12-17 (HSM p. 12-38).

Before calculating the vehicle-bicycle collisions, the predicted average crash frequency of multiple- and single-vehicle crashes must be calculated:

$$N_{spf\ int} = N_{bi\ mv\ int} + N_{bisv\ int}$$

$$N_{spf\ int} = 7.04 + 0.45 = 7.49$$

Apply HSM Part C Crash Modification Factors to Multiple- and Single-Vehicle Collisions



CMFs are applied to adjust the estimated crash frequencies for base conditions to account for the effect of site-specific geometry and traffic features.

Intersection Left-Turn Lanes (CMF_{1i})
The CMF for left-turn lanes is found in HSM Table 12-24 (HSM p. 12-43). Since Intersection 1 does not have left-turn lanes, a CMF of 1.00 is recommended.

Intersection Left-Turn Phasing (CMF_{2i})
HSM Table 12-25 (HSM p. 12-44) provides the CMF for various phasing types. The applied CMF is the product of each leg. For Intersection 1, no protected left-turn phasing is present; therefore, the CMF is equal to 1.00.

This CMF does not apply to stop-controlled intersections.

Intersection Right-Turn Lanes (CMF_{3i})
CMFs for installation of right-turn lanes are found in HSM Table 12-26 (HSM p. 12-44). Intersection 1 does not have right-turn lanes, yielding a CMF of 1.00.

Intersection Right-Turn-on-Red (CMF_{4i})
There are no right-turn-on-red prohibitions in Intersection 1; therefore, a CMF of 1.00 is applied. This CMF is applied by using HSM Equation 12-35 (HSM p. 12-44):

$$CMF_{4i} = 0.98^{(n_{prohib})}$$

This CMF does not apply to stop-controlled control intersections.

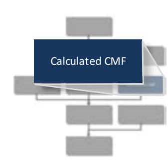
Intersection Lighting (CMF_{5i})
Intersection lighting is not present at the intersection; therefore, the CMF is equal to 1.00. To modify the crashes because of intersection lighting, HSM Equation 12-36 and HSM Table 12-27 (HSM p. 12-45) are used.

$$CMF_{5i} = 1 - 0.38 \times p_{ni}$$

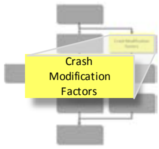
Intersection Red-Light Cameras (CMF_{6i})
Red-light cameras are not present at this intersection; therefore, a CMF of 1.00 is recommended. CMF can be estimated using HSM Equations 12-37 through 12-39 (HSM p. 12-45).

The combined CMF is then calculated by multiplying all the intersection-related CMFs:

$$CMF_{combined} = CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{6i}$$



Apply Part C Crash Modification Factors for Vehicle-Pedestrian and Vehicle-Bicycle Collisions at Signalized Intersections



Vehicle-Pedestrian Collisions at Signalized Intersections

Bus Stops (CMF_{1p})

Intersection 1 has a bus stop within 1,000 feet. The appropriate CMF from HSM Table 12-28 (HSM p. 12-46) is 2.78.

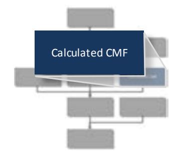
Schools (CMF_{2p})

The CMF for the presence of schools near intersections is presented in HSM Table 12-29 (HSM p. 12-46). No schools are present at Intersection 1; therefore, a CMF of 1.00 is applied.

Alcohol Sales Establishments (CMF_{3p})

Because of alcohol sales in proximity to Intersection 1, a CMF of 1.12 is applied. HSM Table 12-30 (HSM p. 12-47) provides the CMF values.

The combined CMF is then calculated by multiplying all the pedestrian-related CMFs:



$$CMF_{ped\ combined} = CMF_{1p} \times CMF_{2p} \times CMF_{3p}$$

$$CMF_{ped\ combined} = 2.78 \times 1.00 \times 1.12 = 3.11$$

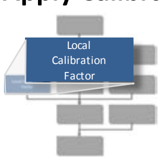
Vehicle-Bicycle Collisions at Signalized Intersections

The sum of the base conditions SPFs ($N_{spf\ int}$) is multiplied by the CMFs to obtain the predicted crash frequency (N_{bi}). This value will be later multiplied by a bicycle adjustment factor.

$$N_{bi} = N_{spf\ int} \times (CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{6i})$$

$$N_{bi} = 7.49 \times (1.00 \times 1.00 \times \ldots \times 1.00) = 7.49$$

Apply Calibration Factor



The next step is to multiply the results obtained above by the appropriate calibration factor. For this example, the intersection calibration factor is 1.15.

Obtain the Predicted Crash Frequency for the Site

Lastly, the predicted average crash frequency is calculated using HSM Equations 12-5, 12-6, and 12-7 (HSM p. 12-5 and 12-6), which combine the predicted average crash frequency, crash modification factors, and calibration factors:

$$N_{predicted\ int} = C_i \times (N_{bi} + N_{pedi} + N_{bikei})$$

$$N_{bi} = N_{spf\ int} \times (CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{6i})$$

$$N_{spf\ int} = N_{bimv} + N_{bisv}$$

Multiple-Vehicle Collisions by Severity Level for Intersection 1

$$N_{bimv\ total} = 7.04 \frac{crashes}{year}$$

$$N_{bimv\ (FI)} = 2.33 \frac{crashes}{year}$$

$$N_{bimv\ (PDO)} = 4.71 \frac{crashes}{year}$$

Single-Vehicle Collisions by Severity Level for Intersection 1

$$N_{bisv\ total} = 0.45 \frac{crashes}{year}$$

$$N_{bisv\ (FI)} = 0.12 \frac{crashes}{year}$$

$$N_{bisv\ (PDO)} = 0.33 \frac{crashes}{year}$$

$$N_{spf\ int} = N_{bimv\ total} + N_{bisv\ total} = 7.04 + 0.45 = 7.49\ \frac{crashes}{year}$$

$$N_{bi} = N_{spf\ int} \times (CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{6i})$$

$$N_{bi} = 7.49 \times (1.00 \times 1.00 \times \ldots \times 1.00) = 7.49\ \frac{crashes}{year}$$

Vehicle-Pedestrian Collisions at Signalized Intersections

$$N_{pedi} = N_{pedbase} \times \left(CMF_{1p} \times CMF_{2p} \times CMF_{3p}\right)$$

$$N_{pedi} = 0.078 \times (2.78 \times 1.00 \times 1.12) = 0.242 \frac{crashes}{year}$$

Vehicle-Bicycle Collisions at Signalized Intersections

The predicted crash frequency N_{bi} is multiplied by the bicycle crash adjustment factor (f_{bikei}) from HSM Table 12-17 (HSM p. 12-38) using the following equation:

$$N_{bikei} = N_{bi} \times f_{bikei}$$

$$N_{bikei} = 7.49 \times 0.015 = 0.112 \frac{crashes}{year}$$

The intersection predicted crash frequency is then calculated by multiplying the calibration factor by the sum of the multiple-, single-vehicle, pedestrian, and bicycle predicted crash frequencies.

$$N_{predicted\ int} = C_i \times \left(N_{bi} \times N_{pedi} \times N_{bikei}\right) = 1.15 \times (7.491 + 0.242 + 0.112)$$

$$N_{predicted\ int\ total} = 9.02 \frac{crashes}{year}$$

$$N_{predicted\ int\ (FI)} = C_i \times \left(N_{bi} \times N_{pedi} \times N_{bikei}\right) = 1.15 \times (2.447 + 0.242 + 0.112)$$

$$N_{predicted\ int\ (FI)} = 3.22 \frac{crashes}{year}$$

$$N_{predicted\ int(PDO)} = C_i \times (N_{bi}) = 1.15 \times (5.044)$$

$$N_{predicted\ int\ (PDO)} = 5.80 \frac{crashes}{year}$$

Multiyear Analysis

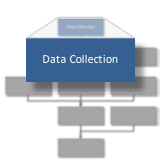
Since 5 years of data are available, all the steps above need to be repeated four more times. In this example, an AADT growth rate of 2 percent is assumed. Table 57 summarizes the calculations for the signalized intersection.

TABLE 57
Example Problem 3 – Intersection 1 Multiyear Analysis Results

Intersection 1	Year				
	2008	2009	2010	2011	2012
$AADT_{major}$	21,248	21,673	22,107	22,549	23,000
$AADT_{minor}$	12,934	13,193	13,456	13,725	14,000
Crashes/year	3	7	4	7	4
N_{brmv}	6.354	6.520	6.690	6.860	7.040
N_{brsv}	0.416	0.420	0.430	0.440	0.450
$N_{pedbase}$	0.075	0.076	0.076	0.077	0.078
N_{pedi}	0.234	0.236	0.238	0.240	0.242
N_{bikei}	0.102	0.104	0.107	0.110	0.112
$CMF_{1i\ 4SG}$	1.00	1.00	1.00	1.00	1.00
$CMF_{2i\ 4SG}$	1.00	1.00	1.00	1.00	1.00
$CMF_{3i\ 4SG}$	1.00	1.00	1.00	1.00	1.00
$CMF_{4i\ 4SG}$	1.00	1.00	1.00	1.00	1.00
$CMF_{5i\ 4SG}$	1.00	1.00	1.00	1.00	1.00
$CMF_{6i\ 4SG}$	1.00	1.00	1.00	1.00	1.00
CMF_{comb}	1.00	1.00	1.00	1.00	1.00
CMF_{1p}	2.78	2.78	2.78	2.78	2.78
CMF_{2p}	1.00	1.00	1.00	1.00	1.00
CMF_{3p}	1.12	1.12	1.12	1.12	1.12
$CMF_{ped\ comb}$	3.11	3.11	3.11	3.11	3.11
C_i	1.15	1.15	1.15	1.15	1.15
$N_{predicted\ int}$	8.171	8.376	8.586	8.801	9.022

The average predicted crash frequency for Intersection 1 is obtained by adding the arithmetic 5-year average of multiple- and single-vehicle, vehicle-pedestrian, and vehicle-bicycle annual predicted crash frequencies. For this example, this value is 8.59 crashes per year.

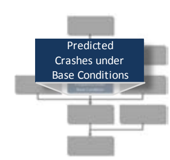
Roadway Segments



The first step in applying the predictive method is collecting the data required to apply the safety performance functions. Detailed information on different recommendations related to data collection can be found in HSM Section 12.4 (HSM p. 12-6).

Select and Apply SPF for Single-Vehicle Collisions and Multiple-Vehicle Driveway- and Nondriveway-Related Collisions

For urban and suburban arterials, SPFs values are calculated to multiple-vehicle non-driveway, single-vehicle, multiple-vehicle, vehicle-pedestrian, and vehicle-bicycle collisions.



The general functional form of the roadway segment multiple- and single-vehicle collision SPFs, excluding the driveway-related SPF (HSM Equation 12-16 [HSM p. 12-22]), is taken from HSM Equations 12-10 (multiple-vehicle collisions [HSM p. 12-18]) and 12-13 (single-vehicle crashes [HSM p. 12-20]), with appropriate regression coefficients selected from HSM Tables 12-3 and 12-5:

$$N_{br} = e^{a+b\times ln(AADT)+ln(L)}$$

Multiple-Vehicle Non-driveway-Related Collisions by Severity

$$N_{brmv\ seg1\ total} = e^{-9.70+1.17\times ln(23{,}000)+ln(0.3)}$$

$$N_{brmv\ seg\ 1\ total} = 2.33\,\frac{crashes}{year}$$

$$N'_{brmv\ seg\ 1\ (FI)} = e^{-10.47+1.12\times ln(23{,}000)+ln(0.3)}$$

$$N'_{brmv\ seg\ 1\ (FI)} = 0.65\,\frac{crashes}{year}$$

$$N'_{brmv\ seg1\ (PDO)} = e^{-9.97+1.17\times ln(23{,}000)+ln(0.3)}$$

$$N'_{brmv\ seg\ 1\ (PDO)} = 1.78\,\frac{crashes}{year}$$

Single-Vehicle Collisions by Severity

$$N'_{brsv\ seg\ 1\ total} = e^{-4.82+0.54\times ln(23{,}000)+ln(0.3)}$$

$$N_{brsv\ seg\ 1\ total} = 0.55\,\frac{crashes}{year}$$

$$N'_{brsv\ seg\ 1\ (FI)} = e^{-4.43+0.35\times ln(23{,}000)+ln(0.3)}$$

$$N'_{brsv\ seg\ 1\ (FI)} = 0.12\,\frac{crashes}{year}$$

$$N'_{brsv\ seg\ 1\ (PDO)} = e^{-5.83+0.61\times ln(23{,}000)+ln(0.3)}$$

$$N'_{brsv\ seg\ 1\ (PDO)} = 0.40\,\frac{crashes}{year}$$

After calculating the initial SPF, adjustment factors are applied to ensure the summation of the fatal-and-injury and PDO annual crashes match the total annual crashes. This is done using the following functional form, HSM Equations 12-11 and 12-14 for fatal-and-injury crashes, and taking the difference for PDO crashes using HSM Equations 12-12 and 12-15 (HSM p. 12-20 and 12-21):

$$N_{br\ seg\ (FI)} = N_{br\ seg\ total} \times \frac{N'_{br\ seg\ (FI)}}{N'_{br\ seg\ (FI)}+N'_{br\ seg\ (PDO)}}$$

Multiple-Vehicle Non-driveway-Related Collisions by Severity

$$N_{brmv\ seg\ 1\ (FI)} = 2.33 \times \frac{0.65}{0.65+1.78} = 0.63\,\frac{crashes}{year}$$

$$N_{brmv\ seg\ 1\ (PDO)} = 2.33 - 0.63 = 1.71\,\frac{crashes}{year}$$

Single-Vehicle Collisions by Severity

$$N_{brsv\ seg\ 1\ (FI)} = 0.55 \times \frac{0.12}{0.12 + 0.40} = 0.13 \frac{crashes}{year}$$

$$N_{brsv\ seg\ 1\ (PDO)} = 0.55 \times \frac{0.40}{0.12 + 0.40} = 0.42 \frac{crashes}{year}$$

Multiple-Vehicle Driveway-Related Collisions

The number of driveway-related collisions is calculated using HSM Equation 12-16 (HSM p. 12-22). Crashes per driveway type and traffic volume adjustment values are from HSM Table 12-7 (HSM p. 12-24). For the project roadway segment, there are two major commercial driveways, eight minor commercial driveways, and two minor residential driveways:

$$N_{brdwy\ seg\ total} = \sum_{\substack{all \\ driveway \\ types}} n_j \times N_j \times \left(\frac{AADT}{15{,}000}\right)^t$$

$$N_{brdwy\ sg\ tot} = 2 \times 0.165 \times \left(\frac{23{,}000}{15{,}000}\right)^{1.172} + 8 \times 0.053 \times \left(\frac{23{,}000}{15{,}000}\right)^{1.172} + 2 \times 0.016 \times \left(\frac{23{,}000}{15{,}000}\right)^{1.172}$$

$$= 1.30 \frac{crashes}{year}$$

The driveway-related fatal-and-injury and PDO crashes are calculated by applying proportions found in HSM Table 12-7 (HSM p. 12-24) is HSM Equations 12-17 and 12-18 (HSM p. 12-27). For this example, for a five-lane arterial with a TWLTL, the proportions of fatal-and-injury and PDO crashes are 0.269 and 0.731, respectively.

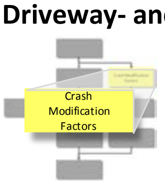
$$N_{brdwy\ seg\ (FI)} = N_{brdwy\ seg\ tot} \times f_{dwy}$$

$$N_{brdwy\ seg\ (FI)} = 1.30 \times 0.269 = 0.35 \frac{crashes}{year}$$

$$N_{brdwy\ seg\ (PDO)} = N_{brdwy\ seg\ tot} \times f_{dwy}$$

$$N_{brdwy\ seg\ (PDO)} = 1.30 \times 0.731 = 0.95 \frac{crashes}{year}$$

Apply HSM Part C Crash Modification Factors to Single-Vehicle Collisions and Multiple-Vehicle Driveway- and Non-driveway-Related Collisions



CMFs are applied to the estimated crash frequencies to adjust for base conditions, to account for the effect of site-specific geometry and traffic features.

On-Street Parking (CMF_{1r})

The CMF for on-street parking is calculated using HSM Equation 12-32 with the factor read from HSM Table 12-19 (HSM p. 12-40):

$$CMF_{1r} = 1 + p_{pk} \times (f_{pk} - 1)$$

$$CMF_{1r\ seg\ 1} = 1 + 0.40 \times (1.709 - 1) = 1.28$$

Roadside Fixed Objects (CMF_{2r})
For this CMF, HSM Equation 12-33 (HSM p. 12-40) is applicable, using the fixed-object offset factor from HSM Table 12-20 and the proportion of fixed-object collisions from HSM Table 12-21 (HSM p. 12-41):

$$CMF_{2r} = f_{offset} \times D_{fo} \times p_{fo} + (1 - p_{fo})$$

$$CMF_{2r\ seg\ 1} = 0.087 \times 20 \times 0.016 + (1 - 0.016) = 1.01$$

Median Width (CMF_{3r})
This CMF is applied to represent the effect of median width in reducing cross-median crashes. However, it is not applicable to medians serving as TWLTL. For this example, a CMF of 1.00 is appropriate, for all other conditions, use HSM Table 12-22 (HSM p. 12-42).

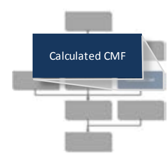
Lighting (CMF_{4r})
The effect of adding lighting along the roadway segment is calculated using HSM Equation 12-34, with proportions from HSM Table 12-23 (HSM p. 12-42):

$$CMF_{4r} = 1 - \left(p_{nr} \times (1 - 0.72 \times p_{inr} - 0.83 \times p_{pnr})\right)$$

$$CMF_{4r\ seg\ 1} = 1 - \left(0.274 \times (1 - 0.72 \times 0.432 - 0.83 \times 0.568)\right) = 0.94$$

Automated Speed Enforcement (CMF_{5r})
Automated speed enforcement is not present at the study segment; therefore, a CMF of 1.00 is appropriate. More information can be found on HSM p. 12-43.

The combined CMF is then calculated by multiplying all the segment-related CMFs.



$$CMF_{combined} = CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{5r}$$

$$CMF_{combined} = 1.28 \times 1.01 \times 1.00 \times 0.94 \times 1.00 = 1.22$$

Multiple-Vehicle Non-driveway-Related Collisions by Severity

$$N_{brmv\ seg\ 1\ total} = 2.33 \frac{crashes}{year}$$

$$N_{brmv\ seg\ 1\ (FI)} = 0.63 \frac{crashes}{year}$$

$$N_{brmv\ seg\ 1\ (PDO)} = 1.71 \frac{crashes}{year}$$

Single-Vehicle Collisions by Severity

$$N_{brsv\ seg\ 1\ total} = 0.55 \frac{crashes}{year}$$

$$N_{brsv\ seg\ 1\ (FI)} = 0.13 \frac{crashes}{year}$$

$$N_{brsv\ seg\ 1\ (PDO)} = 0.42 \frac{crashes}{year}$$

Multiple-Vehicle Driveway-Related Collisions by Severity

$$N_{brdwy\ seg\ 1\ total} = 1.30 \frac{crashes}{year}$$

$$N_{brdwy\ seg\ 1\ (FI)} = 0.35 \frac{crashes}{year}$$

$$N_{brdwy\ seg\ 1\ (PDO)} = 0.95 \frac{crashes}{year}$$

$$N_{spf\ total} = N_{brmv} + N_{brsv} + N_{brdwy}$$

$$N_{spf\ total} = 2.33 + 0.55 + 1.30 = 4.18 \frac{crashes}{year}$$

$$N_{spf\ (FI)} = 0.63 + 0.13 + 0.35 = 1.10 \frac{crashes}{year}$$

$$N_{spf\ (PDO)} = 1.71 + 0.42 + 0.95 = 3.08 \frac{crashes}{year}$$

$$N_{br} = N_{spf} \times (CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{5r})$$

$$N_{br\ total} = 4.18 \times (1.22) = 5.10 \frac{crashes}{year}$$

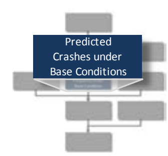
$$N_{br\ (FI)} = 1.10 \times (1.22) = 1.34 \frac{crashes}{year}$$

$$N_{br\ (PDO)} = 3.08 \times (1.22) = 3.75 \frac{crashes}{year}$$

Vehicle-Pedestrian and Vehicle-Bicycle Collisions for Urban Roadway Segments

The predictive method for the urban and suburban arterials does not include SPFs for pedestrian- and bicycle-related crashes. Vehicle-pedestrian and vehicle-bicycle collisions for urban segments are calculated as a proportion of the predicted multiple-vehicle non-driveway-related, single-vehicle, and multiple-vehicle driveway-related crashes (N_{br}). Pedestrian and bicycle adjustment factors are provided in the HSM.

Vehicle-Pedestrian Collisions along Segments



The SPF associated with vehicle-pedestrian collisions along segments is governed by HSM Equation 12-19 with the adjustment factor from HSM Table 12-8 (HSM p. 12-27):

$$N_{pedr} = N_{br} \times f_{pedr}$$

$$N_{pedr} = 5.10 \times 0.023 = 0.12 \frac{crashes}{year}$$

Vehicle-Bicycle Collisions along Segments

The SPF associated with vehicle-bicycle collisions along segments is similarly calculated, governed by HSM Equation 12-20 with the adjustment factor from HSM Table 12-9:

$$N_{biker} = N_{br} \times f_{biker}$$

$$N_{biker} = 5.10 \times 0.012 = 0.06 \frac{crashes}{year}$$

NOTE: These factors apply to the methodology for predicting all severity levels combined. All results obtained by applying these pedestrian and bicycle adjustment factors are treated as fatal-and-injury crashes. The adjustment factor does not apply to PDO crashes.

Apply Calibration Factor

The final step is to multiply the results obtained above by the appropriate calibration factor. For this example, the calibration factor has been assumed to be 1.10.

The predicted average crash frequency is calculated using HSM Equation 12.2 (HSM p. 12-4), which combines the predicted average crash frequency for base conditions, CMFs, and calibration factors:

$$N_{predicted\ rs} = C_r \times \left(N_{br} + N_{pedr} + N_{biker}\right)$$

$$N_{predicted\ rs} = 1.10 \times (5.10 + 0.12 + 0.06) = 5.81\ \frac{crashes}{year}$$

$$N_{predicted\ rs\ (FI)} = 1.10 \times (1.34 + 0.12 + 0.06) = 1.68\ \frac{crashes}{year}$$

$$N_{predicted\ rs\ (PDO)} = 1.10 \times (3.75) = 4.13\ \frac{crashes}{year}$$

Multiyear Analysis

Since 5 years of data are available, all the steps above need to be repeated four more times. In this example, an AADT growth rate of 2 percent is assumed. Table 58 summarizes the calculations for the roadway segment.

TABLE 58
Example Problem 3 – Roadway Segment 1 Multiyear Analysis Results

Roadway Segment 1	Year				
	2008	2009	2010	2011	2012
AADT	21,248	21,673	22,107	22,549	23,000
Crashes/year	11	14	10	13	12
N_{brmv}	2.125	2.175	2.226	2.278	2.332
N_{brsv}	0.526	0.531	0.537	0.543	0.548
N_{brdwy}	1.182	1.210	1.238	1.267	1.297
N_{ped}	0.108	0.110	0.112	0.115	0.117
N_{bike}	0.056	0.057	0.059	0.060	0.061
CMF_{1ru}	1.28	1.28	1.28	1.28	1.28
CMF_{2ru}	1.01	1.01	1.01	1.01	1.01
CMF_{3ru}	1.00	1.00	1.00	1.00	1.00
CMF_{4ru}	0.94	0.94	0.94	0.94	0.94
CMF_{5ru}	1.00	1.00	1.00	1.00	1.00
CMF_{comb}	1.22	1.22	1.22	1.22	1.22
C_r	1.10	1.10	1.10	1.10	1.10
$N_{predicted\ seg}$	5.33	5.45	5.56	5.68	5.81

The average predicted crash frequency for the study segment is obtained by adding the arithmetic 5-year average of multiple- and single-vehicle, multiple-vehicle driveway-related, vehicle-pedestrian, and vehicle-bicycle annual predicted crash frequencies. For this example, this value is 5.57 crashes per year.

Corridor Analysis (Intersections and Roadway Segments)

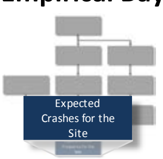
Analysis results for intersections and roadway segments can be combined into a corridor analysis. This approach combines the predicted crash frequency of multiple locations to come up with corridor predicted average crash frequency. Table 59 summarizes the predicted crash frequency of all roadway segments and intersections, and provides the corridor totals.

TABLE 59
Example Problem 3 – Corridor Predicted Average Crash Frequency

Site Type	Predicted Average Crash Frequency (crashes/year)		
	$N_{predicted}$ (Total)	$N_{predicted}$ (Fatal-&-Injury)	$N_{predicted}$ (PDO)
Roadway Segment 1	5.57	1.61	3.96
Intersection 1	8.59	3.07	5.53
Intersection 2	2.74	1.05	1.69
Project Total	**16.90**	**5.72**	**11.18**

HSM Tables 12-4 (HSM p. 12-20) and 12-6 (HSM p. 12-22) provide default distributions of crashes by collision type and severity level for multiple-vehicle non-driveway and single-vehicle roadway segment crashes, respectively. HSM Tables 12-11 (HSM p. 12-32) and 12-13 (HSM p. 12-36) provide default distributions of crashes by collision type and severity level for multiple-vehicle and single-vehicle intersection crashes, respectively. These proportions can be applied to the predicted crash frequencies for selected collision types. The HSM provides information on how to update these values using local data (refer to HSM Part C, Appendix A for details).

Empirical Bayes Adjustment Method



For this example, observed crash data is available by location; therefore, the predictions can be adjusted using the EB method. Details on the applicability of the EB method can be found in HSM Section A.2.1 (HSM p. A-16).

After making the adjustments, the expected average crash frequencies for roadway segments and intersections can be combined to come up with a corridor expected average crash frequency. In this example, crash data is available by site; therefore, the site EB method is applicable. Refer to HSM Sections A.2.4 and A.2.5 (HSM p. A-19 and A-20) for additional details on the different EB methods.

Available observed crash data for segments and intersections has been broken down into multiple-vehicle and single-vehicle crashes, as shown in Table 60.

TABLE 60
Example Problem 3 – Disaggregated Roadway Segment and Intersection Crash Data for the Study Period (2008 to 2012)

Collision Type	Intersection 1						
	2008	**2009**	**2010**	**2011**	**2012**	**Sum**	**Average**
Multiple-Vehicle Non-driveway	3	6	4	7	4	24	4.8
Single-Vehicle	0	1	0	0	0	1	0.2
Total	3	7	4	7	4	25	5
Collision Type	**Intersection 2**						
	2008	**2009**	**2010**	**2011**	**2012**	**Sum**	**Average**
Multiple-Vehicle Non-driveway	2	6	5	3	4	20	4
Single-Vehicle	0	0	0	0	0	0	0
Total	2	6	5	3	4	20	4
Collision Type	**Roadway Segment**						
	2008	**2009**	**2010**	**2011**	**2012**	**Sum**	**Average**
Multiple-Vehicle Non-driveway	5	7	6	8	9	35	7
Single-Vehicle	0	2	1	1	1	5	1
Multiple-Vehicle Driveway-Related	6	5	3	4	2	20	4
Total	11	14	10	13	12	60	12

There were 60 roadway segment crashes and 45 intersection crashes for the study period.

The expected number of crashes for either segments or intersections is calculated using HSM Equation A-4 (HSM p. A-19):

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed}$$

The weighting adjustment factors for each collision type for the sample roadway segments and intersections are needed to complete these calculations. HSM Equation A-5 (HSM p. A-19) is used to obtain the weighting factors:

$$w = \frac{1}{1 + k \times \sum_{\substack{all \\ study \\ years}} N_{predicted}}$$

Overdispersion parameters are also estimated for each SPF set. The overdispersion parameter associated with segment SPFs are found in HSM Table 12-3 (multiple-vehicle non-driveway-related [HSM p. 12-19]), Table 12-5 (single-vehicle [HSM p. 12-21]), and Table 12-7 (multiple-vehicle driveway-related [HSM p. 12-24]). Intersection overdispersion parameters for multiple- and single-vehicle collisions can be found in HSM Tables 12-10 (HSM p. 12-30) and 12-12 (HSM p. 12-33), respectively. Intersection overdispersion parameters for vehicle-pedestrian collisions can be found in HSM Table 12-14 (HSM p. 12-37).

The following is an example of how to calculate the weighting adjustment factor for segment multiple-vehicle non-driveway collisions using the weighting factors equation. The segment overdispersion parameter for this collision type is 0.81, and the sum of all the predicted roadway segment crashes is 14.96:

$$w_{brmv\ seg\ 1\ total} = \frac{1}{1 + 0.81 \times (2.86 + 2.92 + 2.99 + 3.06 + 3.13)} = 0.076$$

The segment predicted average crash frequency for this collision type is 2.99 crashes per year. The expected number of crashes is calculated as follows:

$$N_{expected\ mv\ seg\ 1\ total} = 0.076 \times 2.992 + (1 - 0.076) \times 7 = \ 6.70 \frac{crashes}{year}$$

Expected average crash frequencies are presented in Table 61. Columns 2 through 4 contain the predicted average crash frequency for total crashes, fatal-and-injury, and PDO. The fifth column contains the observed/reported number of crashes per year. Columns 6 and 7 contain the overdispersion parameter and weighted adjustment to be used to obtain the expected average crash frequency (last column).

TABLE 61
Example Problem 3 – Predicted and Expected Crash Frequency Calculations Summary (2008 to 2012)

Collision Type/ Site Type	Predicted Average Crash Frequency (crashes/year)			Observed/ Reported Crashes ($N_{observed}$) (crashes/ year)	Over-dispersion Parameter (k)	Weighted Adjustment (w) (Equation A-5 from HSM Part C, Appendix A)	Expected Average Crash Frequency ($N_{expected}$) (Equation A-4 from HSM Part C, Appendix A)
	$N_{predicted}$ (Total)	$N_{predicted}$ (Fatal-&-Injury)	$N_{predicted}$ (PDO)				
Roadway Segments							
Multiple-Vehicle Non-driveway							
Roadway Segment 1	2.99	0.80	2.19	7	0.810	0.076	6.69
Single-Vehicle							
Roadway Segment 1	0.72	0.17	0.55	1	0.520	0.348	0.90
Multiple-Vehicle Driveway-Related							
Roadway Segment 1	1.66	0.45	1.22	4	0.100	0.546	2.73
Intersections							
Multiple-Vehicle							
Intersection 1	7.70	2.54	5.16	5	0.390	0.062	4.98
Intersection 2	2.38	0.87	1.51	4	0.800	0.095	3.85
Single-Vehicle							
Intersection 1	0.50	0.13	0.37	0	0.360	0.528	0.36
Intersection 2	0.26	0.08	0.18	0	1.140	0.406	0.10
Total	**16.21**	**5.03**	**11.18**	**21**	**–**	**–**	**19.61**

The total expected crashes for the site is the sum of the roadway segment and intersections. Table 62 summarizes the predicted values for bicycle and pedestrian crashes. Agencies with observed/reported crashes for these types can likewise calculate expected crashes with an overdispersion parameter. In the absence of observed/reported pedestrian and bicycle crashes, the total and fatal-and-injury (not applicable to PDO) predicted crash frequencies for roadway segments and intersections are calculated by adding the multiple-vehicle and single-vehicle crashes to the pedestrian and bicycle predicted crashes.

TABLE 62
Example Problem 3 – Predicted Pedestrian and Bicycle Average Crash Frequency (2008 to 2012)

Site Type	Predicted Average Crash Frequency	
	N_{ped}	N_{bike}
Roadway Segments		
Roadway Segment 1	0.124	0.065
Intersections		
Intersection 1	0.274	0.123
Intersection 2	0.064	0.042
Combined	**0.461**	**0.230**

Lastly, the total expected average number of crashes for the corridor is 20.3 crashes per year, as shown in Table 63. Results of the analysis can be found in the sample spreadsheets provided with the *Highway Safety Manual User Guide*.

TABLE 63
Example Problem 3 – Corridor Predicted and Expected Crash Frequencies

Crash Severity Level	$N_{predicted}$	N_{ped}	N_{bike}	$N_{predicted\ Total}$	$N_{expected\ (vehicle)}$	$N_{expected\ Total}$
Total						(3)+(4)+(6)
	16.21	0.46	0.23	16.90	19.61	20.30
Fatal and Injury					(6)Total × (2)FI / (2) Total	(3)+(4)+(6)
	5.03	0.46	0.23	5.72	6.09	6.78
PDO		–	–		(6)Total × (2)PDO / (2) Total	(3)+(4)+(6)
	11.18	0.00	0.00	11.18	13.52	13.52

Alternatives Analysis

The first section of this example demonstrated the application of the predicted method for urban and suburban arterial roadway segments and intersections. The predictive method can also be applied to alternative analysis. This process is more detailed and specific about the impacts of the project. The agency develops potential alternatives and compares performance across alternatives, as shown in Figure 26. The next example shows how to apply the urban and suburban arterials predictive method to compare alternatives. Calculations and formulas are similar to the previous example, and results are provided in summary tables.

No Build. The facility is an urban arterial with commercial development. A TWLTL provides for left-turn movements to and from the median. The current configuration allows for left-turn movements at any location along the corridor. The corridor has on-street parallel parking. The posted speed limit is 35 mph. Properties adjacent to the facility have multiple direct access points to the corridor. The pedestrian sidewalk is restricted to 3 feet at some locations along the corridor.

Alternative 1. No land use changes are anticipated in the facility. A physical 14-foot median is added in one section of the corridor. The remainder of the corridor remains as TWLTL. Bus pullout areas are provided at the existing bus stop. The alternative provides a 12-foot sidewalk with a 2-foot buffer.

Single left-turn lanes and phasing are added to the major road at the signalized intersection. Intersection lighting is added to both intersections.

Alternative 2. No land use changes are anticipated along the corridor. Right-of-way acquisition led to the addition of a median separation and dedicated HOV lane. A 12-foot pedestrian sidewalk is provided with a 3-foot buffer. The TWLTL is replaced with exclusive left-turn lanes, which provide limited access to left-turning vehicles at dedicated locations across the corridor. Minor commercial driveways in large parking areas are consolidated along the corridor, going from eight to four in total.

Left-turn lanes and protected left-turn phasing are provided for all four legs at the signalized intersection. Exclusive right- and left-turn lanes are provided at the three-leg unsignalized intersection.

Figure 26: Example Problem 2 – Project Alternatives

It is assumed that AADT remains the same in each alternative, and the road does not attract any additional traffic. Tables 64 and 65 contain the input data for the different scenarios. Only geometric elements that are being upgraded are listed in Tables 64 and 65. The previous example is the No Build condition.

TABLE 64
Example Problem 3 – Intersection Alternatives Input Data

Characteristics	Input Data by Alternative		
	No Build	**Alternative 1**	**Alternative 2**
Intersection 1			
Intersection type	4SG	4SG	4SG
Intersection lighting	Not present	Present	Present
Data for Signalized Intersections Only			
Number of approaches with left-turn lanes	0	2	4
Number of approaches with left-turn signal phasing	0	2	4
Type of left-turn signal phasing for leg 1		Protected/ Permitted	Protected
Type of left-turn signal phasing for leg 2		Protected/ Permitted	Protected
Type of left-turn signal phasing for leg 3			Protected/ Permitted
Type of left-turn signal phasing for leg 4 (if applicable)			Protected/ Permitted
Maximum number of lanes crossed by a pedestrian	5	5	7
Intersection 2			
Intersection type	3ST	3ST	3ST
Intersection lighting	Not present	Present	Present
Data for Unsignalized Intersections Only	--	--	--
Number of major-road approaches with left-turn lanes	0	0	2
Number of major-road approaches with right-turn lanes	0	0	1

TABLE 65
Example Problem 3 – Roadway Segments Alternatives Input Data

Segment Characteristics	Input Data by Alternative		
	No Build	**Alternative 1**	**Alternative 2**
Roadway type	5T	5T	4D
Type of on-street parking	Parallel (Commercial/ Industrial)	None	None
Proportion of curb length with on-street parking	0.4	0	0
Median width (feet) – for divided only	Not present	Not present	10
Minor commercial driveways	8	8	4
Offset to roadside fixed objects (feet)	10	2	15

The effect of the multiple safety countermeasures (such as lighting and adding left-turn lanes) is reflected in the decrease of predicted average crash frequency.

These safety improvements are all taken into account by the application of CMFs, which are used to adjust the SPF base condition estimate of predicted average crash frequency for the effect of the individual geometric design and traffic control features. The CMF for the SPF base condition of each geometric design or traffic control feature has a value of 1.00.

Calculations for the No Build scenario are the same as the first part of the example. Table 66 summarizes the results for all alternatives. Total predicted, observed, and expected average crash frequencies are provided.

TABLE 66
Example Problem 3 – Alternative Analysis Summary Results

Alternative	Site Type	$N_{predicted}$	$N_{observed}$	$N_{expected}$
No Build	Roadway Segment 1	5.6	12	10.5
	Intersection 1	8.6	5	5.8
	Intersection 2	2.7	4	4.1
	Total	**16.9**	**21**	**20.4**
Alternative 1	Roadway Segment 1	4.5	12	10
	Intersection 1	6.2	5	5.5
	Intersection 2	2.5	4	4
	Total	**13.3**	**21**	**19.5**
Alternative 2	Roadway Segment 1	1.5	12	9.2
	Intersection 1	4.6	5	5.3
	Intersection 2	1	4	3.4
	Total	**7.1**	**21**	**17.9**

Results and Discussion

The use of the HSM in alternative evaluation allows the agency to quantify the impact of safety improvements such as removing on-street parking, consolidating driveways, installing a raised median, and adding left-turn lanes and phasing. This gives the agency a tool that provides valuable information in the decision-making process.

NOTE: The HSM does not require any agency to implement a particular alternative based solely on the safety performance evaluation, and it is not intended to be a substitute for the exercise of sound engineering judgment.

The No Build predicted crash frequency is lower than the observed crash frequency. This indicates that more crashes are occurring on the site than the average site with similar characteristics.

The results in Table 66 also indicate that implementation of Alternatives 1 and 2 would reduce the predicted number of crashes by 22 percent and 58 percent, respectively. However, after the EB adjustment using observed crash data, the expected number of crashes for Alternatives 1 and 2 are 4 percent and 12 percent lower, respectively, than the No Build scenario.

Overall, the different improvement projects are anticipated to reduce the total crashes for both alternatives. However, an economic evaluation is required to better understand which alternative is the most cost-effective. Refer to HSM Chapter 7, Economic Appraisal, for methods to compare the benefits of potential safety countermeasures to crash costs.

3.3 HSM in Design

3.3.1 Overview

Historically, the highway design process was based on the application of established design criteria. Adherence to design standards was viewed as the means to establish an acceptable level of safety. With the release of the HSM, designers are provided with tools to perform safety performance-based design. This allows development of solutions based not just on design standards, but also on quantifying the safety performance of different design considerations. For instance, designers can establish the safety impact of changing a design parameter, evaluate the impact of design exceptions on safety performance, assess the interactions of the road user with the highway, and evaluate design solutions based on user abilities and limitations using the human factors information included in the manual.

3.3.2 Example Problem 4 Evaluation of Curve Realignment versus Design Exception

Introduction

The example is a rural two-lane road that is being upgraded from a posted speed limit of 40 mph to 60 mph. Several changes in the roadway alignment are expected, particularly around curves. However, one curve location is adjacent to a high-quality wetland, and reconstructing such a curve may present a challenge from permitting, constructability, and cost points of view. The other option is to leave the existing curve geometry untouched and request a design exception. To mitigate the potential adverse effects of the design exception, some improvements are considered, including shoulder widening and paving shoulders. To better understand the safety benefits of reconstruction versus design exception, an analysis for both alternatives was conducted and is described in the following sections. Five years of crash data are available (2008 to 2012). In this example, the existing curve will be referred as Roadway Segment 1, the proposed curve will be referred to as Roadway Segment 2, and the existing curve with mitigation measures will be referred to as Roadway Segment 3.

Objectives

This example is focused on determining the safety performance of two design alternatives of a curve location to help design engineers with the decision-making process. The problem illustrates how to calculate the predictive and expected average crash frequency for two curve locations with different radii.

After reviewing this example, the user should be able to:

- Understand what input data are required and the assumptions that are commonly made regarding default values for the HSM procedures
- Calculate the predicted and expected crash frequency of rural two-lane curve segments using the HSM
- Understand how to reasonably interpret the results from an HSM analysis and how these results can be used to support a particular decision
- Understand the limitations of the HSM procedures and when it is appropriate to use other models or computational tools

Data Requirements

Roadway Segment Data

Table 67 contains the input data for this analysis.

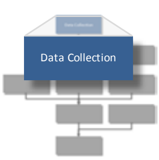
TABLE 67
Example Problem 4 – Curve Segments Input Data

Characteristics	Input Data	
	Roadway Segment 1	**Roadway Segment 2**
Segment length (feet)	0.24	0.30
Traffic volume (vpd)	13,500	13,500
Lane width (feet)	12	12
Shoulder width (feet)	2	6
Shoulder type	Gravel	Paved
Length of horizontal curve (feet)	0.24	0.30
Radius of curvature (feet)	1,600	2,000
Spiral transition curve	Not present	Not present
Superelevation variance	0.02	0
Grade	2	2
Driveway density	0	0
Centerline rumble strips	Not present	Not present
Passing lanes	Not present	Not present
TWLTL	Not present	Not present
Roadside hazard rating (RHR)	4	3
Segment lighting	Not present	Not present
Auto speed enforcement	Not present	Not present
Calibration factor (C_r)	1.23	1.23
Observed crash data (crashes/year)	12	12

Analysis

Calculations for Roadway Segments 1 and 2 shown next are for year 2012. Details on the multiyear analysis are provided in the following sections.

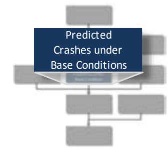
Roadway Segments



Segment data required to apply the predictive method are summarized in Table 67. Roadway Segment 1 length is 0.24 mile with a curve radius of 1,600 feet. Roadway Segment 2 length is 0.3 mile with a curve radius of 2,000 feet. As part of the new realignment, Roadway Segment 2 shoulder type is upgraded from gravel to paved and widened from 2 feet to 6 feet. The superelevation and roadside hazard rating (RHR) for

Roadway Segment 2 are also upgraded. All remaining parameters are the same for both locations. Information on different recommendations related to data collection is presented in HSM Section 10.4.

Select and Apply SPFs



For the selected site, apply the appropriate SPF for rural two-lane, two-way roads. The SPF can be calculated using HSM Equation 10-6 (HSM p. 10-15):

$$N_{spf} = AADT \times L \times 365 \times 10^{-6} \times e^{-0.312}$$

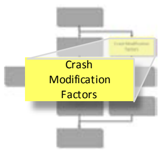
$$N_{spf\ seg\ 1} = 13{,}500 \times 0.24 \times 365 \times 10^{-6} \times e^{-0.312}$$

$$N_{spf\ seg\ 1} = 0.87\ crashes/year$$

$$N_{spf\ seg\ 2} = 13{,}500 \times 0.30 \times 365 \times 10^{-6} \times e^{-0.312}$$

$$N_{spf\ seg\ 2} = 1.08\ crashes/year$$

Apply HSM Part C Crash Modification Factors



Multiply the result obtained above by the appropriate CMFs to adjust the estimated crash frequency for base conditions to the site-specific geometry and traffic features.

Lane Width (CMF_{1r})
CMF_{1r} can be calculated using HSM Equation 10-11 (HSM p. 10-24) shown below:

$$CMF_{1r} = (CMF_{ra} - 1) \times p_{ra} + 1$$

CMF_{ra} is estimated using HSM Table 10-8 (HSM p. 10-24). For a 12-foot lane width and AADT greater than 2,000, the CMF for the effect of lane width on related crashes (such as single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes) is 1.00.

For this example, since the lane width is the same as the base conditions, the applicable CMF for both roadway segments is 1.00.

Shoulder Width and Type (CMF_{2r})
CMF_{2r} can be calculated using HSM Equation 10-12 (HSM p. 10-27). For this example, a 2-foot gravel shoulder yields a CMF_{wra} of 1.18 for Roadway Segment 1 and 1.00 for Roadway Segment 2 (shoulder width HSM Table 10-9 [HSM p. 10-25]) and CMF_{tra} of 1.0 (shoulder type HSM Table 10-10 [HSM p. 10-26]). The percentage of related crashes is the same as that calculated for the lane width CMF:

$$CMF_{2r} = (CMF_{wra} \times CMF_{tra} - 1) \times p_{ra} + 1$$

$$CMF_{2r\ seg\ 1} = (1.3 \times 1.01 - 1) \times (0.521 + 0.016 + 0.037) + 1$$

$$CMF_{2r\ seg\ 1} = 1.18$$

$$CMF_{2r\ seg\ 2} = (1 \times 1 - 1) \times (0.574) + 1$$

$$CMF_{2r\ seg\ 2} = 1.00$$

Horizontal Curve (CMF_{3r})

For this example, Roadway Segment 1 length is 0.24 mile with a radius of curvature of 1,600 feet, and Roadway Segment 2 length is 0.30 mile with a radius of curvature of 2,000 feet. The CMF is calculated using HSM Equation 10-13 (HSM p. 10-27):

$$CMF_{3r} = \frac{1.55 \times L_c + \frac{80.2}{R} - 0.012 \times S}{1.55 \times L_c}$$

$$CMF_{3r\ seg\ 1} = \frac{1.55 \times 0.24 + \frac{80.2}{1{,}600} - 0.012 \times 0}{1.55 \times 0.24}$$

$$CMF_{3r\ seg\ 1} = 1.13$$

$$CMF_{3r\ seg\ 2} = \frac{1.55 \times 0.30 + \frac{80.2}{2{,}000} - 0.012 \times 0}{1.55 \times 0.30}$$

$$CMF_{3r\ seg\ 2} = 1.09$$

Superelevation (CMF_{4r})

The superelevation variance for Roadway Segment 1 is 0.02 feet per foot, and for Roadway Segment 2 it is 0. Therefore, the superelevation is calculated using HSM Equation 10-16 (HSM p. 10-28):

$$CMF_{4r\ SV \geq 0.02} = 1.06 + 3 \times (SV - 0.02)$$

$$CMF_{4r\ SV \geq 0.02} = 1.06 + 3 \times (0.02 - 0.02)$$

$$CMF_{4r\ seg\ 1} = 1.06$$

$$CMF_{4r\ seg\ 2<0.01} = 1.00$$

Grade (CMF_{5r})

A 2 percent grade section falls under the level grade category in HSM Table 10-11 (HSM p. 10-28), resulting in a CMF of 1.00 for both roadway segments.

Driveway Density (CMF_{6r})

Driveway density of less than five driveways per mile leads to a CMF_{6R} of 1.00. Otherwise, the CMF is calculated using HSM Equation 10-17 (HSM p. 10-29):

$$CMF_{6r} = \frac{0.322 + DD \times [0.05 - 0.005 \times ln(AADT)]}{0.322 + 5 \times [0.05 - 0.005 \times ln(AADT)]}$$

$$CMF_{6r\ seg\ 1} = 1.00$$

$$CMF_{6r\ seg\ 2} = 1.00$$

Centerline Rumble Strips (CMF_{7r})

The roadway segments do not have centerline rumble strips; therefore, a CMF of 1.00 is applied. See HSM p. 10-29 for additional details.

Passing Lanes (CMF_{8r})

Passing lanes are not present in the example; therefore, a CMF of 1.00 is appropriate for both roadway segments. See HSM p. 10-29 for additional details.

Two-Way, Left-Turn Lane (CMF_{9r})

TWLTLs are not present; therefore, a CMF of 1.00 is applied for this example. See HSM p. 10-29 for additional details.

Roadside Design (CMF_{10r})
The data in this example indicate a roadside hazard rating of 4 for Roadway Segment 1, and a rating of 3 for Roadway Segment 2. The CMF is calculated using HSM Equation 10-20 (HSM p. 10-30):

$$CMF_{10r} = \frac{e^{-0.6869+0.0668\times RHR}}{e^{-0.4865}}$$

$$CMF_{10r} = \frac{e^{-0.6869+0.0668\times 4}}{e^{-0.4865}}$$

$$CMF_{10r\ seg\ 1} = 1.07$$

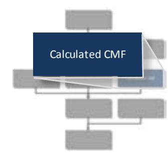
$$CMF_{10r} = \frac{e^{-0.6869+0.0668\times 3}}{e^{-0.4865}}$$

$$CMF_{10r\ seg\ 2} = 1.00$$

Lighting (CMF_{11r})
Lighting is not present at this location; therefore, a CMF of 1.00 is applied. See HSM p. 10-30 for additional details.

Automated Speed Enforcement (CMF_{12r})
The site does not have automated speed enforcement available; therefore, a CMF of 1.00 is applied. See HSM p. 10-30 for additional details.

The combined CMF is then calculated by multiplying all the intersection CMFs:

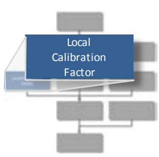


$$CMF_{combined} = CMF_{1r} \times CMF_{2r} \times CMF_{3r} \times \cdots \times CMF_{12r}$$

$$CMF_{comb\ seg\ 1} = 1.0 \times 1.18 \times 1.13 \times 1.06 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.07 \times 1.0 \times 1.0 = 1.517$$

$$CMF_{comb\ seg\ 2} = 1.0 \times 1.0 \times 1.09 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 1.086$$

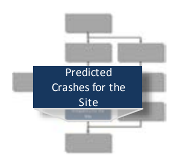
Apply Calibration Factor



Multiply the predicted average crash frequency and CMF results obtained in previous steps by the appropriate calibration factor. For this example, the calibration factor has been assumed to be 1.23.

Obtain the Predicted Crash Frequency for the Site

The predicted average crash frequency is calculated using HSM Equation 10-2 (HSM p. 10-3), combining results from previous steps:



$$N_{predicted\ rs} = N_{spf\ rs} \times C_r \times (CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{12r})$$

$$N_{predicted\ rseg\ 1} = 0.87 \times 1.23 \times (1.52)$$

$$N_{predicted\ rseg\ 1} = 1.62\ crashes/year$$

$$N_{predicted\ rseg\ 2} = 1.08 \times 1.23 \times (1.09)$$

$$N_{predicted\ rseg\ 2} = 1.45\ crashes/year$$

Multiyear Analysis

Since 5 years of data are available, all the previous steps need to be repeated four more times. In this example, a growth rate of 1.5 percent is assumed. Table 68 summarizes the calculations for the study period.

TABLE 68
Example Problem 4 – Roadway Segment 1 Multiyear Analysis Results

Roadway Segment 1	Year				
	2008	2009	2010	2011	2012
AADT	12,719	12,910	13,104	13,300	13,500
N_{spf}	0.816	0.828	0.84	0.853	0.866
CMF_{1r}	1.00	1.00	1.00	1.00	1.00
CMF_{2r}	1.18	1.18	1.18	1.18	1.18
CMF_{3r}	1.13	1.13	1.13	1.13	1.13
CMF_{4r}	1.06	1.06	1.06	1.06	1.06
CMF_{5r}	1.00	1.00	1.00	1.00	1.00
CMF_{6r}	1.00	1.00	1.00	1.00	1.00
CMF_{7r}	1.00	1.00	1.00	1.00	1.00
CMF_{8r}	1.00	1.00	1.00	1.00	1.00
CMF_{9r}	1.00	1.00	1.00	1.00	1.00
CMF_{10r}	1.07	1.07	1.07	1.07	1.07
CMF_{11r}	1.00	1.00	1.00	1.00	1.00
CMF_{12r}	1.00	1.00	1.00	1.00	1.00
CMF_{comb}	1.52	1.52	1.52	1.52	1.52
C_r	1.23	1.23	1.23	1.23	1.23
$N_{predicted\ seg}$	1.52	1.54	1.57	1.59	1.62

TABLE 69
Example Problem 4 – Roadway Segment 2 Multiyear Analysis Results

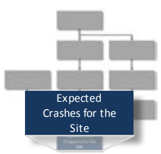
Roadway Segment 2	Year				
	2008	2009	2010	2011	2012
AADT	12,719	12,910	13,104	13,300	13,500
N_{spf}	1.019	1.035	1.050	1.066	1.082
CMF_{1r}	1.00	1.00	1.00	1.00	1.00
CMF_{2r}	1.00	1.00	1.00	1.00	1.00
CMF_{3r}	1.09	1.09	1.09	1.09	1.09
CMF_{4r}	1.00	1.00	1.00	1.00	1.00
CMF_{5r}	1.00	1.00	1.00	1.00	1.00
CMF_{6r}	1.00	1.00	1.00	1.00	1.00
CMF_{7r}	1.00	1.00	1.00	1.00	1.00
CMF_{8r}	1.00	1.00	1.00	1.00	1.00

TABLE 69
Example Problem 4 – Roadway Segment 2 Multiyear Analysis Results

Roadway Segment 2	Year				
	2008	**2009**	**2010**	**2011**	**2012**
CMF_{9r}	1.00	1.00	1.00	1.00	1.00
CMF_{10r}	1.00	1.00	1.00	1.00	1.00
CMF_{11r}	1.00	1.00	1.00	1.00	1.00
CMF_{12r}	1.00	1.00	1.00	1.00	1.00
CMF_{comb}	1.09	1.09	1.09	1.09	1.09
C_r	1.23	1.23	1.23	1.23	1.23
$N_{predicted\ seg}$	1.36	1.38	1.40	1.42	1.45

The average predicted crash frequency for Roadway Segments 1 and 2 are obtained through the arithmetic average of the annual predicted crash frequencies ($N_{predicted\ seg}$). The average for Roadway Segments 1 and 2 are 1.57 and 1.40 crashes per year, respectively.

Empirical Bayes Adjustment Method



The next step in the process is to update predictions based on the observed/reported crashes. Twelve roadway segment crashes occurred per year. The predictive models indicate that the total predicted average crash frequencies for Roadway Segments 1 and 2 are 1.57 and 1.40 crashes per year, respectively.

The predicted average crash frequency is then adjusted using the EB method by applying the following steps.

In this example, the proposed geometric upgrade represents a minor change in alignment; therefore, the EB method is applicable. Refer to HSM Section A.2.1 (HSM p. A-16) for additional details about the applicability of the EB method.

The site EB method is applicable. Refer to HSM Section A.2.5 (HSM p. A-20 to A-22) for additional details on the different EB methods.

The expected number of crashes for segments is calculated by HSM Equation A-4 (HSM p. A-19):

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed}$$

To complete this calculation, weighting adjustment factors are needed for the samples. Calculate using the previous crash predictions with HSM Equation A-5 (HSM p. A-19):

$$w = \frac{1}{1 + k \times \sum_{\substack{all\\study\\years}} N_{predicted}}$$

For this calculation, the overdispersion parameter from each of the applied SPFs is needed. The overdispersion parameter for Roadway Segment 1 is 0.983 and for Roadway Segment 2 is 0.787. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. On a per-mile basis, the overdispersion parameter is calculated by using HSM Equation 10-7 (HSM p. 10-16):

$$k_{seg} = \frac{0.236}{L}$$

$$k_{seg\ 1} = \frac{0.236}{0.24}$$

$$k_{seg\ 1} = 0.983$$

$$k_{seg\ 2} = \frac{0.236}{0.30}$$

$$k_{seg\ 2} = 0.787$$

Using these overdispersion parameters, the weighting adjustment factors are found to be 0.115 and 0.153 for Roadway Segments 1 and 2, respectively:

$$w_{seg\ 1} = \frac{1}{1 + 0.983 \times (1.52 + 1.54 + 1.57 + 1.59 + 1.62)}$$

$$w_{seg\ 1} = 0.115$$

$$w_{seg\ 2} = \frac{1}{1 + 0.787 \times (1.36 + 1.38 + 1.40 + 1.42 + 1.45)}$$

$$w_{seg\ 2} = 0.153$$

Twelve observed/reported crashes per year were reported in the curve. The expected number of crashes for the roadway segments is then calculated as follows:

$$N_{expected\ seg} = w \times N_{predicted} + (1 - w) \times N_{observed}$$

$$N_{expected\ seg\ 1} = 0.115 \times 1.57 + (1 - 0.115) \times 12$$

$$N_{expected\ seg1} = 10.8\ crashes/year$$

$$N_{expected\ seg\ 2} = 0.153 \times 1.40 + (1 - 0.153) \times 12$$

$$N_{expected\ seg\ 2} = 10.4\ crashes/year$$

Results of the analysis can be found in the sample spreadsheets provided with the *Highway Safety Manual User Guide*.

Table 70 presents a summary of the predictive method calculations. Columns 2 through 4 contain the predicted average crash frequency for total crashes, fatal-and-injury, and PDO. The fifth column contains the observed/reported number of crashes per year. Columns 6 and 7 contain the overdispersion parameter and weighted adjustment to be used to obtain the expected average crash frequency (last column).

TABLE 70
Example Problem 4 – Predicted, Expected, and Observed Crash Frequency Calculations Summary (2008 to 2012)

Site Type	Predicted Average Crash Frequency (crashes/year)			Observed/ Reported Crashes ($N_{observed}$) (crashes/ year)	Over-dispersion Parameter (k)	Weighted Adjustment (w) (Equation A-5 from HSM Part C Appendix A)	Expected Average Crash Frequency ($N_{expected}$) (Equation A-4 from HSM Part C Appendix A)
	$N_{predicted}$ (Total)	$N_{predicted}$ (Fatal-&-Injury)	$N_{predicted}$ (PDO)				
Roadway Segment 1	1.568	0.503	1.065	12	0.983	0.115	10.8
2008	1.522	0.488	1.033	12	0.983		
2009	1.545	0.496	1.049	12	0.983		
2010	1.568	0.503	1.065	12	0.983		
2011	1.591	0.511	1.080	12	0.983		
2012	1.615	0.518	1.097	12	0.983		
Roadway Segment 2	1.404	0.451	0.953	12	0.787	0.153	10.4
2008	1.362	0.437	0.925	12	0.787		
2009	1.383	0.444	0.939	12	0.787		
2010	1.403	0.450	0.953	12	0.787		
2011	1.424	0.457	0.967	12	0.787		
2012	1.446	0.464	0.982	12	0.787		

Details about the predictive method calculations can be found in the *Highway Safety Manual User Guide* spreadsheets. Comparison of the predicted and observed crash frequencies shows that the site is experiencing more crashes than the average site with similar characteristics.

Application of Mitigation Measures

The next step is to calculate the safety effects of the mitigation measures on the existing roadway. The existing curve gravel shoulders are 2 feet wide. The proposed improvements include paving the shoulders and increasing the width to 4 feet. Since the changes only involve shoulder type and width, all the other steps shown in the previous section are the same.

Shoulder Width and Type (CMF_{2r})

CMF_{2r} can be calculated using HSM Equation 10-2 (HSM p. 10-3). For this example, a 4-foot paved shoulder yields a CMF_{wra} of 1.15 (shoulder width HSM Table 10-9 [HSM p. 10-25]) and CMF_{tra} of 1.0 (shoulder type HSM Table 10-10 [HSM p. 10-26]). The percentage of related crashes is the same as that calculated for the lane width CMF:

$$CMF_{2r} = (CMF_{wra} \times CMF_{tra} - 1) \times p_{ra} + 1$$

$$CMF_{2r\ seg\ 1} = (1.15 \times 1 - 1) \times (0.574) + 1$$

$$CMF_{2r\ seg\ mit\ 1} = 1.09$$

Table 71 summarizes the results of the three scenarios. Roadway Segment 3 refers to the existing curve with the addition of mitigation measures to request the design exception.

TABLE 71
Example Problem 4 – Predicted, Expected, and Observed Crash Frequency Calculations Summary for the Three Scenarios (2008 to 2012)

Site Type	Predicted Average Crash Frequency (crashes/year)			Observed/ Reported Crashes ($N_{observed}$) (crashes/ year)	Over-dispersion Parameter (k)	Weighted Adjustment (w) (Equation A-5 from HSM Part C Appendix A)	Expected Average Crash Frequency ($N_{expected}$) (Equation A-4 from HSM Part C Appendix A)
	$N_{predicted}$ (Total)	$N_{predicted}$ (Fatal-&-Injury)	$N_{predicted}$ (PDO)				
Roadway Segment 1	1.568	0.503	1.065	12	0.983	0.115	10.80
2008	1.522	0.488	1.033	12	0.983		
2009	1.545	0.496	1.049	12	0.983		
2010	1.568	0.503	1.065	12	0.983		
2011	1.591	0.511	1.080	12	0.983		
2012	1.615	0.518	1.097	12	0.983		
Roadway Segment 2	1.404	0.451	0.953	12	0.787	0.153	10.37
2008	1.362	0.437	0.925	12	0.787		
2009	1.383	0.444	0.939	12	0.787		
2010	1.403	0.450	0.953	12	0.787		
2011	1.424	0.457	0.967	12	0.787		
2012	1.446	0.464	0.982	12	0.787		
Roadway Segment 3	1.444	0.463	0.980	12	0.983	0.123	10.70
2008	1.401	0.450	0.951	12	0.983		
2009	1.422	0.456	0.966	12	0.983		
2010	1.443	0.463	0.980	12	0.983		
2011	1.465	0.470	0.995	12	0.983		
2012	1.487	0.477	1.010	12	0.983		

The results, summarized in Table 72, indicate that the proposed curve will reduce the total expected crash frequency by about 4 percent (0.4 crash per year). The existing curve with mitigation measures reduces the total expected crash frequency by only 1 percent (0.1 crash per year).

TABLE 72
Example Problem 4 – Analysis Results Summary

Site Type	Length (miles)	$N_{observed}$	Crash Frequencies (crashes/year)	
			$N_{predicted}$	$N_{expected}$
Roadway Segment 1 – Existing Curve	0.24	12	1.6	10.8
Roadway Segment 2 – Proposed Curve	0.3	12	1.4	10.4
Roadway Segment 3 – Existing Curve with Mitigation Measures	0.24	12	1.4	10.7

Results and Discussion

The application of the HSM in the design stage provides engineers with valuable information in the decision-making process. NOTE: The HSM does not require agencies to implement specific alternatives based solely on safety performance evaluation but instead provides the means to make an informed decision.

The analysis conducted to determine the crash reduction impacts of upgrading the existing curve to comply with the current roadway design guidance indicates that more crashes are occurring at the site than the average site with similar characteristics. The predicted crash frequencies for curve Roadway Segments 1 and 2 are 1.6 and 1.4 crashes per year, respectively. However, the observed annual crash frequency for the site is 12 crashes per year.

After application of the EB adjustment, the expected crash frequency resulted in 10.8 and 10.4 crashes per year for Roadway Segments 1 and 2, respectively. To request a design exception, mitigation measures were applied to the existing curve Roadway Segment 1. Results of the application of the predictive method to the curve with mitigation measures show predicted and expected crash frequencies of 1.4 and 10.7 crashes per year, respectively.

For the analysis, the proposed curve alignment would reduce expected crash frequency by 4 percent, or 0.4 crashes per year. This reduction may not seem significant, so the analyst may need to look at other factors such as crash severity and specific collision types that are being addressed with the improved alignment or mitigation measures. In addition, this might be only one element of the entire corridor project, and significant differences may become obvious when reviewing the corridor as a whole.

Results may not always be favorable. A treatment (such as concrete median barriers) may increase the total crash frequency but reduce the severe crashes. Sound engineering judgment is ultimately the main driver of the decision-making process.

Results from this analysis offer engineers additional information to make an informed decision. The next step is to conduct an economic evaluation to determine the most cost-effective investment. Refer to HSM Chapter 7, Economic Appraisal, for methods to compare the benefits of potential crash countermeasures to crash costs.

3.3.3 Example Problem 5: Intersection Skew Angle

Introduction

A four-leg stop-controlled intersection on a rural multilane highway has a leg on the minor road with a skew angle of 40 degrees. Due to an increase in crash frequency at this location, the local jurisdiction has considered removing the skew angle (perpendicular). They would like to assess the potential change in expected average crash frequency.

Objectives

This example is focused on determining the change in expected average crash frequency as a result of realigning an intersection approach. The problem shows how to apply a CMF from the HSM Part D.

After reviewing this example, the user should be able to:

- Understand what input data are required to apply the HSM Part D procedures
- Calculate the change in expected average crash frequency using the HSM

- Understand how to reasonably interpret the results from an HSM analysis, and how these results can be used to support a particular decision

Data Requirements

The existing skew angle is 40 degrees. The expected average crash frequency for this site is 12 crashes per year. The applicable CMF is calculated using HSM Equation 14-3 (HSM p. 14-19). The CMF applies to total intersection crashes:

- Expected average crash frequency: 12 crashes per year
- Existing skew angle: 40 degrees

Analysis

The first step in the analysis is calculating the CMF for the existing condition. The skew angle is 40 degrees. The skew angle CMF is calculated using the following equation:

$$CMF = \frac{0.053 \times skew}{(1.43 + 0.053 \times skew)} + 1$$

$$CMF = \frac{0.053 \times 40}{(1.43 + 0.053 \times 40)} + 1 = 1.60$$

Then calculate the CMF for the after condition. The skew angle is 0 degrees:

$$CMF = \frac{0.053 \times skew}{(1.43 + 0.053 \times skew)} + 1 = 1.00$$

$$CMF = \frac{0.053 \times 0}{(1.43 + 0.053 \times 0)} + 1 = 1.00$$

The treatment CMF is then calculated by dividing the CMF for the after condition by the CMF for the existing condition:

$$CMF_{\ treatment} = \frac{1.00}{1.60} = 0.63$$

This result is used to quantify the difference between the existing condition and the change after the application of the treatment. The CMF treatment is applied to the expected crash frequency without the treatment:

$$Expected\ crashes\ after\ treatment = 0.63 \times 12 = 7.5\ \frac{crashes}{year}$$

Lastly, the change between the expected average crash frequency with and without treatment is calculated:

$$Change\ expected\ crashes = 12.0 - 7.5 = 4.5\ \frac{crashes}{year}\ reduced$$

Results and Discussion

The example shows how to compute the change in expected average crash frequency after implementation of a treatment. The reduction in skew angle from 40 degrees to 0 degrees yielded a reduction of 4.5 crashes per year. This CMF did not have a standard error available; therefore, a confidence interval for the reduction could not be calculated.

3.3.4 Example Problem 6: Deceleration Ramp Lengthening

Introduction

As part of a rehabilitation project, a local jurisdiction is considering to make improvements to an urban grade-separated diamond interchange. One of the improvements is the lengthening of an existing eastbound off-ramp deceleration lane. The current length is 450 feet, which is planned to be lengthened by 350 feet. The engineers would like to assess the change in average crash frequency by implementing this improvement.

Objectives

The following example is focused on determining the change in average crash frequency caused by a deceleration ramp lengthening. The problem shows how to apply a CMF from HSM Part D. After reviewing this example, the user should be able to:

- Understand what input data are required for applying HSM Part D procedures
- Calculate the change in crash frequency and apply standard error using the HSM
- Understand how to reasonably interpret the results from an HSM analysis and how these results can be used to support a particular decision

Data Requirements

The existing ramp 5-year average crash frequency is 19 crashes per year. The applicable CMF can be found in HSM Table 15-4, Potential Effect of Extending Deceleration Lanes (HSM p. 15-6). The CMF applies to all collision and severity types. The desired level of confidence for this example is 95 percent:

- The CMF is 0.93
- CMF standard error is 0.06

Analysis

The first step in the analysis is to calculate the 95th percentile confidence interval estimation of crashes with the treatment in place by using HSM Equation 3-8 (HSM p. 3-22):

$$Confidence\ Interval\ (CI\%) = [CMF \pm (SE \times MSE)]$$

where:

CMF = the crash modification factor to be applied
SE = the standard error of the CMF
MSE = the multiple of standard error for the desired level of confidence

A low desired level of confidence yields a confidence interval of 65 to 70 percent; a medium desired level of confidence yields a confidence interval of 95 percent; and a high desired level of confidence yields a confidence interval of 99.9 percent. HSM Section 3.5.3 (HSM p. 3-19) provides details about CMFs and detailed explanation of standard errors.

Then, the estimation of crashes with the treatment in place is calculated as follows:

$$Crashes\ with\ treatment = [CMF \pm (SE \times 2)] \times crashes/year$$

$$Crashes\ with\ treatment = [0.93 \pm (0.06 \times 2)] \times 19 = 15.39\ or\ 19.95\ crashes/year$$

An MSE value of 2 yields a 95-percent probability that the true value is between 15.39 and 19.95 crashes per year. The change in average crash frequency is calculated as follows:

$$Low\ Estimate = 19.95 - 19.00 = 0.95\frac{crashes}{year}increase$$

$$High\ Estimate = 19.00 - 15.39 = 3.61\frac{crashes}{year}decrease$$

Results and Discussion

The range of values suggests that lengthening the deceleration ramp by 350 feet may potentially increase, decrease, or cause no change in the average crash frequency at the study location.

3.4 HSM in Operations and Maintenance

3.4.1 Overview

Agencies are responsible for providing a reasonably safe and efficient transportation system for users on their daily operations. Typical operation activities include minimizing recurring congestion, managing incidents, weather-related events, work zones, handling special events, and managing daily traffic operations of the roadway network. Typical maintenance activities include improving pavements, roadside elements, and bridge facilities. The HSM provides users data-driven and science-based methods to supplement system monitoring, identify opportunities for improvement, and assess safety impacts of operations and maintenance activities.

Examples of the HSM application to improve operations include changes in signal timing, addition of passing lanes, and addition of left- and right-turn lanes. Systemic safety treatments can also be evaluated. Maintenance improvements such as signs, guardrail, and lighting upgrades and work zone closures can also be quantified by applying the HSM tools.

3.4.2 Example Problem 7: Adding Protected Left Turn Phases

Introduction

An urban four-leg signalized intersection with permissive left-turn phases in all four approaches is experiencing left-turn queuing issues on the major road. The city is evaluating the addition of exclusive left-turn phases on the major road and would like to assess the change in expected average crash frequency due to this improvement.

Objectives

The following example is focused on determining the change in average crash frequency caused by the addition of protected left-turn phases on the major road. The problem shows how to apply a CMF from the HSM Part D.

After reviewing this example, the user should be able to:

- Understand what input data are required for applying HSM Part D procedures
- Calculate the change in crash frequency using the HSM Part D
- Understand how to reasonably interpret the results from an HSM analysis and how these results can be used to support a particular decision

Data Requirements

The signalized intersection expected average crash frequency is 28 crashes per year. The intersection has four permissive left-turn phases, and the improvement considers upgrading the major movement approaches to protected left turn.

Analysis

The first step in the analysis is calculating the CMF for the existing condition. The permissive left-turn phase CMF is equal to 1.00 (HSM Table 14-24 [HSM p. 14-36]). The CMF for left-turn phasing is applied to each approach and multiplied together. The existing CMF for the intersection is calculated as follows:

$$CMF_{existing} = CMF_{approach\ 1} \times CMF_{approach\ 2} \times CMF_{approach\ 3} \times CMF_{approach\ 4}$$

$$CMF_{existing} = 1.00 \times 1.00 \times 1.00 \times 1.00 = 1.00$$

The next step is to calculate the CMF for the after condition. The protected left-turn phase CMF is 0.94 for each protected approach (HSM Table 14-24 [HSM p. 14-36]):

$$CMF_{future} = 0.94 \times 0.94 \times 1.00 \times 1.00 = 0.88$$

The treatment CMF is then calculated by dividing the future CMF by the existing CMF:

$$CMF_{\ treatment} = \frac{0.88}{1.00} = 0.88$$

This result is used to quantify the difference between the existing and future condition after the application of the treatment. The CMF treatment is applied to the expected crash frequency without the treatment:

$$Expected\ crashes\ after\ treatment = 0.88 \times 28 = 24.7\ \frac{crashes}{year}$$

Lastly, the change between the expected average crash frequency with and without treatment is calculated as follows:

$$Change\ expected\ crashes = 28.0 - 24.7 = 3.3\ \frac{crashes}{year}\ reduction$$

Results and Discussion

The example shows how to compute the change in expected average crash frequency after the implementation of a treatment. The change of left-turn signal phasing from permissive to protected-only in the major road led to a reduction of 3.3 crashes per year. The CMF did not have a standard error available; therefore, a confidence interval for the reduction could not be calculated.

3.4.3 Example Problem 8: Work Zone Analysis

Introduction

Predicting crashes under existing and proposed conditions can be challenging if the site is very different from base conditions and the calculations may be more complicated if the site is under construction. There are many factors that may be considered to predict crashes in a work zone. These may include work zone length, work zone duration, type of construction work (reconstruction, rehabilitation, etc.), contracting limitations (area available for contractor work operations), construction season (winter, spring), available right-of-way (available shoulder width), barrier type (drums, concrete barriers). The HSM simplifies the potential factors by focusing on the work zone length and duration.

The HSM provides two work zone crash modification factors (CMFs) that take into account work zone length and duration. Although more information is needed for a comprehensive work design, the following example is intended to illustrate the use of such CMFs and illustrate how maintenance of traffic (MOT) designers can obtain additional information to make informed decisions during the work zone design process

Example

A 5-mile rural freeway corridor is scheduled to undergo rehabilitation. The MOT team is designing the work zone layout and assessing the likely change in crash frequency between three work zone length and duration scenarios. The scenarios under consideration include constructing the overlay using one 5-mile work zone in 60 days; the second scenario involves two 2.5-mile work zones with a total duration of 90 days; and the third scenario involves five 1-mile work zone sections with a total duration of 120 days.

Objectives

This example is focused on determining the change in average crash frequency as a result of the increase of work zone length and duration. The example shows how to apply a CMF from HSM Part D.

After reviewing this example, the user should be able to:

- Understand what input data are required for applying HSM Part D procedures.
- Calculate the change in crash frequency using these CMFs.
- Understand how to interpret the results from CMF calculations to support a particular decision.

Data Requirements

The sensitivity analysis scenarios include work zones of 5-, 2.5-, and 1-mile lengths with durations of 60, 90, and 120 days, respectively. The corridor expected average crash frequency under base conditions is 4.0 crashes per year.

Analysis

The first step in the analysis is calculating the CMF for the increase in work zone length using HSM Equation 16-2 (HSM p. 16-7).

$$CMF_{length} = 1 + \frac{\%increase\ in\ length \times 0.67}{100}$$

$$CMF_{5\ mile} = 1 + \frac{880\ \times 0.67}{100} = 6.90$$

$$CMF_{2.5\ mile} = 1 + \frac{390\ \times 0.67}{100} = 3.61$$

$$CMF_{1\ mile} = 1 + \frac{96\ \times 0.67}{100} = 1.64$$

The next step is to calculate the CMF for increase in work zone duration using HSM Equation 16-1 (HSM p. 16-6):

$$CMF_{duration} = 1 + \frac{\%increase\ in\ duration \times 1.11}{100}$$

$$CMF_{60\ days} = 1 + \frac{275\ \times 1.11}{100} = 4.05$$

$$CMF_{90\ days} = 1 + \frac{463\ \times 1.11}{100} = 6.13$$

$$CMF_{120\ days} = 1 + \frac{650\ \times 1.11}{100} = 8.22$$

Next, calculate the combined effect of work zone length and duration under the proposed work zone condition:

$$CMF\ _{total} = CMF_{length} \times CMF_{duration}$$

$$CMF\ _{Scenario\ 1} = 6.90 \times 4.05\ = 27.96$$

$$CMF\ _{Scenario\ 2} = 3.61 \times 6.13\ = 22.17$$

$$CMF\ _{Scenario\ 3} = 1.64 \times 8.22\ = 13.50$$

This result is used to quantify the expected number of crashes under the proposed work zone scenario:

$$Expected\ crashes\ Scenario\ 1\ _{5\ mile|60\ days} = 27.96 \times 4 = 111.8\ \frac{crashes}{year}$$

$$Expected\ crashes\ Scenario\ 2\ _{2.5\ mile|90\ days} = 22.17 \times 4 = 88.7\ \frac{crashes}{year}$$

$$Expected\ crashes\ Scenario\ 3\ _{1\ mile|120\ days} = 13.5 \times 4 = 54.0\ \frac{crashes}{year}$$

Lastly, the change in expected crash frequency under the proposed work zone scheme is calculated as follows:

$$Change\ expected\ crashes\ Scenario\ 1 = 111.8 - 4.0 = 107.8\ \frac{crashes}{year}\ increase$$

$$Change\ expected\ crashes\ Scenario\ 2 = 88.7 - 4.0 = 84.7\ \frac{crashes}{year}\ increase$$

$$Change\ expected\ crashes\ Scenario\ 3 = 54.0 - 4.0 = 50.0\ \frac{crashes}{year}\ increase$$

Results and Discussion

The work zone example shows how to compute the change in expected average crash frequency for three proposed work zone scenarios. The results indicate that the varying conditions of the scenarios are likely to increase the crash frequency significantly. From the results, Scenario 1 with the longest work zone length and shorter duration yields the highest CMF_{length} and a lowest $CMF_{duration}$. However, when combined, the total CMF for Scenario 1 (5 miles, 60 days) yields highest annual average crash frequency with respect to the base condition among other scenarios. Similarly, Scenario 3 (1 mile, 120 days) yields lowest annual average crash frequency with respect to the base condition among other scenarios.

As a result, the combined effect of CMF related to length and duration yields to the lowest increase in the expected annual average crashes. The standard errors for these CMFs were not available; therefore, a confidence interval in the estimate could not be calculated.

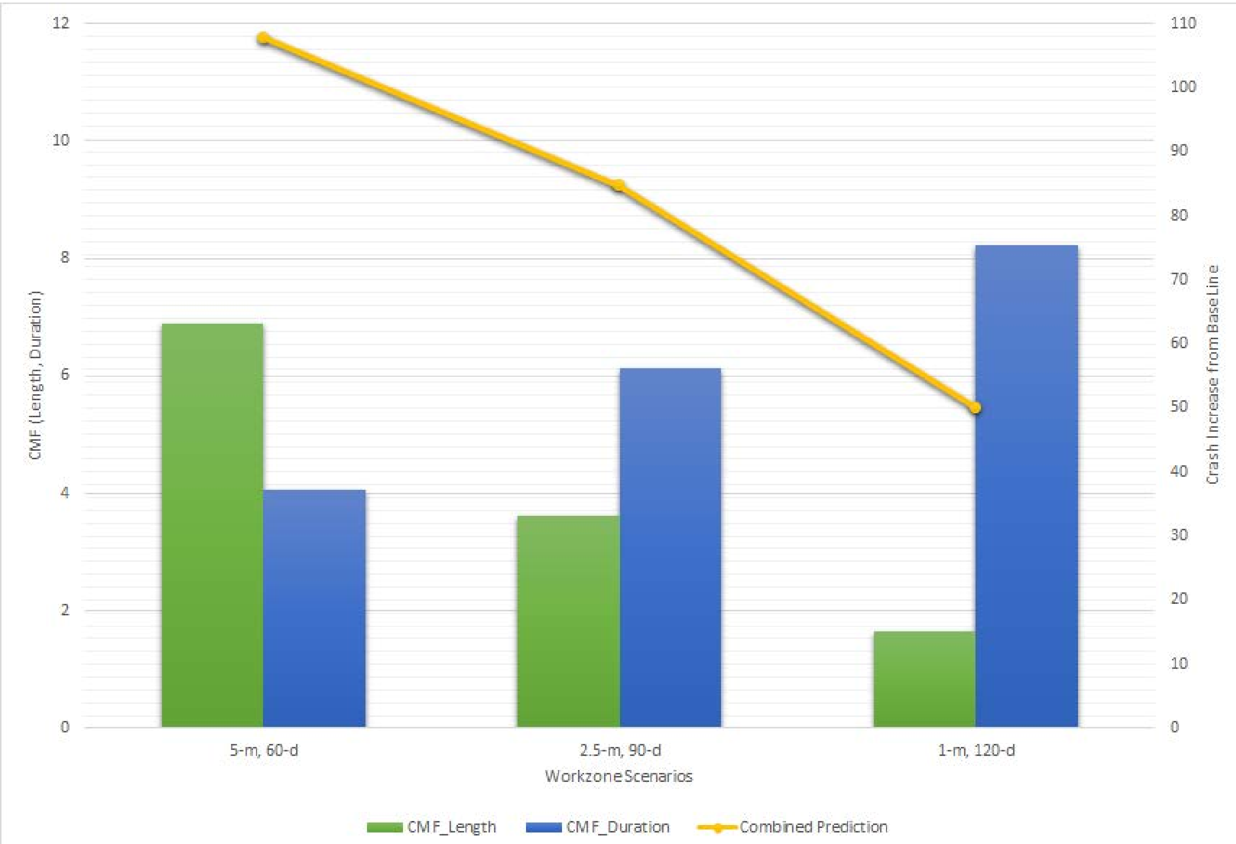


Figure 27: CMF Related to Length and Duration of Work Zone with Expected Annual Average Crash Increase for Work Zone Scenarios

Although Scenario 1 having 5-mile of length (farthest from the baseline CMF for length) and 60-day of duration (closest from the baseline CMF for duration), this option yields the highest expected annual average crash increase (108 crashes per year from the baseline). On the contrary, Scenario 3 having 1-mile of length (closest from the baseline CMF for length) and 120-days of duration (farthest from baseline CMF for duration) yields the lowest expected annual average crash increase (50 crashes per year) among other scenarios. With the respect to baseline work zone conditions (CMF of 1.0 for length and duration 1.0), Scenario 3 yields the lowest expected annual average crash increase among all other scenarios.

3.5 HSM Part D: CMF Applications Guidance

3.5.1 Overview

The HSM Part D provides information on the effectiveness of various safety treatments that can be used to estimate how effective how effective will be in reducing crashes at a specific location. This effectiveness is expressed in terms of CMFs values, trends, or no effect. The CMFs can be used to evaluate the expected average crash frequency with or without a particular treatment, or they can be used to estimate the expected average crash frequency with one treatment versus a different treatment. CMFs are provided for roadway segments (HSM Chapter 13), intersections (HSM Chapter 14), interchanges (HSM Chapter 15), special facilities and geometric situations (HSM Chapter 16), and roadway networks (HSM Chapter 17).

Part D includes all CMFs in the HSM. Some Part D CMFs are included in Part C for use with specific SPFs. The remaining Part D CMFs can be used with the outcomes of the predictive method to estimate the change in crash frequency described in HSM Section C.7 (HSM p. C-19).

3.5.2 Example Problem 9: Centerline Rumble Strips and Markings

Introduction

A safety engineer needs to select an appropriate countermeasure to reduce the roadway departure crashes on a rural two-lane highway segment. Candidate treatments for the roadway segment include centerline rumble strips and centerline markings. The safety engineer wants to find the change in average crash frequency for both countermeasures. Based on the Part C predictive method, the average injury and PDO crashes for the segment without treatment are 24 and 76 crashes per year, respectively, of which 15 and 49 are roadway departure crashes, respectively.

Data Requirements

The data requirements for this example are as follows:

- Average crash frequency without treatment

Analysis

The first step for this example is to determine whether the CMFs for relevant treatments can be determined using Part D of the HSM. Based on HSM Tables 13-34 and 13-43 (HSM p. 13-32 and 13-37), the CMFs for centerline rumble strips and centerline markings on rural two-lane highway segments are both available. The confidence interval, defined by the CMF plus or minus two times the standard error (95-percent confidence interval/MSE = 2), will also be used here to consider the possible range of safety effects of the treatment.

The CMFs and standard errors for centerline markings are listed in HSM Table 13-38 (HSM p. 13-34). The CMFs for injury crashes and PDO crashes are 0.99 and 1.01 with the corresponding standard errors being 0.06 and 0.05, respectively. However, no CMFs were provided for roadway departure crashes (head-on and opposite-direction sideswipe crashes) specifically. Tables 73 and 74 list the CMF applications for centerline markings.

TABLE 73
Example Problem 9 – CMF Applications – Centerline Markings

Expected Average Crash Frequency	**Fatal-and-Injury**	**PDO**	**Total**			
All Crash Types	24	76	100			
Run-off-the-Road Crashes	15	49	64			
CMF – Centerline Markings	**CMF**	**SE**				
Centerline Markings – Fatal-and-Injury	0.99	0.06				
Centerline Markings - PDO	1.01	0.05				
	Injury	**PDO**				
Expected Crashes with Treatment	23.76	76.76				
Change in Expected Crashes	-0.24	0.76				
Confidence Intervals	**Low**	**High**				
Fatal-and-Injury	0.87	1.11				
PDO	0.91	1.11				
95th Percentile Confidence Interval Estimation of Crashes			**Confidence Interval Range Variation**			
	Low	**High**	**Δ Low**	**Δ High**	**Δ % Low**	**Δ % High**
Fatal-and-Injury	20.88	26.64	-3.12	2.64	-13%	11%
PDO	69.16	84.36	-6.84	8.36	-9%	11%

$$Confidence\ Interval\ (CI\%) = [CMF \pm (SE \times MSE)]$$

The CMFs and the relevant standard errors for centerline rumble strips on rural two-lane highway are listed in HSM Table 13-46 (HSM p. 13-40). It should be noted that the values are provided for both total crashes and roadway departure crashes (head-on and opposite-direction sideswipe crashes) at different severity levels.

TABLE 74
Example Problem 9 – CMF Applications – Centerline Rumble Strips Part 2

CMF – Centerline Rumble Strips	**CMF**	**SE**		
Rumble Strips – All Severities	0.86	0.05		
Rumble Strips – Injury	0.85	0.08		
Run-off-the-Road – All Severities	0.79	0.1		
Run-off-the-Road – Injury	0.75	0.2		
	All Crash Types		**Run-Off-Road**	
	All Severities	**Injury**	**All Severities**	**Injury**
Expected Crashes with Treatment	86	20.4	50.6	11.3
Change in Expected Crashes	-14	-3.6	-13.4	-3.8
Confidence Intervals	**Low**	**High**		
All Severities	0.76	0.96		
Injury	0.69	1.01		
Run-off-the-Road – All Severities	0.59	0.99		

TABLE 74
Example Problem 9 – CMF Applications – Centerline Rumble Strips Part 2

Run-off-the-Road – Injury	0.35	1.15				
95th percentile Confidence Interval Estimation of Crashes			**Confidence Interval Range Variation**			
	Low	**High**	**Δ Low**	**Δ High**	**Δ % Low**	**Δ % High**
All Severities	76.0	96.0	-24	-4	-24%	-4%
Injury	16.6	24.2	-7.44	0.24	-31%	1%
Run-off-the-Road – All Severities	37.8	63.4	-26.2	-0.6	-41%	-1%
Run-off-the-Road – Injury	5.3	17.3	-9.8	2.3	-65%	15%

Results and Discussion

Based on the CMFs in HSM Table 13-38, the fatal-and-injury crashes will decrease 0.24 crash per year and PDO crashes will increase 0.76 crash per year after placing the centerline markings.

With the application of the centerline rumble strips and based on HSM Table 13-46, the total crashes and fatal-and-injury crashes will decrease 14 and 3.6 per year, respectively; and for roadway departure crashes, the total crashes and fatal-and-injury crashes will decrease 13.4 and 3.8, respectively.
The centerline rumble strip would be more effective in reducing all crash types and roadway departure crashes if the crash frequency reduction was the only metric considered.

For the centerline markings, the confidence intervals are 0.87 and 1.11 for fatal-and-injury crashes, and 0.91 and 1.11 for PDO crashes. These results indicate that the centerline markings could result in an increase, decrease, or no change in crashes.

For centerline rumble strips, the confidence intervals for fatal-and-injury and all crashes are 0.69 to 1.01 and 0.76 to 0.96, respectively, when all crash types are considered. The confidence intervals for fatal-and-injury crashes and all crashes are 0.35 to 1.15 and 0.59 to 0.99, respectively, if only roadway departure crashes are considered. The results also indicate that for all fatal-and-injury crashes and roadway departure fatal-and-injury crashes, the centerline rumble strips could result in an increase, decrease, or no change in crashes.

The 95th percentile confidence interval estimation of crashes was calculated to show the range of results expected by applying the two different CMFs. After placing the centerline markings, the low and high ranges for fatal-and-injury crashes are 20.88 and 26.64 crashes, respectively. These represent a reduction of 13 percent and an increase of 11 percent, respectively, of the expected average fatal-and-injury crash frequency.

Similarly, after the application of centerline rumble strips, the fatal-and-injury crashes are expected to fluctuate between 16.6 and 24.2 (low and high range). These represent a reduction of 31 percent and an increase of 1 percent from the expected average fatal-and-injury crash frequency.

The results show that by using a 95th percentile confidence interval, the application of centerline rumble strips is more likely to reduce the number of crashes.

Understanding the standard error and reliability of the different CMFs will help analysts build awareness of what can be expected from each safety treatment. A CMF with a high standard error does not mean that it should not be used; it means that the analyst should keep in mind the range of CMF results that could be obtained.

3.5.3 Example Problem 10: Improving Urban Four-Leg Signalized Intersection

Introduction

An urban four-leg signalized intersection was identified as candidate for roadway modifications after applying the roadway safety management process on the selected roadway network. The safety engineer requested a list of possible treatments for the intersection that have specific CMF and standard error values.

Data Requirements

No additional data will be required for this example.

Analysis

Tables 14-1 (HSM p. 14-5), 14-8 (HSM p. 14-14), 14-9 (HSM p. 14-15), and 14-19 (HSM p. 14-30) in Chapter 14 of HSM Part D list the crash effects of intersection types, access management, intersection design elements, and intersection traffic control and operational elements. Those roadway modifications that have CMF and standard error values could be further identified from these tables.

Results and Discussion

Table 75 lists the treatments for which the CMFs were provided in HSM Part D. The safety engineer could use these treatments as the first step for identifying the appropriate roadway modifications for the intersection.

TABLE 75
Example Problem 10 – Intersection Treatment Summary

Treatment Category	Treatment
Intersection type	Convert signalized intersection to a modern roundabout
	Remove unwarranted signal on one-way streets
Access management	Not applicable
Intersection design elements	Provide a left-turn lane on approaches to four-leg intersections
	Provide a right-turn lane on approaches to an intersection
	Increase intersection median width
	Provide intersection lighting
Intersection traffic control and operational elements	Prohibit left-turns and/or U-turns with NO LEFT TURN and NO U-TURN signs
	Modify left-turn phase
	Modify change and clearance interval
	Install red-light cameras

Appendices

Appendix A: References

Crash Modification Factors (CMF) Clearinghouse: http://www.cmfclearinghouse.org.

FHWA Training Courses: http://nhi.fhwa.dot.gov.

Highway Safety Manual website: www.highwaysafetymanual.org.
Purchase the HSM: http://bookstore.transportation.org; search under code HSM-1.

Interactive Highway Safety Design Module (IHSDM) website:
http://www.tfhrc.gov/safety/ihsdm/ihsdm.htm.

NCHRP Research Results Digest 329: www.trb.org/Publications/Blurbs/Highway_Safety_Manual_Data_Needs_Guide_159984.aspx.

SafetyAnalyst website: http://www.safetyanalyst.org.

Appendix B: Glossary

This chapter defines the terms used in the *Highway Safety Manual User Guide*.

Empirical Bayes Method: Method in which the evidence about the true state of the world is expressed in terms of degrees of belief (Bayesian probabilities). This method incorporates knowledge from history or other sites to obtain the best estimation. Then, the method considers the likelihood of certain types of events as part of the analysis process. Last, the method uses the Bayes theorem to convert probabilistic statements into degrees of belief instead of the traditional confidence interval interpretation.

Crash: The HSM definition of a crash is a set of events that result in injury or property damage due to the collision of a motorized vehicle with another motorized vehicle, bicyclist, pedestrian, or an object.

Crash Estimation: The term crash estimation is related to the methodology used to predict the crash frequency of a existing roadway for existing conditions or alternative conditions during a past or future period, or to predict the crash frequency of a new roadway for given conditions for a future period.

Crash Evaluation: The term crash evaluation relates to determining the effectiveness of a particular treatment or treatment program after its implementation.

Crash Frequency: The HSM definition of crash frequency is the number of crashes occurring at a particular site, facility, or network in one year. Crash frequency is calculated as the number of crashes divided by the period in years, and the unit is number of crashes per year.

Crash Severity: See KABCO definition.

KABCO: Crash severity scale, which provides five levels of injury severity. Even if the KABCO scale is used, the definition of an injury may vary between jurisdictions. The five KABCO crash severity levels are: Fatal injury (K): an injury that results in death; Incapacitating injury(A): any injury, other than a fatal injury, that prevents the injured person from walking, driving, or normally continuing the activities the person was capable of performing before the injury occurred; Non incapacitating evident injury (B): any injury, other than a fatal injury or an incapacitating injury, that is evident to observers at the scene of the crash in which the injury occurred; Possible injury (C): any injury reported or claimed that is not a fatal injury, incapacitating injury, or non-incapacitating evident injury and includes claim of injuries not evident; No injury/property damage only (O; also known as PDO).

Effectiveness: The term effectiveness refers to a change in the predicted average crash frequency or severity for a site or project.

Expected Average Crash Frequency: This term is used to describe the average crash frequency, under a given set of geometric design and traffic volumes for a given time period, of a site or network.

Observed Average Crash Frequency: This is the historical average crash frequency at a given site.

Predicted Average Crash Frequency: This is the average crash frequency at a site or network obtained with the application of an SPF for the study period under the given conditions.

Predictive Method: This term refers to the HSM Part C methodology used to estimate the long-term average crash frequency of a site or network under given geometric design and traffic volumes for a specific number of years. The result from the predictive method is the expected crash frequency.

Appendix C: Frequently Asked Questions

Q1: What if my site is not exactly like a site in the HSM?

A: The site under investigation should follow strictly with the facility types described in HSM because any minor differences will significantly affect the calculated crash frequency.

Q2: Should minimum segment lengths be established for use in HSM Part C analyses? HSM Part B analyses?

A: There is no minimum segment length necessary for use in HSM Part C analyses to estimate the predicted crash frequency (N_p). The procedures have been developed so they can be applied to homogeneous segments as long or short as necessary. If a project being analyzed includes numerous segments shorter than 0.1 mile, consideration might be given to using the project-level Empirical Bayes (EB) procedure rather than the site-specific EB procedure to determine the expected crash frequency (N_e), because the locations of observed/reported crashes may not be sufficiently accurate for application of the site-specific EB procedure. The site-specific and project-level EB procedures are presented in HSM Part C Appendices A.2.4 and A.2.5, respectively.

There are no explicitly prescribed procedures for HSM Part B analyses. Highway agency databases with roadway characteristics often have many short segments because, whenever any of the many data elements in such data sets changes, a new roadway segment begins. Longer segments can be used for network screening.

Q3: When average annual daily traffic (AADT) values are estimated, results vary dramatically and data dispersion is extreme. Is using estimated AADT values (particularly on minor crossroads and rural, low-volume roads) a good idea in HSM Part C?

A: In general, the application of the predictive methods depends on the number of crashes and accurate AADT estimates. All AADT values are, to some extent, estimates unless a permanent count station is located on the site in question. Highway agencies generally have reasonable AADT estimates for roadway segments on the state highway system. AADT values are sometimes unavailable for local roads, including minor-road legs of intersections with state highways. In these cases, estimates need to be made to provide exposure data for crash analysis tools. Overall, the better the estimates made, the better the results that will be obtained.

Q4: What is the difference between observed, predicted, and expected average crash frequency?

The HSM predictive method can calculate both the **predicted** crash frequency and the expected crash frequency under different scenarios. The **predicted** average crash frequency of an individual site is the crash frequency calculated with the SPF and CMFs based on the geometric design, traffic control features, and traffic volume of the site. This method will be used when estimating the crash frequency for a past or future year or when the **observed** crash frequency is not available. The **observed** crash frequency refers to the historical crash data observed/reported at the site during the period of analysis.

When the **observed** crash frequency is available, the **expected** crash frequency can be calculated. The **expected** crash frequency uses the EB method to combine the **observed** crash frequency with the **predicted** average crash frequency to produce a more statistically reliable measure. A weighted factor is applied to both estimates; this reflects the statistical reliability of the SPF. The **expected** crash frequency is the long-term average crash frequency that would be expected from the specific site and is more statistically reliable as compared with the predicted crash frequency.

Q5: What is the difference between safety performance function (SPFs) for network screening versus SPFs for prediction?

A: SPFs used for network screening are more general and require less data than the predictive methods, and they allow agencies to identify high-priority locations for potential improvement. SPFs in Part C of the HSM are more specific. An example of a network screening SPF may be a rural two-lane road, while a Part C SPF may be a rural two-lane road under base conditions (6-foot shoulder and 12-foot lanes).

Q6: How much is gained in accuracy by using an agency-developed SPF rather than a calibrated SPF? Under what circumstances are calibrated SPFs satisfactory, and under what circumstances is there a clear advantage for agencies that develop their own SPFs?

A: The SPFs presented in HSM Part C, when calibrated to local conditions, should provide acceptable levels of accuracy for application of HSM Part C procedures. The HSM does not require that each agency develop their own SPFs, because a requirement for SPF development might become an impediment to highway agency implementation of the HSM. However, agency-developed SPFs should be even more accurate than calibrated SPFs from the HSM. As long as local SPFs are developed with properly applied statistical techniques, it is reasonable that statistical models developed with local data should be more accurate than models developed with data from elsewhere and calibrated to local conditions. Guidance for the development of SPFs with highway agency data has been provided in HSM Part C Appendix A.1.2, and more detailed guidance is being developed. (See highwaysafetymanual.org for additional information.) An expanded guide on SPF development is currently being prepared for a Federal Highway Administration (FHWA) project. In summary, use of SPFs presented in HSM Part C and calibrated to local conditions is acceptable; use of SPFs developed from an agency's own data using proper statistical techniques is also acceptable.

There can be no general quantitative answer as to how much better an agency-developed SPF will be in comparison to a calibrated SPF. This will vary on a case-by-case basis.

Q7: What if I do not have a local calibration factor for the HSM Part C SPFs? What is the effect on the result?

A: The user can let the calibration factor be the default value of 1.00 if no local calibration factor is available. However, the SPFs were developed based on crash data extracted from several states, and 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 other variables. The local calibration factor was developed to account for the differences on safety performance among different jurisdictions. The results calculated by the HSM Part C predictive models can be used as a relative comparison if not local calibration factor is available. See HSM Appendix A for details on calculating calibration factors. An HSM calibration guide is also being developed. (See highwaysafetymanual.org for more details.)

Q8: What is the difference between crash modification factors (CMFs) in the *Highway Safety Manual* Part C and Part D, and the CMF Clearinghouse?

A: The HSM provides highest quality available research-based CMFs, while the CMF Clearinghouse is a comprehensive listing of available CMFs. Part D includes all CMFs in the HSM. Some Part D CMFs are included in Part C for use with specific SPFs. Other Part D CMFs not included in Part C can be used in conjunction with the methods to estimate change in crash frequency provided in HSM Section C.7 (HSM p. C-19).

Q9: Is the lighting CMF (Equations 12-34 and 12-36) applicable even if the intersections/ roadway segments do not meet the highway lighting standards?

A: The current HSM considers the presence of roadway lighting on safety performance and does not account for the effects of luminance intensity. Therefore, the lighting CMF is applicable even if the intersections/roadway segments do not meet the highway lighting standards.

Q10: How can I justify the need of the HSM to management?

A: The FHWA has developed a number of resources to assist states with HSM implementation efforts. Please refer to the *HSM Implementation Guide for Managers* for additional information: http://safety.fhwa.dot.gov/hsm/hsm_mgrsguide/.

Q11: How can data gained from implementation of the HSM be a benefit to other areas of the department?

A: The FHWA has developed a number of resources to assist states with HSM implementation efforts. Please refer to the FHWA Office of Safety website for additional information: http://safety.fhwa.dot.gov/hsm/. The *HSM Implementation Guide for Managers* and *HSM Integration Guide* may provide additional information.

Q12: How should I best determine whether a roadway segment or intersection is rural or urban?

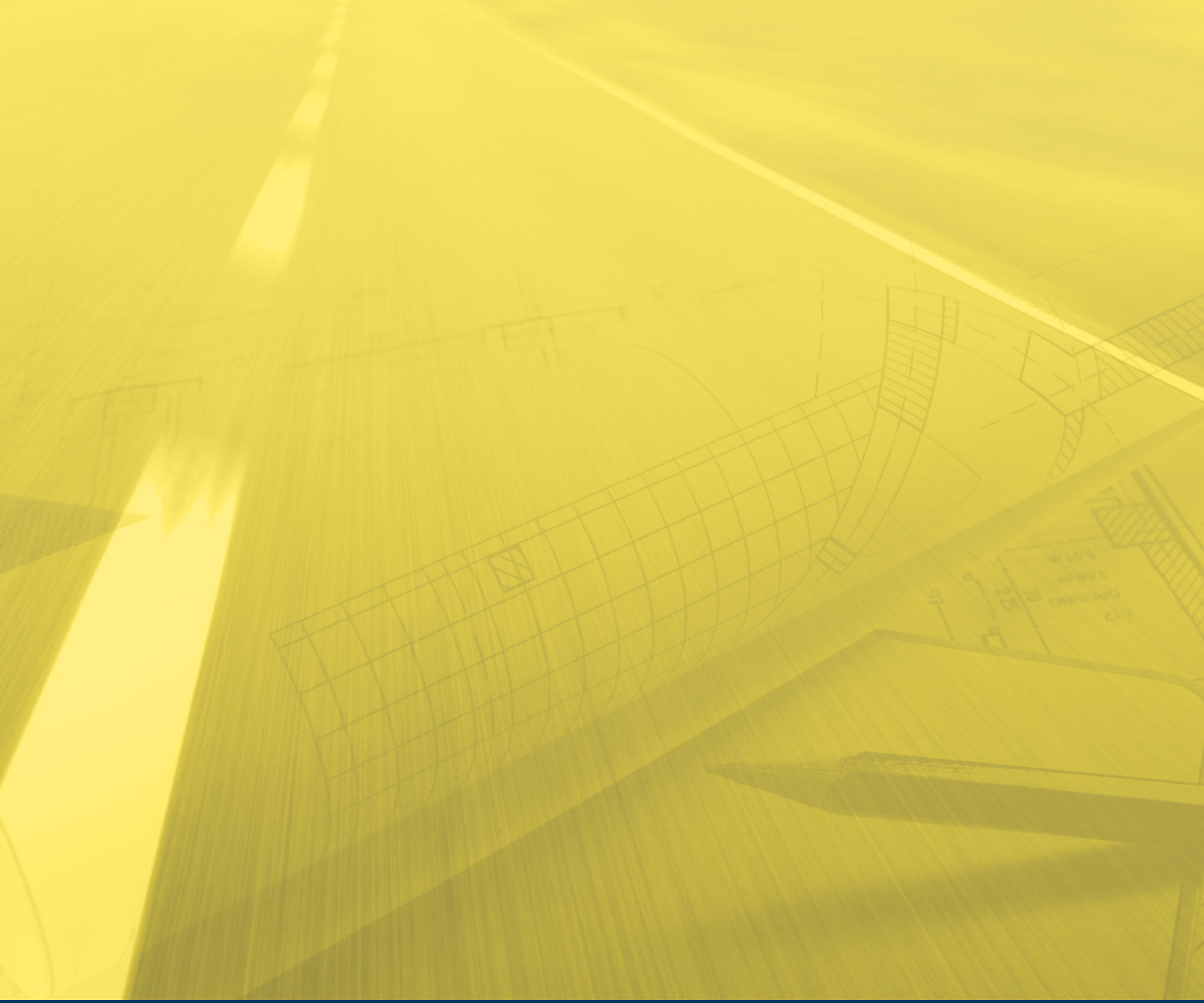
A: The definition of urban versus rural is established based on population. Any population center with less than 5,000 persons is considered rural. Typically, agencies have GIS shapefiles containing this information; therefore, segments or intersections can be assigned to one or another based on their location.

Q13: Where can I get additional information about how to calculate the roadside hazard rating?

A: Refer to HSM Appendix 13A (p. 13-59) for additional roadside hazard rating information.

Q14: Where can I get additional assistance?

A: HSM users are encouraged to visit the official website of the HSM at www.highwaysafetymanual.org and check information under User Discussion Forum.



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